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LRFD Cold-formed Steel Design Manual

American Iron and Steel Institute

LRFD COLD-FORMED STEEL DESIGN MANUAL

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PREFACE

This edition of the *LRFD Cold-Formed Steel Design Manual* is based on the *March 16, 1991 Edition of the Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members*. The *Manual* includes the following sections:

Part I—*Specification*

Part II—*Commentary*

Part III—*Supplementary Information*

Part IV—*Illustrative Examples*

Part V—*Charts and Tables*

Part VI—*Computer Aids*

Part VII—*Test Procedures*

The *Specification* and the *Commentary* are both also available as separately bound booklets.

American Iron and Steel Institute gratefully acknowledges the time and effort devoted to the preparation of the Manual by the Committee on Specifications for the Design of Cold-Formed Steel Structural Members and its working subcommittees. A special thanks go to the following: the Cold-Formed Steel Design Manual Subcommittee—P. A. Seaburg, Chairman; R. E. Brown, C. R. Clauer, E. R. diGirolamo, E. R. Estes, J. M. Fisher, R. S. Glauz, M. Golovin, R. B. Haws, R. E. Hodges, M. Johanningsmeler, D. L. Johnson, H. Klein, R. A. LaBoube, T. M. Murray, J. N. Nunnery, R. M. Schuster, and W. W. Yu; the Editorial Subcommittee—C. W. Pinkham, Chairman, C. R. Clauer, D. A. Cuoco, J. M. Fisher, and T. B. Pekoz; Clauer Associates, and R. J. Schrader.

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LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD–FORMED STEEL STRUCTURAL MEMBERS

MARCH 16, 1991 EDITION

LRFD Cold-Formed Steel Design Manual - Part I

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PREFACE

The American Iron and Steel Institute allowable stress design specification has long been used for the design of cold–formed steel structural members. The Load and Resistance Factor Design (LRFD) Specification has recently been developed from a research project sponsored by AISI at the University of Missouri–Rolla under the direction of Wei–Wen Yu with consultation of T.V. Galambos and initial contribution of M.K. Ravindra. In this LRFD Specification, separate load and resistance factors are applied to specified loads and nominal resistance to ensure that the probability of reaching a limit state is acceptably small. These factors reflect the uncertainties of analysis, design, loading, material properties and fabrication. They are derived on the basis of the first order probabilistic methodology as used for the development of the AISC Load and Resistance Factor Design Specification for Structural Steel Buildings.

This Specification contains six chapters of the LRFD recommendations for cold–formed steel structural members and connections. The background information for the design criteria is discussed in the Commentary and other related references.

AISI acknowledges the devoted efforts of the members of the Committee on Specifications for the Design of Cold–Formed Steel Structural Members. This group, comprised of consulting engineers, researchers, designers from companies manufacturing cold–formed steel members, components, assemblies, and complete structures, and specialists from the steel producing industry, has met two to three times per year since its establishment in 1973. Its current members, who have made extensive contributions of time and effort in developing and reaching consensus on this LRFD Specification are:

R. L. Brockenbrough, Chairman
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C.R. Clauer
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P.A. Seaburg
F.V. Slocum
D.L. Tarlton
D.S. Wolford*
W.W. Yu
A.S. Zakrezewski

*Past Chairman

The activities of the Committee are sponsored by AISI's Light Construction Subcommittee of the Construction Marketing Committee. The Specification is issued under the auspices of AISI's Committee on Construction Codes and Standards.

Users of the Specification are invited to continue to offer their valuable comments and suggestions. The cooperation of all involved, the users as well as the writers, is needed to continue to keep the Specification up to date and a useful tool for the designer.

American Iron and Steel Institute
March 16, 1991

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SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
A	Full unreduced cross–sectional area of the member	C3.1.1, C3.1.2, C4, C6.2, D4.1
A_b	$b_1t + A_s$, for transverse stiffeners at interior support and under concentrated load, and $b_2t + A_s$, for transverse stiffeners at end support	B6.1
A_b	Gross cross–sectional area of bolt	E3.4
A_c	$18t^2 + A_s$, for transverse stiffeners at interior support and under concentrated load, and $10t^2 + A_s$, for transverse stiffeners at end support	B6.1
A_e	Effective area at the stress F_n	C4, C6.2, D4.1
A_n	Net area of cross section	C2, E3.2
A_s	Cross–sectional area of transverse stiffeners	B4, B4.1, B4.2, B6.1
A'_s	Effective area of stiffener	B4, B4.1, B4.2
A_{st}	Gross area of shear stiffener	B6.2
A_{wn}	Net web area	E4
A_1	Bearing area	E5.1
A_2	Full cross sectional area of concrete support	E5.1
a	Shear panel length of the unreinforced web element. For a reinforced web element, the distance between transverse stiffeners	B6.2, C3.2
a	Length of bracing interval	D3.2
B	Stud spacing	D4, D4.1
B_c	Term for determining the tensile yield point of corners	A5.2.2
b	Effective design width of compression element	B2.1, B2.2, B2.3, B3.1, B3.2, B4.1, B4.2, B5
b	Flange width, Z–section	D3.2.1
b_d	Effective width for deflection calculation	B2.1, B2.2
b_e	Effective design width of sub–element or element	A1.2, B2.3, B5
b_o	See Figure B4.1	B4, B4.1, B5
C	For flexural members, ratio of the total corner cross –sectional area of the controlling flange to the full cross –sectional area of the controlling flange	A5.2.2
C_b	Bending coefficient dependent on moment gradient	C3.1.2
C_m	End moment coefficient in interaction formula	C5

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
C_{ms}	Coefficient for lateral bracing of Z–section	D3.2.1
C_{mx}	End moment coefficient in interaction formula	C5
C_{my}	End moment coefficient in interaction formula	C5
C_p	Correction Factor	F1
C_s	Coefficient for lateral torsional buckling	C3.1.2
C_{TF}	End moment coefficient in interaction formula	C3.1.2
C_{th}	Coefficient for lateral bracing of Z–sections	D3.2.1
C_{tr}	Coefficient for lateral bracing of Z–sections	D3.2.1
C_v	Shear stiffener coefficient	B6.2
C_w	Torsional warping constant of the cross–section	C3.1.2
C_y	Compression strain factor	C3.1.1
C_o	Initial column imperfection	D4.1
C_1	Term used to compute shear strain in wall board	B4, B4.1, D4.2
C_2	Coefficient as defined in Figure B4–2	B4, B4.2
c_f	Amount of curling	B1.1b
D	Outside diameter of cylindrical tube	C6, C6.1, C6.2, D4.2
D	Overall depth of lip	B1.1, B4, D1.1
D	Shear stiffener coefficient	B6.2
D	Nominal dead load	A5.1.4
D_o	Initial column imperfection	D4.1
d	Depth of section	B1.1b, B4, C3.1.1, C3.1.3, D1.1, D3.2.1, D4, D4.1
d	Width of arc seam weld	E2.3
d	Visible diameter of outer surface of arc spot weld	E2.2
d	Diameter of bolt	E3, E3.1, E3.2, E3.4
d_a	Average diameter of the arc spot weld at mid–thickness of t	E2.2
d_a	Average width of seam weld	E2.3
d_e	Effective diameter of fused area	E2.2
d_e	Effective width of arc seam weld at fused surfaces	E2, E2.3
d_h	Diameter of standard hole	B2.2, E3.1, E4
d_s	Reduced effective width of stiffener	B4, B4.2
d'_s	Actual effective width of stiffener	B4, B4.2
d_{wc}	Coped web depth	E4
E	Modulus of elasticity of steel (29.5×10^3 ksi)	B1.1b, B2.1, B6.1, C3.1.1, C3.2, C3.5, C4, C4.1, C5, C6.1, D1.2, D4.1, D4.2 , E2.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
E	Nominal earthquake load	A5.1.4
E_o	Initial column imperfection; a measure of the initial twist of the stud from the initial, ideal, unbuckled location	D4.1
E_1	Term used to compute shear strain in wallboard	D4.1
E'	Inelastic modulus of elasticity	D4.1
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 toward which the force is directed	E3.1
e_{min}	Minimum allowable distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed	E2.2
e_y	Yield strain = F_y/E	C3.1.1
F	Loads due to fluids	A5.1.4
F_e	Elastic buckling stress	C4, C4.1, C4.2, C4.3, C6.2, D4.1
F_m	Mean value of the fabrication factor	F1
F_n	Nominal buckling stress	C4, C6.2, D4.1
F_{nt}	Nominal tensile strength of bolts	E3.4
F_{nv}	Nominal shear strength of bolts	E3.4
F'_{nt}	Nominal tensile strength for bolts subject to combination of shear and tension	E3.4
F_{sy}	Yield point as specified in Sections A3.1 or A3.2	A3.1, A3.2, A3.3.2, E2.2, E3.1, E3.2
F_u	Tensile strength as specified in Sections A3.1 or A3.2, or as reduced for low ductility steel	A3.1, A3.2, A3.3, A3.3.2, E2.2, E2.3, E2.4,E2.5, E3.1, E3.2, E3.3, E4
F_{uv}	Tensile strength of virgin steel specified by Section A3 or established in accordance with Section F3.3	A3, A5.2.2, E2.2, F3.3
F_{wy}	Yield point for design of transverse stiffeners	B6.1
F_{xx}	Strength level designation in AWS electrode classification	E2.2, E2.3, E2.4, E2.5

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
F_y	Yield point used for design, not to exceed the specified yield point or established in accordance with Section F3, or as increased for cold work of forming in Section A5.2.2 or as reduced for low ductility steels in Section A3.3.2	A1.2, A3.3, A5.2.1, A5.2.2, B2.1, B5, B6.1, C2, C3.1, C3.1.1, C3.1.3, C3.2, C3.5, C4, C6.1, C6.2, D1.2, D4, D4.2, E2
F_{ya}	Average yield point of section	A5.2.2
F_{yc}	Tensile yield point of corners	A5.2.2
F_{yf}	Weighted average tensile yield point of the flat portions	A5.2.2, F3.2
F_{ys}	Yield point of stiffener steel	B6.1
F_{yv}	Tensile yield point of virgin steel specified by Section A3 or established in accordance with Section F3.3	A3, A5.2.2, F3.3
f	Stress in the compression element computed on the basis of the effective design width	B2.1, B2.2, B3.2, B4, B4.1
f_{av}	Average computed stress in the full, unreduced flange width	B1.1b
f_c	Stress at service load in the cover plate or sheet	D1.2
f'_c	Specified compression stress of concrete	E5.1
f_d	Computed compressive stress in the element being considered. Calculations are based on the effective section at the load for which deflections are determined.	B2.1, B2.2, B3.1, B4.1, B4.2
f_{d1}, f_{d2}	Computed stresses f_1 and f_2 as shown in Figure B2.3–1. Calculations are based on the effective section at the load for which deflections are deteremined	B2.3
f_{d3}	Computed stress f_3 in edge stiffener, as shown in Figure B4–2. Calculations are based on the effective section at the load for which deflections are determined	B3.2
f_v	Computed shear stress on a bolt	E4
f_1, f_2	Web stresses defined by Figure B2.3–1	B2.3
f_3	Edge stiffener stress defined by Figure B4–2	B3.2
G	Shear modulus of steel (11,300 ksi)	C3.1.1, D4.1
G'	Inelastic shear modulus	D4.1
g	Vertical distance between two rows of connections nearest to the top and bottom flanges	D1.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
H	Loads due to the weight and lateral pressure of soil and water in soil	A5.1.4
h	Depth of flat portion of web measured along the plane of web	B1.2, B6.2, C3.2, C3.4
I_a	Adequate moment of inertia of stiffener so that each component element will behave as a stiffened element	B1.1, B4, B4.1, B4.2
I_b	Moment of inertia of the full unreduced section about the bending axis	C5
I_s	Actual moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened	B1.1, B4, B4.1, B4.2, B5
I_{sf}	Moment of inertia of the full area of the multiple stiffened element, including the intermediate stiffeners, about its own centroidal axis parallel to the element to be stiffened	B5
I_x, I_y	Moment of inertia of full section about principal axis	D1.1, D3.2.2
I_{xy}	Product of inertia of full section about major and minor centroidal axes	D3.2.2, D4.1
I_{yc}	Moment of inertia of the compression portion of a section about the centroidal axis of the entire section parallel to the web, using the full unreduced section	C3.1.2
J	St. Venant torsion constant	C3.1.2
j	Section property for torsional–flexural buckling	C3.1.2
K	Effective length factor	C3.1.2, C4, C4.1, C5
K'	A constant	D3.2.2
K_b	Effective length factor in the plane of bending	C5
K_t	Effective length factor for torsion	C3.1.2
K_x	Effective length factor for bending about x–axis	C3.1.2
K_y	Effective length factor for bending about y–axis	C3.1.2
k	Plate buckling coefficient	B2.1, B2.3, B3.1, B3.2, B4 B4.1, B4.2
k_v	Shear buckling coefficient	B6.2, C3.2
L	Full span for simple beams, distance between inflection points for continuous beams, twice the length of cantilever beams	B1.1c, D3.2.1

SYMBOLS AND DEFINITIONS

Symbols	Definition	Section
L	Length of seam weld not including the circular ends	E2.3
L	Length of fillet weld	E2.4, E2.5
L	Unbraced length of member	C3.1.2, C4.1, D1.1, D4, D4.1
L	Nominal live load	A5.1.4
L_r	Nominal roof live load	A5.1.4
L_{st}	Length of transverse stiffener	B6.1
L_t	Unbraced length of compression member for torsion	C3.1.2
L_x	Unbraced length of compression member for bending about x–axis	C3.1.2
L_y	Unbraced length of compression member for bending about y–axis	C3.1.2
M_c	Critical moment	C3.1.2
M_e	Elastic critical moment	C3.1.2
M_m	Mean value of the material factor	F1
M_n	Nominal flexural strength	C3.1, C3.1.1, C3.1.2, C3.1.3, C6.1
M_{nx}, M_{ny}	Nominal flexural strengths about the centroidal axes determined in accordance with Section C3	C5
M_{nxo}, M_{nyo}	Nominal flexural strengths about the centroidal axes determined in accordance with Section C3.1 excluding the provisions of Section C3.1.2	C3.3, C3.5, D4.2, D4.3
M_u	Required flexural strength	C3.3, C3.5
M_{ux}	Required flexural strength about x–axis	C5
M_{uy}	Required flexural strength about y–axis	C5
M_y	Moment causing a maximum strain e_y	B2.1, C3.1
M_1	Smaller end moment	C3.1.2, C5
M_2	Larger end moment	C3.1.2, C5
m	Distance from the shear center of one channel to the mid–plane of its web	D1.1, D3.2.2
N	Actual length of bearing	D3.6
n	Number of holes	E4
n	Number of tests	F1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
n_p	Number of parallel purlin lines	D3.2.1
P	Loads, forces, and effects due to ponding	A5.1.4
P_E	$\pi^2 EI_b/(K_bL_b)^2$	C5
P_L	Force to be resisted by intermediate beam brace	D3.2.1
P_m	Mean value of the tested–to–predicted load ratios	F1
P_n	Nominal axial strength of member	C4, C6.2
P_n	Nominal strength of connection component	E2, E2.2, E2.3, E2.4, E2.5
P_{no}	Nominal axial strength of member determined in accordance with Section C4 for L = 0	C5
P_p	Nominal bearing capacity on concrete	E5.1
P_s	Concentrated load or reaction based on factored loads	D1.1
P_u	Required axial strength	C5
$\overline{Q}$	Design shear rigidity for sheathing on both sides of the wall assembly	D4.1
Q_i	Load effect	F1
q	Uniformly distributed factored load in the plane of the web	D1.1
$\overline{q}$	Design shear rigidity for sheathing per inch of stud spacing	D4.1
$\overline{q}_o$	Factor used to determine design shear rigidity	D4.1
R	Reduction Factor	C3.1.3
R	Coefficient	C4, C6.2
R	Inside bend radius	A5.2.2, C3.4
R_n	Nominal resistance	A1.2, F1
R_P	Average value of all test results	F1
R_r	Nominal roof rain load	A5.1.4
r	Radius of gyration of full unreduced cross section	C3.1.1, C4, C4.1
r	Force transmitted by the bolt or bolts at the section considered, divided by the tension force in the member at that section	E3.2
r_{cy}	Radius of gyration of one channel about its centroidal axis parallel to web	D1.1
r_I	Radius of gyration of I–section about the axis perp–pendicular to the direction in which buckling would occur for the given conditions of end support and intermediate bracing	D1.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
r_o	Polar radius of gyration of cross section about the shear center	C3.1.1, C4.2, D4.1
r_x, r_y	Radius of gyration of cross section about centroidal principal axis	C3.1.1
S	$1.28\sqrt{E/f}$	B4, B4.1
S	Nominal snow load	A5.1.4
S_c	Elastic section modulus of the effective section calculated at a stress M_c/S_f in the extreme compression fiber	C3.1.1, C3.1.2, C4
S_e	Elastic section modulus of the effective section calculated with extreme compression or tension fiber at F_y	C3.1.1, C3.1.3
S_f	Elastic section modulus of full, unreduced section for the extreme compression fiber	C3.1.1, C3.1.2, C6.1
s_{max}	Maximum permissible longitudinal spacing of welds or other connectors joining two channels to form an I–section	D1.1
s	Fastener spacing	D1.2, D4.1
s	Spacing in line of stress of welds, rivets, or bolts connecting a compression coverplate or sheet to a non–integral stiffener or other element	E3.2
s	Weld spacing	D1.1
T_n	Nominal tensile strength	C2
T_s	Design strength of connection in tension	D1.1
t	Base steel thickness of any element or section	A1.2, A3.4, A5.2.1, B1.1, B1.1b, B1.2, B2.1, B4, B4.1, B4.2, B5, B6.1, C3.1.1., C3.2, C3.4, C3.5, C4, C6.1, C6.2, D1.2, D4, E2.4, E2.5
t	Total thickness of the two welded sheets	E2.2
t	Thickness of thinnest connected part	E2.2, E3.1, E4
t_s	Equivalent thickness of a multiple–stiffened element	B5, B6.1
t_w	Effective throat of weld	E2.4, E2.5
V_F	Coefficient of variation of the fabrication factor	F1
V_M	Coefficient of variation of the material factor	F1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
V_n	Nominal shear strength	B6.2, C3.2, C3.3
V_P	Coefficient of variation of the tested–to–predicted load ratios	F1
V_Q	Coefficient of variation of the load effect	F1
V_u	Required shear strength	C3.3
W	Factored load supported by all purlin lines being restrained	D3.2.1
W	Nominal wind load	A5.1.4
w	Flat width of element exclusive of radii	A1.2, B1.1, B2.1,B2.2, B3.1, B4, B4.1, B4.2, B5, C3.1.1, C4, D1.2
w	Flat width of the beam flange which contacts the bearing plate	C3.5
w_f	Width of flange projection beyond the web or half the distance between webs for box– or U–type sections	B1.1c
w_f	Projection of flanges from inside face of web	B1.1b, D1.1
w_1	Leg on weld	E2.4
w_2	Leg on weld	E2.4
x	Distance from concentrated load to brace	D3.2
x_o	Distance from shear center to centroid along the principal x–axis	C3.1.1, C4.2, D4.1
Y	Yield point of web steel divided by yield point of stiffener steel	B6.2
$1/\alpha_{nx}$, $1/\alpha_{ny}$	Magnification factors	C5
β	Coefficient	C4.2, D4.1
β_o	Target reliability index	F1
γ	Actual shear strain in the sheathing	D4.1
$\bar{\gamma}$	Permissible shear strain of the sheathing	D4.1
γ_i	Load factor	F1
θ	Angle between web and bearing surface >45° but no more than 90°	C3.4
θ	Angle between the vertical and the plane of the web of the Z–section, degrees	D3.2.1
σ	Stress related to shear strain in sheathing	D4.1
σ_{CR}	Theoretical elastic buckling stress	D4.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
σ_{ex}	$(\pi^2 E)/(K_x L_x/r_x)^2$	C3.1.2, C4.2
	$(\pi^2 E)/(L/r_x)^2$	D4.1
σ_{exy}	$(\pi^2 E I_{xy})/(AL^2)$	D4.1
σ_{ey}	$(\pi^2 E)/(K_y L_y/r_y)^2$	C3.1.2
	$(\pi^2 E)/(L/r_y)^2$	D4.1
σ_t	Torsional buckling stress	C3.1.1, C4.2, D4.1
ρ	Reduction factor	B2.1
λ, λ_c	Slenderness factors	B2.1, C3.5
ψ	f_2/f_1	B2.3
ϕ	Resistance factor	A5.1.5, E2, E2.1, E2.2, E2.3, E2.4, E2.5, E2.6, E3.1, E3.2, E3.3, E3.4, E4, F1
ϕ_b	Resistance factor for bending strength	A5.1.5, C3, C3.1.1, C3.1.2, C3.1.3, C3.3, C3.5, C5, C6.1, C6.3, D4.2, D4.3
ϕ_c	Resistance factor for concentrically loaded compression member	A3.3.1, A5.1.5, B6.1, C4, C5, C6.2, C6.3, D4.1, D4.3
ϕ_c	Resistance factor for bearing strength	E5.1
ϕ_t	Resistance factor for tension member	C2
ϕ_v	Resistance factor for shear strength	B6.2, C3.2, C3.3
ϕ_w	Resistance factor for web crippling strength	C3.4, C3.5

LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD–FORMED STEEL STRUCTURAL MEMBERS

A. GENERAL PROVISIONS

A1 Limits of Applicability and Terms

A1.1 Scope and Limits of Applicability

This Load and Resistance Factor Design Specification is an alternate to the Specification for the Design of Cold–Formed Steel Structural Members of the American Iron and Steel Institute.

This Specification shall apply to the design of structural 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. It may also be used for structures other than buildings provided appropriate allowances are made for thermal and/or dynamic effects.

A1.2 Terms

Where the following terms appear in this Specification they shall have the meaning herein indicated:

(a) *Stiffened or Partially Stiffened Compression Elements.* A stiffened or partially stiffened compression element is a flat compression element (i.e., a plane compression flange of a flexural member or a plane web or flange of a compression member) of which both edges parallel to the direction of stress are stiffened either by a web, flange, stiffening lip, intermediate stiffener, or the like.

(b) *Unstiffened Compression Elements.* An unstiffened compression element is a flat compression element which is stiffened at only one edge parallel to the direction of stress.

(c) *Multiple–Stiffened Elements.* A multiple–stiffened element is an element that is stiffened between webs, or between a web and a stiffened edge, by means of intermediate stiffeners which are parallel to the direction of stress. A *sub–element* is the portion between adjacent stiffeners or between web and intermediate stiffener or between edge and intermediate stiffener.

(d) *Flat–Width–to–Thickness Ratio.* The flat width of an element measured along its plane, divided by its thickness.

(e) *Effective Design Width.* Where the flat width of an element is reduced for design purposes, the reduced design width is termed the effective width or effective design width.

(f) *Thickness.* The thickness, t, of any element or section shall be the base steel thickness, exclusive of coatings.

(g) *Torsional–Flexural Buckling.* Torsional–flexural buckling is a mode of buckling in which compression members can bend and twist simultaneously.

(h) *Point–Symmetric Section.* A point–symmetric section is a section symmetrical about a point (centroid) such as a Z–section having equal flanges.

(i) *Yield Point.* Yield point, F_y or F_{sy}, as used in this Specification shall mean yield point or yield strength.

(j) *Stress.* Stress as used in this Specification means force per unit area.

(k) *Confirmatory Test.* A confirmatory test is a test made, when desired, on members, connections, and assemblies designed according to the provisions of Sections A through E of this Specification or its specific references, in order to compare actual versus calculated performance.

(l) *Performance Test.* A performance test is a test made on structural members, connections, and assemblies whose performance cannot be determined by the provisions of Sections A through E of this Specification or its specific references.

(m) *Virgin Steel.* Virgin steel refers to steel as received from the steel producer or warehouse before being cold worked as a result of fabricating operations.

(n) *Virgin Steel Properties.* Virgin steel properties refer to mechanical properties of virgin steel such as yield point, tensile strength, and elongation.

(o) *Specified Minimum Yield Point.* The specified minimum yield point is the lower limit of yield point which must be equalled or exceeded in a specification test to qualify a lot of steel for use in a cold–formed steel structural member designed at that yield point.

(p) *Cold–Formed Steel Structural Members.* Cold–formed steel structural members are shapes which are manufactured by press–braking blanks sheared from sheets, cut lengths of coils or plates, or by roll forming cold– or hot–rolled coils or sheets; both forming operations being performed at ambient room temperature, that is, without manifest addition of heat such as would be required for hot forming.

(q) *LRFD (Load and Resistance Factor Design).* A method of proportioning structural components (members, connectors, connecting elements and assemblages) such that no applicable limit state is exceeded when the structure is subjected to all appropriate load combinations.

(r) *Design Strength.* Factored resistance or strength (force, moment, as appropriate), ϕR_n, provided by the structural component.

(s) *Required Strength.* Load effect (force, moment, as appropriate) acting on the structural component determined by structural analysis from the factored loads (using most appropriate critical load combinations).

A1.3 Units of Symbols and Terms

The Specification is written so that any compatible system of units may be used except where explicitly stated otherwise in the text of these provisions.

A2 Non–Conforming Shapes and Construction

The provisions of the Specification are not intended to prevent the use of alternate shapes or constructions not specifically prescribed herein. Such alternates shall meet the provisions of Section F of the Specification and be approved by the appropriate building code authority.

A3 Material

A3.1 Applicable Steels

This Specification requires the use of steel of structural quality as defined in general by the provisions of the following specifications of the American Society for Testing and Materials:

ASTM A36/A36M, Structural Steel

ASTM A242/A242M, High-Strength Low-Alloy Structural Steel

ASTM A441M, High-Strength Low-Alloy Structural Manganese Vanadium Steel

ASTM A446/A446M (Grades A, B, C, D, & F) Steel, Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process, Structural (Physical) Quality

ASTM A500, Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

ASTM A529/A529M, Structural Steel with 42 ksi Minimum Yield Point (1/2 in. Maximum Thickness)

ASTM A570/A570M Steel, Sheet and Strip, Carbon, Hot-Rolled, Structural Quality

ASTM A572/A572M, High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality

ASTM A588/A588M, High-Strength Low-Alloy Structural Steel with 50 ksi Minimum Yield Point to 4 in. Thick

ASTM A606 Steel, Sheet and Strip, High Strength, Low Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance

ASTM A607 Steel Sheet and Strip, High Strength, Low Alloy, Columbium or Vanadium, or both, Hot-Rolled and Cold-Rolled

ASTM A611 (Grades A, B, C, & D) Steel, Sheet, Carbon, Cold-Rolled, Structural Quality

ASTM A715 (Grades 50 and 60) Sheet Steel and Strip, High-Strength, Low-Alloy, Hot-Rolled, With Improved Formability

ASTM A792 (Grades 33, 37, 40 & 50) Steel Sheet, Aluminum-Zinc Alloy-Coated by the Hot-Dip Process, General Requirements

A3.2 Other Steels

The listing in Section A3.1 does not exclude the use of steel up to and including one inch in thickness ordered or produced to other than the listed specifications provided such steel conforms to the chemical and mechanical requirements of one of the listed specifications or other published specification which establishes its properties and suitability, and provided it is subjected by either the producer or the purchaser to analyses, tests and other controls to the extent and in the manner prescribed by one of the listed specifications and Section A3.3.

A3.3 Ductility

Steels not listed in Section A3.1 and used for structural members and connections shall comply with one of the following ductility requirements:

A3.3.1 The ratio of tensile strength to yield point shall not be less than 1.08, and the total elongation shall not be less than 10 percent for a two-inch gage length or 7 percent for an eight-inch gage length standard specimen tested in accordance with ASTM A370. If these requirements cannot be met, the following criteria shall be satisfied: (1) local elongation in a 1/2 inch gage length across the fracture shall not be less than 20%, (2) uniform elongation outside the fracture shall not be less than 3%*. When material ductility is determined on the basis of the local and uniform elongation criteria, the use of such material is restricted to the design of purlins and girts** in

* Further information on the test procedures should be obtained from the Commentary.

** Horizontal structural members which support roof deck or panel covering and applied loads principally by bending.

accordance with Sections C3.1.1(a), C3.1.2, and C3.1.3. For purlins and girts subject to combined axial load and bending moment (Section C5), $P_u/\phi_c P_n$ shall not exceed 0.15.

A3.3.2 Steels conforming to ASTM A446 Grade E and A611 Grade E and other steels which do not meet the provisions of Section A3.3.1 may be used for particular configurations provided (1) the yield strength, F_y, used for design in Chapters B, C and D is taken as 75 percent of the specified minimum yield point or 60 ksi, whichever is less and (2) the tensile strength, F_u, used for design in Chapter E is taken as 75 percent of the specified minimum tensile stress or 62 ksi, whichever is less. Alternatively, the suitability of such steels for the configuration shall be demonstrated by load tests in accordance with Section F1. Design strengths based on these tests shall not exceed the strengths calculated according to Chapters B through E, using the specified minimum yield point, F_{sy}, for F_y and the specified minimum tensile strength, F_u.

Design strengths based on existing use shall not exceed the strengths calculated according to Chapters B through E, using the specified minimum yield point, F_{sy}, for F_y and the specified minimum tensile strength, F_u.

A3.4 Delivered Minimum Thickness

The uncoated minimum steel thickness of the cold–formed product as delivered to the job site shall not at any location be less than 95 percent of the thickness, t, used in its design; however, lesser thicknesses shall be permitted at bends, such as corners, due to cold–forming effects.

A4 Loads

A4.1 Dead Load

The dead load to be assumed in design shall consist of the weight of steelwork and all material permanently fastened thereto or supported thereby.

A4.2 Live or Snow Load

The live or snow load shall be that stipulated by the applicable code or specification under which the structure is being designed or that dictated by the conditions involved.

A4.3 Impact Load

For structures carrying live loads which induce impact, the assumed live load shall be increased sufficiently to provide for impact.

A4.4 Wind or Earthquake Loads

Wind or earthquake load shall be that stipulated by the applicable code or specification under which the structure is being designed or that dictated by the conditions involved.

A4.5 Ponding

Unless a roof surface is provided with sufficient slope toward points of free drainage or adequate individual drains to prevent the accumulation of rainwater, the roof system shall be investigated by rational analysis to assure stability under ponding conditions.

A5 Structural Analysis and Design

A5.1 Design Basis

This Specification is based on the Load and Resistance Factor Design concept. Load and Resistance Factor Design is a method of proportioning cold–formed steel structural components (i.e., members, connectors and connections) such that any applicable limit state is not exceeded when the structure is subjected to any appropriate load combination.

Two types of limit states are to be considered: 1) the limit state of the strength required to resist the extreme loads during the intended life of the structure, and 2) the limit state of the ability of the structure to perform its intended function during its life. These limit states will be called the Limit State of Strength and the Limit State of Serviceability, respectively, in these criteria.

A5.1.1 Limit State – Strength

The design meets this Specification when the required strengths, as determined from the assigned nominal loads which are multiplied by appropriate load factors, are smaller than or equal to the design strength of each structural component.

The design strength is equal to ϕR_n, where ϕ is a resistance factor and R_n is the nominal strength determined according to the formulas given in Chapter C for members, in Chapter D for structural assemblies and in Chapter E for connections. Values of resistance factors ϕ are given in Section A5.1.5 for the appropriate limit states governing member and connection strength.

A5.1.2 Limit State – Serviceability

Serviceability is satisfactory if a nominal structural response (e.g. live load deflection) due to the applicable nominal loads is less than or equal to the appropriate acceptable or allowable value of this response.

A5.1.3 Nominal Loads

The nominal loads shall be the minimum design loads stipulated by the applicable code under which the structure is designed or dictated by the conditions involved. In the absence of a code, the loads and load combinations shall be those stipulated in the American Society of Civil Engineers Standard, ANSI/ASCE 7–88, Minimum Design Loads for Buildings and Other Structures. For design purposes, the loads stipulated by the applicable code shall be taken as nominal loads.

A5.1.4 Load Factors and Load Combinations*

The structure and its components must be designed for the appropriate most critical load combination. The following load combinations of the factored nominal loads shall be used in the computation of the required strengths:

1. $1.4\,D + L$
2. $1.2\,D + 1.6\,L + 0.5(L_r \text{ or } S \text{ or } R_r)$
3. $1.2\,D + (1.4\,L_r \text{ or } 1.6\,S \text{ or } 1.6\,R_r) + (0.5\,L \text{ or } 0.8\,W)$

* For roof and floor construction, recommended load combinations for dead load, weight of wet concrete, and construction load including equipment, workmen and formwork are given in Section A5.1 of the Commentary.

4. 1.2 D + 1.3 W + 0.5 L + 0.5(L_r or S or R_r)
5. 1.2 D + 1.5 E + (0.5 L or 0.2 S)
6. 0.9 D – (1.3 W or 1.5 E)

where D = nominal dead load
E = nominal earthquake load
L = nominal live load
L_r = nominal roof live load
R_r = nominal roof rain load
S = nominal snow load
W = nominal wind load (Exception: For wind load on individual purlins, girts, wall panels and roof decks, multiply the load factor for W by 0.9)

Exception: The load factor for L in combinations (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.

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 change, shrinkage, moisture changes, creep in component materials, movement due to differential settlement, or combinations thereof.

A5.1.5 Resistance Factors

The resistance factors to be used for determining the design strengths, ϕR_n, of structural members and connections shall be taken as follows:

Type of Strength	**Resistance Factor, ϕ**
(a) Stiffeners	
Transverse stiffeners	0.85
Shear stiffeners*	0.90
(b) Tension members	0.95
(c) Flexural members	
Bending strength	
For sections with stiffened or partially stiffened compression flanges	0.95
For sections with unstiffened compression flanges	0.90
Laterally unbraced beams	0.90
Beams having one flange through–fastened to deck or sheathing (C– or Z–sections)	0.90
Web design	
Shear strength*	0.90
Web Crippling	
For single unreinforced webs	0.75
For I–sections	0.80

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

Type of Strength	Resistance Factor, ϕ
(d) Concentrically loaded compression members	0.85
(e) Combined axial load and bending	
ϕ_c for compression	0.85
ϕ_b for bending	
Using Section C3.1.1	0.90 – 0.95
Using Section C3.1.2	0.90
(f) . Cylindrical tubular members	
Bending strength	0.95
Axial compression	0.85
(g) Wall studs and wall stud assemblies	
Wall studs in compression	0.85
Wall studs in bending	
For sections with stiffened or partially stiffened compression flanges	0.95
For sections with unstiffened compression flanges	0.90
(h) Welded connections	
Groove welds	
Tension or compression	0.90
Shear (welds)	0.80
Shear (base metal)	0.90
Arc spot welds	
Welds	0.60
Connected part	0.50 – 0.65
Minimum edge distance	0.60 – 0.70
Arc seam welds	
Welds	0.60
Connected part	0.60
Fillet welds	
Longitudinal loading (connected part)	0.55 – 0.60
Transverse loading (connected part)	0.60
Welds	0.60
Flare groove welds	
Transverse loading (connected part)	0.55
Longitudinal loading (connected part)	0.55
Welds	0.60
Resistance Welds	0.65
(i) Bolted connections	
Minimum spacing and edge distance	0.60 – 0.70
Tension strength on net section	
With washers	
Double shear connection	0.65
Single shear connection	0.55
Without washers	0.65
Bearing strength	
See Tables E3.3–1 and E3.3–2	0.55 – 0.70
Shear strength of bolts	0.65
Tensile strength of bolts	0.75
(j) Shear rupture	0.75
(k) Connections to other materials (Bearing)	0.60

A5.2 Yield Point and Strength Increase from Cold Work of Forming

A5.2.1 Yield Point

The yield point used in design, F_y, shall not exceed the specified minimum yield point of steels as listed in Section A3.1 or A3.2, as established in accordance with Chapter F, or as increased for cold work of forming in Section A5.2.2, or as reduced for low ductility steels in Section A3.3.2.

A5.2.2 Strength Increase from Cold Work of Forming

Strength increase from cold work of forming shall be permitted by substituting F_{ya} for F_y, where F_{ya} is the average yield point of the full section. Such increase shall be limited to Sections C3.1 (excluding Section C3.1.1(b)), C4, C5, C6 and D4. The limitations and methods for determining F_{ya} are as follows:

(a) For axially loaded compression members and flexural members whose proportions are such that the quantity ρ for load capacity is unity as determined according to Section B2 for each of the component elements of the section, the design yield stress, F_{ya}, of the steel shall be determined on the basis of one of the following methods:

(1) full section tensile tests [see paragraph (a) of Section F3.1]

(2) stub column tests [see paragraph (b) of Section F3.1]

(3) computed as follows:

$$F_{ya} = CF_{yc} + (1 - C) F_{yf} \quad (Eq.\ A5.2.2\text{–}1)$$

where

F_{ya} = Average yield point of the steel in the full section of compression members or full flange sections of flexural members

C = For compression members, ratio of the total corner cross–sectional area to the total cross–sectional area of the full section; for flexural members, ratio of the total corner cross–sectional area of the controlling flange to the full cross–sectional area of the controlling flange

F_{yf} = Weighted average tensile yield point of the flat portions established in accordance with Section F3.2 or virgin steel yield point if tests are not made

F_{yc} = $B_cF_{yv}/(R/t)^m$, tensile yield point of corners. This formula is applicable only when $F_{uv}/F_{yv} \geq 1.2$, $R/t \leq 7$, and minimum included angle $\leq 120°$ (*Eq.* A5.2.2–2)

B_c = $3.69\,(F_{uv}/F_{yv}) - 0.819\,(F_{uv}/F_{yv})^2 - 1.79$ (*Eq.* A5.2.2–3)

m = $0.192\,(F_{uv}/F_{yv}) - 0.068$ (*Eq.* A5.2.2–4)

R = Inside bend radius.

F_{yv} = Tensile yield point of virgin steel* specified by Section A3 or established in accordance with Section F3.3

F_{uv} = Ultimate tensile strength of virgin steel* specified by Section A3 or established in accordance with Section F3.3

(b) For axially loaded tension members the yield point of the steel shall be determined by either method (1) or method (3) prescribed in paragraph (a) of this Section.

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* Virgin steel refers to the condition (i.e., coiled or straight) of the steel prior to the cold–forming operation.

(c) The effect of any welding on mechanical properties of a member shall be determined on the basis of tests of full section specimens containing within the gage length, such welding as the manufacturer intends to use. Any necessary allowance for such effect shall be made in the structural use of the member.

A5.3 Durability

A structure shall be designed to perform its required functions during its expected life for durability considerations.

A6 Reference Documents

The following documents are referenced in this Specification:

1. American Society of Civil Engineers, ANSI/ASCE 7–88, "Minimum Design Loads in Buildings and Other Structures," American Society of Civil Engineers (ASCE), 345 East 47th Street, New York, N.Y. 10017
2. American Institute of Steel Construction, "Load and Resistance Factor Design Specification for Structural Steel Buildings", American Institute of Steel Construction (AISC), One East Wacker Drive, Suite 3100, Chicago, Illinois 60601–2001, September 1, 1986
3. American Welding Society, AWS D1.3–89, "Structural Welding Code – Sheet Steel," American Welding Society (AWS), 550 N.W. LeJeune Road, Miami, Florida 33135
4. American Welding Society, AWS C1.1–66, "Recommended Practices for Resistance Welding," American Welding Society (AWS), 550 N.W. LeJeune Road, Miami, Florida 33135
5. American Welding Society, AWS C1.3–70, "Recommended Practices for Resistance Welding Coated Low Carbon Steels," American Welding Society (AWS), 550 N.W. LeJeune Road, Miami, Florida 33135
6. American Society for Testing and Materials (ASTM), 1916 Race Street, Philadelphia, Pennsylvania 19013:

   ASTM A36/A36M–84a, Structural Steel

   ASTM A194–88, Carbon and Alloy Steel Nuts for Bolts for High–Pressure and High–Temperature Service

   ASTM A242/A242M–85, High–Strength Low–Alloy Structural Steel

   ASTM A307–84 (Type A), Carbon Steel Externally and Internally Threaded Standard Fasteners

   ASTM A325–84, High Strength Bolts for Structural Steel Joints

   ASTM A354–84 (Grade BD), Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners (for diameter of bolt smaller than $^1/_2$ inch)

   ASTM A370–77 Mechanical Testing of Steel Products

   ASTM A441M–85, High–Strength Low–Alloy Structural Manganese Vanadium Steel

   ASTM A446/A446M–85 (Grades A, B, C, D, & F) Steel, Sheet, Zinc–Coated (Galvanized) by the Hot–Dip Process, Structural (Physical) Quality

   ASTM A449–84a, Quenched and Tempered Steel Bolts and Studs (for diameter of bolt smaller than $^1/_2$ inch)

ASTM A490–84, Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints.

ASTM A500–84, Cold–Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

ASTM A529/A529M–85, Structural Steel with 42 ksi Minimum Yield Point (1/2 in. Maximum Thickness)

ASTM A563–88a, Carbon and Alloy Steel Nuts

ASTM A570/A570M–85 Steel, Sheet and Strip, Carbon, Hot–Rolled, Structural Quality

ASTM A572/A572M–85, High–Strength Low–Alloy Columbium–Vanadium Steels of Structural Quality

ASTM A588/A588M–85, High–Strength Low–Alloy Structural Steel with 50 ksi Minimum Yield Point to 4 in. Thick

ASTM A606–85 Steel, Sheet and Strip, High Strength, Low Alloy, Hot–Rolled and Cold–Rolled, with Improved Atmospheric Corrosion Resistance

ASTM A607–85 Steel Sheet and Strip, High Strength, Low Alloy, Columbium or Vanadium, or both, Hot–Rolled and Cold–Rolled

ASTM A611–85 (Grades A, B, C, & D) Steel, Sheet, Carbon, Cold–Rolled, Structural Quality

ASTM A715–85 (Grades 50 & 60) Sheet Steel and Strip, High–Strength, Low–Alloy, Hot–Rolled, With Improved Formability

ASTM A792–85a (Grades 33, 37, 40 & 50) Steel Sheet, Aluminum–Zinc Alloy–Coated by the Hot–Dip Process, General Requirements

ASTM F436–86, Hardened Steel Washers

ASTM F844–83(1988), Washers, Steel, Plain (Flat), Unhardened for General Use

ASTM F959–85, Compressible Washer–Type Direct Tension Indicators for Use with Structural Fasteners

B. ELEMENTS

B1 Dimensional Limits and Considerations

B1.1 Flange Flat-Width-to-Thickness Considerations

(a) *Maximum Flat-Width-to-Thickness Ratios*

Maximum allowable overall flat-width-to-thickness ratios, w/t, disregarding intermediate stiffeners and taking as t the actual thickness of the element, shall be as follows:

(1) Stiffened compression element having *one* longitudinal edge connected to a web or flange element, the other stiffened by:

Simple lip	60
Any other kind of stiffener having $I_s > I_a$ and $D/w < 0.8$ according to Section B4.2	90

(2) Stiffened compression element with *both* longitudinal edges connected to other stiffened elements — 500

(3) Unstiffened compression element and elements with an edge stiffener having $I_s < I_a$ and $D/w \leq 0.8$ according to Section B4.2 — 60

Note: Unstiffened compression elements that have w/t ratios exceeding approximately 30 and stiffened compression elements that have w/t ratios exceeding approximately 250 are likely to develop noticeable deformation at the full design strength, without affecting the ability of the member to develop required strength.

Stiffened elements having w/t ratios larger than 500 can be used with adequate design strength to sustain the required loads; however, substantial deformations of such elements usually will invalidate the design formulas of this Specification.

(b) *Flange Curling*

Where the flange of a flexural member is unusually wide and it is desired to limit the maximum amount of curling or movement of the flange toward the neutral axis, the following formula applies to compression and tension flanges, either stiffened or unstiffened:

$$w_f = \sqrt{0.061 t d E/f_{av}}\sqrt[4]{(100 c_f/d)} \qquad (Eq.\ B1.1\text{-}1)$$

where

w_f = Width of flange projecting beyond the web; or half of the distance between webs for box- or U-type beams

t = Flange thickness

d = Depth of beam

c_f = Amount of curling*

f_{av} = Average stress in the full, unreduced flange width. (Where members are designed by the effective design width procedure, the average stress equals the maximum stress multiplied by the ratio of the effective design width to the actual width.)

(c) *Shear Lag Effects – Short Spans Supporting Concentrated Loads*
Where the span of the beam is less than $30w_f$ (w_f as defined below) and it carries one concentrated load, or several loads spaced farther apart than $2w_f$, the effective design width of any flange, whether in tension or compression, shall be limited to the following:

TABLE B1.1(c)
SHORT, WIDE FLANGES
MAXIMUM ALLOWABLE RATIO OF EFFECTIVE DESIGN WIDTH TO ACTUAL WIDTH

L/w_f	Ratio	L/w_f	Ratio
30	1.00	14	0.82
25	0.96	12	0.78
20	0.91	10	0.73
18	0.89	8	0.67
16	0.86	6	0.55

where

L = Full span for simple beams; or the distance between inflection points for continuous beams; or twice the length of cantilever beams.

w_f = Width of flange projection beyond the web for I–beam and similar sections or half the distance between webs of box or U–type sections.

For flanges of I–beams and similar sections stiffened by lips at the outer edges, w_f shall be taken as the sum of the flange projection beyond the web plus the depth of the lip.

B1.2 Maximum Web Depth–to–Thickness Ratio

The ratio, h/t, of the webs of flexural members shall not exceed the following limitations:

(a) For unreinforced webs: $(h/t)_{max} = 200$

(b) For webs which are provided with transverse stiffeners satisfying the requirements of Section B6.1:

(1) When using bearing stiffeners only, $(h/t)_{max} = 260$

(2) When using bearing stiffeners and intermediate stiffeners, $(h/t)_{max} = 300$

In the above,

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

* The amount of curling that can be tolerated will vary with different kinds of sections and must be established by the designer. Amount of curling in the order of 5 percent of the depth of the section is usually not considered excessive.

t = Web thickness

Where a web consists of two or more sheets, the h/t ratio shall be computed for the individual sheets.

B2 Effective Widths of Stiffened Elements

B2.1 Uniformly Compressed Stiffened Elements

(a) *Load Capacity Determination*

The effective widths, b, of uniformly compressed elements shall be determined from the following formulas:

$b = w$ when $\lambda \leq 0.673$ (*Eq.* B2.1–1)

$b = \rho w$ when $\lambda > 0.673$ (*Eq.* B2.1–2)

where

w = Flat width as shown in Figure B2.1–1

$\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq.* B2.1–3)

λ is a slenderness factor determined as follows:

$$\lambda = \frac{1.052}{\sqrt{k}}\left(\frac{w}{t}\right)\sqrt{\frac{f}{E}} \qquad (Eq.\ B2.1\text{–}4)$$

where

t = Thickness of the uniformly compressed stiffened elements, and

f for load capacity determination is as follows:

For flexural members:

(1) If Procedure I of Section C3.1.1 is used, $f = F_y$ if the initial yielding is in compression in the element considered.
If the initial yielding is in tension, the compressive stress, f, in the element considered shall be determined on the basis of the effective section at M_y (moment causing initial yield).

(2) If Procedure II of Section C3.1.1 is used then f is the stress in the element considered at M_n determined on the basis of the effective section.

(3) If Section C3.1.2 is used, then f is the stress $\frac{M_c}{S_f}$ as described in that Section in determining S_c.

For compression members f is taken equal to F_n as determined in Section C4 or D4.1 as applicable.

E = Modulus of elasticity

k = Plate buckling coefficient

= 4 for stiffened elements supported by a web on each longitudinal edge. Values for different types of elements are given in the applicable sections.

(b) *Deflection Determination*

The effective widths, b_d, used in computing deflection shall be determined from the following formulas:

$b_d = w$ when $\lambda \leq 0.673$ (*Eq.* B2.1–5)

$b_d = \rho w$ when $\lambda > 0.673$ (*Eq.* B2.1–6)

where

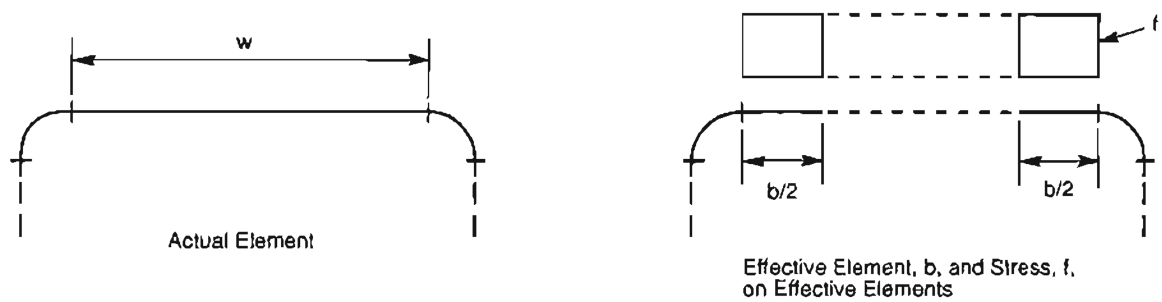

Figure B2.1–1 Stiffened Elements

w = Flat width

ρ = Reduction factor determined by either of the following two procedures:

(1) Procedure I.
A low estimate of the effective width may be obtained from Eqs. B2.1–3 and B2.1–4 where f_d is substituted for f where f_d is the computed compressive stress in the element being considered.

(2) Procedure II.
For stiffened elements supported by a web on each longitudinal edge an improved estimate of the effective width can be obtained by calculating ρ as follows:

$\rho = 1$ when $\lambda \leq 0.673$ (*Eq.* B2.1–7)

$\rho = (1.358 - 0.461/\lambda)/\lambda$ when $0.673 < \lambda < \lambda_c$ (*Eq.* B2.1–8)

$\rho = (0.41 + 0.59\sqrt{F_y/f_d} - 0.22/\lambda)/\lambda$ when $\lambda \geq \lambda_c$ (*Eq.* B2.1–9)

ρ shall not exceed 1.0 for all cases.

where

$\lambda_c = 0.256 + 0.328\,(w/t)\sqrt{F_y/E}$ (*Eq.* B2.1–10)

and λ is as defined by Eq. B2.1–4 except that f_d is substituted for f.

B2.2 Uniformly Compressed Stiffened Elements with Circular Holes

(a) *Load Capacity Determination*
The effective width, b, of stiffened elements with uniform compression having circular holes shall be determined as follows:

for $0.50 \geq \frac{d_h}{w} \geq 0$, and $\frac{w}{t} \leq 70$

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

$b = w - d_h$ when $\lambda \leq 0.673$ (*Eq.* B2.2–1)

$$b = \frac{w\left[1 - \frac{(0.22)}{\lambda} - \frac{(0.8d_h)}{w}\right]}{\lambda} \quad \text{when } \lambda > 0.673$$ (*Eq.* B2.2–2)

b shall not exceed $w - d_h$

where

w = Flat width

d_h = Diameter of holes

λ is as defined in Section B2.1.

(b) *Deflection Determination*

The effective width, b_d, used in deflection calculations shall be equal to b determined in accordance with Procedure I of Section B2.2a except that f_d is substituted for f, where f_d is the computed compressive stress in the element being considered.

B2.3 Effective Widths of Webs and Stiffened Elements with Stress Gradient

(a) *Load Capacity Determination*

The effective widths, b_1 and b_2, as shown in Figure B2.3–1 shall be determined from the following formulas:

$$b_1 = b_e/(3 - \psi) \qquad (Eq.\ B2.3\text{–}1)$$

For $\psi \leq -0.236$

$$b_2 = b_e/2 \qquad (Eq.\ B2.3\text{–}2)$$

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

For $\psi > -0.236$

$$b_2 = b_e - b_1 \qquad (Eq.\ B2.3\text{–}3)$$

where

b_e = Effective width b determined in accordance with Section B2.1 with f_1 substituted for f and with k determined as follows:

$$k = 4 + 2(1 - \psi)^3 + 2(1 - \psi) \qquad (Eq.\ B2.3\text{–}4)$$

$\psi = f_2/f_1$

f_1, f_2 = Stresses shown in Figure B2.3–1 calculated on the basis of effective section.

f_1 is compression (+) and f_2 can be either tension (–) or compression. In case f_1 and f_2 are both compression, $f_1 \geq f_2$

(b) *Deflection Determination*

The effective widths in computing deflections at a given load shall be determined in accordance with Section B2.3a except that f_{d1} and f_{d2} are substituted for f_1 and f_2, where f_{d1}, f_{d2} = Computed stresses f_1 and f_2 as shown in Figure B2.3–1. Calculations are based on the effective section at the load for which deflections are determined.

B3 Effective Widths of Unstiffened Elements

B3.1 Uniformly Compressed Unstiffened Elements

(a) *Load Capacity Determination*

Effective widths, b, of unstiffened compression elements with uniform compression shall be determined in accordance with Section B2.1a with the exception that k shall be taken as 0.43 and w as defined in Figure B3.1–1.

(b) *Deflection Determination*

The effective widths used in computing deflections shall be determined in accordance with Procedure I of Section B2.1b except that f_d is substituted for f and k = 0.43.

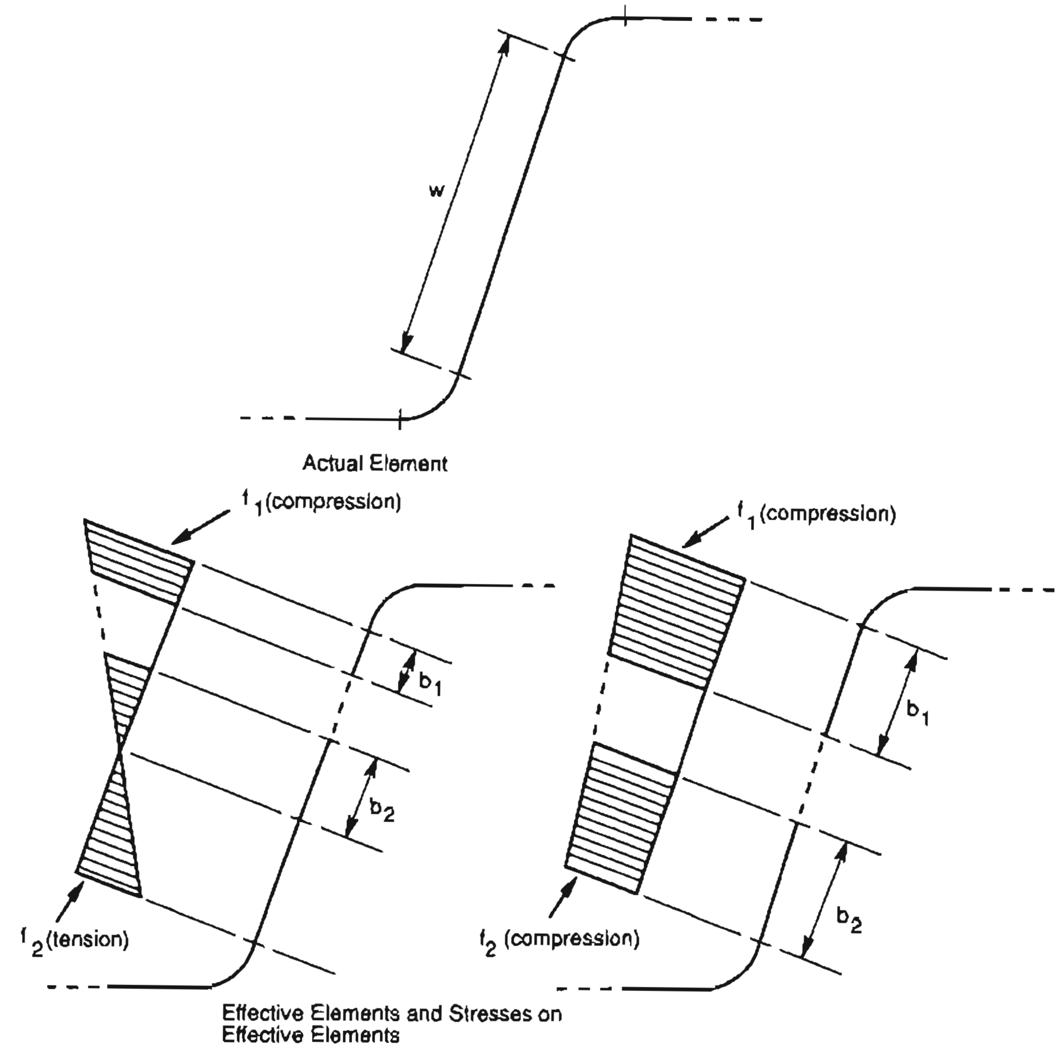

Figure B2.3–1 Stiffened Elements with Stress Gradient and Webs

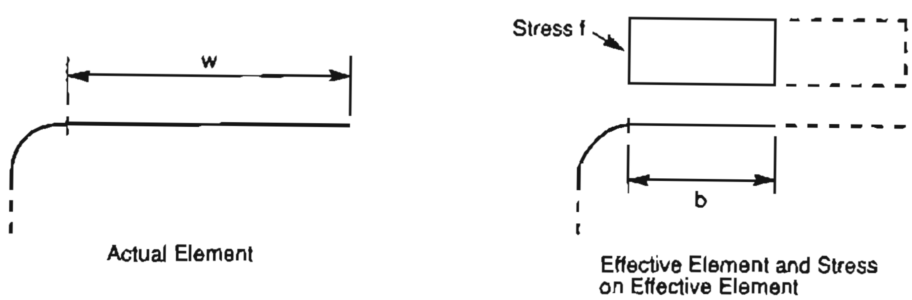

Figure B3.1–1 Unstiffened Element with Uniform Compression

B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

(a) *Load Capacity Determination*
Effective widths, b, of unstiffened compression elements and edge stiffeners with stress gradient shall be determined in accordance with Section B2.1a with $f = f_3$ as in Figure B4–2 in the element and $k = 0.43$.

(b) *Deflection Determination*
Effective widths, b, of unstiffened compression elements and edge stiffeners with

stress gradient shall be determined in accordance with Procedure I of Section B2.1b except that f_{d3} is substituted for f and $k = 0.43$.

B4 Effective Widths of Elements with an Edge Stiffener or One Intermediate Stiffener

The following notation is used in this section.

S = $1.28\sqrt{E/f}$ (*Eq*. B4–1)
k = Buckling coefficient
b_o = Dimension defined in Figure B4–1
d, w, D = Dimensions defined in Figure B4–2
d_s = Reduced effective width of the stiffener as specified in this section. d_s, calculated according to Section B4.2, is to be used in computing the overall effective section properties (see Figure B4–2)
d'_s = Effective width of the stiffener calculated according to Section B3.1 (see Figure B4–2)
C_1, C_2 = Coefficients defined in Figure B4–2
A_s = Reduced area of the stiffener as specified in this section. A_s is to be used in computing the overall effective section properties. The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener, and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener.
I_a = Adequate moment of inertia of stiffener, so that each component element will behave as a stiffened element.
I_s, A'_s = Moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened and the effective area of the stiffener, respectively. For edge stiffeners the round corner between the stiffener and the element to be stiffened shall not be considered as a part of the stiffener.

For the stiffener shown in Figure B4–2:

I_s = $(d^3 t \sin^2\theta)/12$ (*Eq*. B4–2)
A'_s = $d'_s t$ (*Eq*. B4–3)

B4.1 Uniformly Compressed Elements with an Intermediate Stiffener

(a) *Load Capacity Determination*

Case I: $b_o/t \le S$ (*Eq*. B4.1–1)

$I_a = 0$ (no intermediate stiffener needed) (*Eq*. B4.1–2)

$b = w$ (*Eq*. B4.1–3)

$A_s = A'_s$ (*Eq*. B4.1–4)

Case II: $S < b_o/t < 3S$ (*Eq*. B4.1–5)

$I_a/t^4 = [50(b_o/t)/S] - 50$ (*Eq*. B4.1–6)

b and A_s are calculated according to Section B2.1a where

$k = 3(I_s/I_a)^{1/2} + 1 \le 4$ (*Eq*. B4.1–7)

$A_s = A'_s(I_s/I_a) \le A'_s$ (*Eq*. B4.1–8)

Case III: $b_o/t \ge 3S$

$I_a/t^4 = [128(b_o/t)/S] - 285$ (*Eq*. B4.1–9)

b and A_s are calculated according to Section B2.1a where

$k = 3(I_s/I_a)^{1/3} + 1 \le 4$ (*Eq*. B4.1–10)

$A_s = A'_s\,(I_s/I_a) \le A'_s$ (*Eq*. B4.1–11)

(b) *Deflection Determination*

Effective widths shall be determined as in Section B4.1a except that f_d is substituted for f.

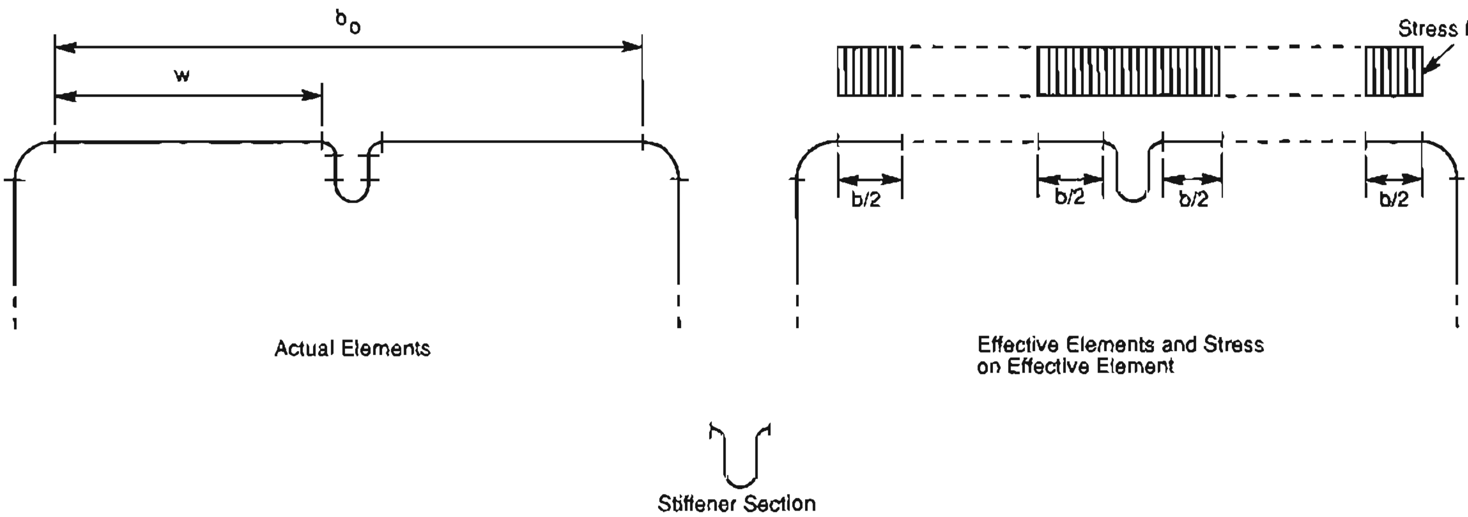

Figure B4-1 Elements with Intermediate Stiffener

B4.2 Uniformly Compressed Elements with an Edge Stiffener

(a) *Load Capacity Determination*

Case I: $w/t \leq S/3$ (*Eq.* B4.2-1)

$I_a = 0$ (no edge stiffener needed) (*Eq.* B4.2-2)

$b = w$ (*Eq.* B4.2-3)

$d_s = d'_s$ for simple lip stiffener (*Eq.* B4.2-4)

$A_s = A'_s$ for other stiffener shapes (*Eq.* B4.2-5)

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

$I_a/t^4 = 399\{[(w/t)/S] - 0.33\}^3$ (*Eq.* B4.2-6)

$n = 1/2$

$C_2 = I_s/I_a \leq 1$ (*Eq.* B4.2-7)

$C_1 = 2 - C_2$ (*Eq.* B4.2-8)

b shall be calculated according to Section B2.1 where

$k = [4.82 - 5(D/w)](I_s/I_a)^n + 0.43 \leq 5.25 - 5(D/w)$ (*Eq.* B4.2-9)

for $0.8 \geq D/w > 0.25$

$k = 3.57(I_s/I_a)^n + 0.43 \leq 4.0$ (*Eq.* B4.2-10)

for $(D/w) \leq 0.25$

$d_s = d'_s\,(I_s/I_a) \leq d'_s$ (*Eq.* B4.2-11)

for simple lip stiffener

$A_s = A'_s\,(I_s/I_a) \leq A'_s$ (*Eq.* B4.2-12)

for other stiffener shape

Case III: $w/t \geq S$

$I_a/t^4 = [115\,(w/t)/S] + 5$ (*Eq.* B4.2-13)

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

(b) *Deflection Determination*

Effective widths shall be determined as in Section B4.2a except that f_d is substituted for f.

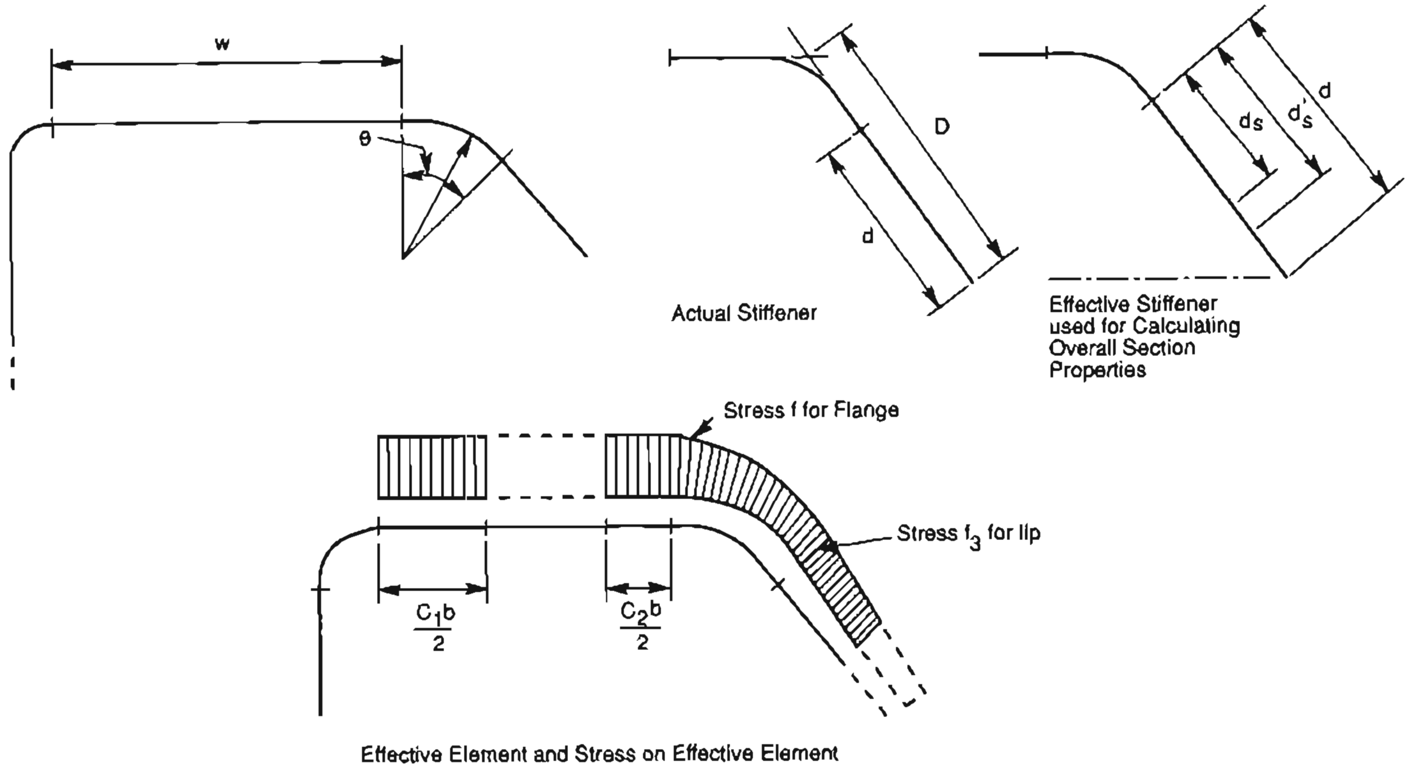

Figure B4–2 Elements with Edge Stiffener

B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener

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

$$I_{min} = \left[3.66\sqrt{\left((w/t)^2 - (0.136E)/F_y\right)}\right]t^4 \qquad (Eq.\ B5\text{–}1)$$

but not less than $18.4\ t^4$

where

w/t = Width–thickness ratio of the larger stiffened sub–element

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

(a) If the spacing of intermediate stiffeners between two webs is such that for the sub–element between stiffeners $b < w$ as determined in Section B2.1, only two intermediate stiffeners (those nearest each web) shall be considered effective.

(b) If the spacing of intermediate stiffeners between a web and an edge stiffener is such that for the sub–element between stiffeners $b < w$ as determined in Section B2.1, only one intermediate stiffener, that nearest the web, shall be considered effective.

(c) If intermediate stiffeners are spaced so closely that for the elements between stiffeners $b = w$ as determined in Section B2.1, all the stiffeners may be considered effective. In computing the flat–width to thickness ratio of the entire multiple–stiffened element, such element shall be considered as replaced by an "equivalent element" without intermediate stiffeners whose width, b_o, is the full width between webs or from web to edge stiffener, and whose equivalent thickness, t_s, is determined as follows:

$$t_s = \sqrt[3]{12 I_{sf} / b_o} \qquad (Eq.\ B5\text{-}2)$$

where

I_{sf} = Moment of inertia of the full area of the multiple-stiffened element, including the intermediate stiffeners, about its own centroidal axis. The moment of inertia of the entire section shall be calculated assuming the "equivalent element" to be located at the centroidal axis of the multiple stiffened element, including the intermediate stiffener. The actual extreme fiber distance shall be used in computing the section modulus.

(d) If $w/t > 60$, the effective width, b_e, of the sub-element or element shall be determined from the following formula:

$$\frac{b_e}{t} = \frac{b}{t} - 0.10\left[\frac{w}{t} - 60\right] \qquad (Eq.\ B5\text{-}3)$$

where

w/t = flat-width ratio of sub-element or element

b = effective design width determined in accordance with the provisions of Section B2.1

b_e = effective design width of sub-element or element to be used in design computations

For computing the effective structural properties of a member having compression sub-elements or element subject to the above reduction in effective width, the area of stiffeners (edge stiffener or intermediate stiffeners) shall be considered reduced to an effective area as follows:

For $60 < w/t < 90$:

$$A_{ef} = \alpha A_{st} \qquad (Eq.\ B5\text{-}4)$$

where

$$\alpha = (3 - 2b_e / w) - \frac{1}{30}\left[1 - \frac{b_e}{w}\right]\left[\frac{w}{t}\right] \qquad (Eq.\ B5\text{-}5)$$

For $w/t \geq 90$:

$$A_{ef} = (b_e/w)\ A_{st} \qquad (Eq.\ B5\text{-}6)$$

In the above expressions, A_{ef} and A_{st} refer only to the area of the stiffener section, exclusive of any portion of adjacent elements.

The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener, and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener.

B6 Stiffeners

B6.1 Transverse Stiffeners

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. Required strengths for the concentrated loads or reactions shall not exceed the design strength, $\phi_c P_n$, where $\phi_c = 0.85$ and P_n is the smaller value given by (a) and (b) as follows:

(a) $P_n = F_{wy}A_c$ (*Eq.* B6.1-1)

(b) P_n = Nominal axial strength evaluated according to Section C4(a) with A_e replaced by A_b

where

$A_c = 18t^2 + A_s$, for transverse stiffeners at interior support and under concentrated load (*Eq.* B6.1-2)

$A_c = 10t^2 + A_s$, for transverse stiffeners at end support (*Eq.* B6.1-3)

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

$A_b = b_1t + A_s$, for transverse stiffeners at interior support and under concentrated load (*Eq.* B6.1-4)

$A_b = b_2t + A_s$, for transverse stiffeners at end support (*Eq.* B6.1-5)

A_s = Cross sectional area of transverse stiffeners

$b_1 = 25t\,[0.0024(L_{st}/t) + 0.72] \leq 25t$ (*Eq.* B6.1-6)

$b_2 = 12t\,[0.0044(L_{st}/t) + 0.83] \leq 12t$ (*Eq.* B6.1-7)

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.

B6.2 Shear Stiffeners

Where shear stiffeners are required, the spacing shall be such that the required shear strength shall not exceed the design shear strength, ϕ_vV_n, permitted by Section C3.2, and the ratio a/h shall not exceed $[260/(h/t)]^2$ nor 3.0.

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

$$I_{smin} = 5ht^3[h/a - 0.7(a/h)] \geq (h/50)^4 \quad (Eq.\ B6.2\text{-}1)$$

The gross area of shear stiffeners shall be not less than

$$A_{st} = \frac{1-C_v}{2}\left[\frac{a}{h} - \frac{(a/h)^2}{(a/h)+\sqrt{1+(a/h)^2}}\right]YDht \quad (Eq.\ B6.2\text{-}2)$$

where

$$C_v = \frac{45{,}000k_v}{F_y(h/t)^2} \quad \text{when } C_v \leq 0.8 \quad (Eq.\ B6.2\text{-}3)$$

$$C_v = \frac{190}{h/t}\left(\sqrt{\frac{k_v}{F_y}}\right) \quad \text{when } C_v > 0.8 \quad (Eq.\ B6.2\text{-}4)$$

$$k_v = 4.00 + \frac{5.34}{(a/h)^2} \quad \text{when } a/h \leq 1.0 \quad (Eq.\ B6.2\text{-}5)$$

$$k_v = 5.34 + \frac{4.00}{(a/h)^2} \quad \text{when } a/h > 1.0 \quad (Eq.\ B6.2\text{-}6)$$

a = Distance between transverse stiffeners

$$Y = \frac{\text{Yield point of web steel}}{\text{Yield point of stiffener steel}}$$

D = 1.0 for stiffeners furnished in pairs

D = 1.8 for single–angle stiffeners

D = 2.4 for single–plate stiffeners

t and h are as defined in Section B1.2

B6.3 Non–Conforming Stiffeners

The design strength of members with transverse stiffeners that do not meet the requirements of Section B6.1 or B6.2, such as stamped or rolled–in transverse stiffeners shall be determined by tests in accordance with Chapter F of this Specification.

C. MEMBERS

C1 Properties of Sections

Properties of sections (cross–sectional area, moment of inertia, section modulus, radius of gyration, etc.) shall be determined in accordance with conventional methods of structural design. Properties shall be based on the full cross section of the members (or net sections where the use of net section is applicable) except where the use of a reduced cross section, or effective design width, is required.

C2 Tension Members

For axially loaded tension members, the design tensile strength, $\phi_t T_n$, shall be determined as follows:

$\phi_t = 0.95$

$$T_n = A_n F_y \quad (Eq.\ C2\text{–}1)$$

where

T_n = Nominal strength of member when loaded in tension
ϕ_t = Resistance factor for tension
A_n = Net area of the cross section
F_y = Design yield stress as determined in Section A5.2.1

For tension members using bolted connections, the design tensile strength shall also be limited by Section E3.2.

C3 Flexural Members

C3.1 Strength for Bending Only

The design flexural strength, $\phi_b M_n$, shall be the smallest of the values calculated according to Sections C3.1.1, C3.1.2, and C3.1.3.

C3.1.1 Nominal Section Strength

The design flexural strength, $\phi_b M_n$, shall be determined with $\phi_b = 0.95$ for sections with stiffened or partially stiffened compression flanges and 0.90 for sections with unstiffened compression flanges, and the nominal section strength, M_n, 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:

$$M_n = S_e F_y \quad (Eq.\ C3.1.1\text{–}1)$$

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

The nominal flexural strength, M_n, shall not exceed either $1.25\ S_eF_y$ 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 $w/t \leq \lambda_1$

$$C_y = 3 - 2\left(\frac{w/t - \lambda_1}{\lambda_2 - \lambda_1}\right) \text{ for } \lambda_1 < \frac{w}{t} < \lambda_2$$

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

where

$$\lambda_1 = \frac{1.11}{\sqrt{F_y/E}} \qquad (Eq.\ C3.1.1\text{–}2)$$

$$\lambda_2 = \frac{1.28}{\sqrt{F_y/E}} \qquad (Eq.\ C3.1.1\text{–}3)$$

(b) Unstiffened compression elements

$C_y = 1$

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

$C_y = 1$

When applicable, effective design widths shall be used in calculating section properties. M_n shall be calculated considering equilibrium of stresses, assuming an ideally elastic–plastic stress–strain curve which is the same in tension as in compression, assuming small deformation and assuming that plane sections remain plane during bending. Combined bending and web crippling shall be checked by provisions of Section C3.5.

C3.1.2 Lateral Buckling Strength

The design strength of the laterally unbraced segments of singly–, doubly–, and point–symmetric sections* subject to lateral buckling, ϕ_bM_n shall be determined with $\phi_b = 0.90$ and M_n calculated as follows:

$$M_n = S_c \frac{M_c}{S_f} \qquad (Eq.\ C3.1.2\text{–}1)$$

where

* The provisions of this Section apply to I–, Z–, C– and other singly–symmetric section flexural members (not including multiple–web deck, U– and closed box–type members, and curved or arch members). The provisions of this Section do not apply to laterally unbraced compression flanges of otherwise laterally stable sections. Refer to C3.1.3 for C– and Z–purlins in which the tension flange is attached to sheathing.

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 singly–, doubly–, and point–symmetric sections:

For $M_e > 0.5M_y$

$$M_c = M_y\left(1 - \frac{M_y}{4M_e}\right) \qquad (Eq.\ C3.1.2\text{–}2)$$

For $M_e \leq 0.5M_y$

$$M_c = M_e \qquad (Eq.\ C3.1.2\text{–}3)$$

where

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

$= S_f F_y$ $\qquad$ (*Eq.* C3.1.2–4)

M_e = Elastic critical moment computed by the following equations:

M_e $= C_b r_o A\sqrt{\sigma_{ey}\sigma_t}$ for bending about the symmetry axis. For singly–symmetric sections, x–axis is the axis of symmetry oriented such that the shear center has a negative x–coordinate. For point–symmetric sections, use 0.5 M_e. $\qquad$ (*Eq.* C3.1.2–5)

Alternatively, M_e can be calculated using the formula for doubly–symmetric I–sections or point–symmetric sections given in (b)

M_e $= C_s A\sigma_{ex}\left[j + C_s\sqrt{j^2 + r_o^2(\sigma_t / \sigma_{ex})}\right] / C_{TF}$ for bending about the centroidal axis perpendicular to the symmetry axis for singly–symmetric sections only $\qquad$ (*Eq.* C3.1.2–6)

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

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

$$\sigma_{ex} = \frac{\pi^2 E}{(K_x L_x / r_x)^2} \qquad (Eq.\ C3.1.2\text{–}7)$$

$$\sigma_{ey} = \frac{\pi^2 E}{(K_y L_y / r_y)^2} \qquad (Eq.\ C3.1.2\text{–}8)$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \qquad (Eq.\ C3.1.2\text{–}9)$$

A = Full cross–sectional area

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

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

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

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

$$= \sqrt{r_x^2 + r_y^2 + x_o^2} \qquad (Eq.\ C3.1.2\text{–}10)$$

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

G = Shear modulus

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

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

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

J = St. Venant torsion constant of the cross section

C_w = Torsional warping constant of the cross section

$$j = \frac{1}{2I_y}\left[\int_A x^3 dA + \int_A xy^2 dA\right] - x_o \qquad (Eq.\ C3.1.2\text{–}11)$$

(b) For I– or Z–sections bent about the centroidal axis perpendicular to the web (x–axis):

In lieu of (a), the following equations may be used to evaluate M_c:

For $M_e \geq 2.78M_y$

$$M_c = M_y \qquad (Eq.C3.1.2\text{–}12)$$

For $2.78M_y > M_e > 0.56M_y$

$$M_c = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_e}\right) \qquad (Eq.\ C3.1.2\text{–}13)$$

For $M_e \leq 0.56M_y$

$$M_c = M_e \qquad (Eq.C3.1.2\text{–}14)$$

where

M_e = Elastic critical moment determined either as defined in (a) above or as follows:

$$= \frac{\pi^2 E C_b d I_{yc}}{L^2} \text{ for doubly-symmetric I-sections} \quad (Eq.\ C3.1.2\text{–}15)$$

$$= \frac{\pi^2 E C_b d I_{yc}}{2L^2} \text{ for point-symmetric Z-sections} \quad (Eq.\ C3.1.2\text{–}16)$$

d = Depth of section

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 (a).

C3.1.3 Beams Having One Flange Through-Fastened to Deck or Sheathing

This section does not apply to a continuous beam for the region between inflection points adjacent to a support, or to a cantilever beam.

The design flexural strength, $\phi_b M_n$, of a Channel or Z-section loaded in a plane parallel to the web, with the tension flange attached to deck or sheathing and with the compression flange laterally unbraced shall be determined with ϕ_b = 0.90 and the nominal flexural strength, M_n, calculated as follows:

$$M_n = R S_e F_y \quad (Eq.\ C3.1.3\text{–}1)$$

where

R =0.40 for simple span C sections
=0.50 for simple span Z sections
=0.60 for continuous span C sections
=0.70 for continuous span Z sections

S_e and F_y are defined in Section C3.1.1

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

(1) Member depth less than 11.5 inches
(2) The flanges are edge stiffened compression elements
(3) 60 ≤ depth/thickness ≤ 170
(4) 2.8 ≤ depth/flange width ≤ 4.5
(5) 16 ≤ flat width/thickness of flange ≤ 43
(6) For continuous span systems, the lap length at each interior support in each direction (distance from center of support to end of lap) shall not be less than:
   1.5d for Zee sections
   3.0d for Channel sections
(7) Member span length no greater than 33 feet
(8) For continuous span systems, the longest member span shall not be more than 20% greater than the shortest span
(9) Both flanges are prevented from moving laterally at the supports
(10) Roof or wall panels shall be steel sheets, minimum of 0.019 in. coated thickness, having a minimum rib depth of 1 in., spaced a maximum of 12 in. on

centers and attached in a manner to effectively inhibit relative movement between the panel and purlin flange

(11) Insulation shall be glass fiber blanket 0 to 6 inches thick compressed between the member and panel in a manner consistent with the fastener being used

(12) Fastener type: minimum No. 12 self–drilling or self–tapping sheet metal screws or 3/16 – in. rivets, washers 1/2 in. diameter

(13) Fasteners shall not be standoff type screws

(14) Fasteners shall be spaced not greater than 12 in. on centers and placed near the center of the beam flange

If variables fall outside any of the above stated limits, the user must perform full scale tests in accordance with Section F1 of the Specification, or apply another rational analysis procedure. In any case, the user is permitted to perform tests, in accordance with Section F1, as an alternate to the procedure described in this section.

C3.2 Strength for Shear Only

The design shear strength, $\phi_v V_n$, at any section shall be calcuated as follows:

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

$\phi_v = 1.0$

$$V_n = 0.577 F_y ht \qquad (Eq.\ C3.2.\text{–}1)$$

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

$\phi_v = 0.90$

$$V_n = 0.64t^2\sqrt{k_v F_y E} \qquad (Eq.\ C3.2\text{–}2)$$

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

$\phi_v = 0.90$

$$V_n = 0.905Ek_v t^3/h \qquad (Eq.\ C3\text{–}2.3)$$

where

ϕ_v = Resistance factor for shear

V_n = Nominal shear strength of beam

t = Web thickness

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

k_v = Shear buckling coefficient determined as follows:

1. For unreinforced webs, $k_v = 5.34$
2. For beam webs with transverse stiffeners satisfying the requirements of Section B6

when $a/h \le 1.0$

$$k_v = 4.00 + \frac{5.34}{(a/h)^2} \qquad (Eq.\ C3.2\text{–}4)$$

when $a/h > 1.0$

$$k_v = 5.34 + \frac{4.00}{(a/h)^2} \qquad (Eq.\ C3.2\text{–}5)$$

where

a = the shear panel length for unreinforced web element
= distance between transverse stiffeners for reinforced web elements.

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

C3.3 Strength for Combined Bending and Shear

For beams with unreinforced webs, the required flexural strength, M_u, and the required shear strength, V_u, shall satisfy the following interaction equation:

$$\left(\frac{M_u}{\phi_b M_{nxo}}\right)^2 + \left(\frac{V_u}{\phi_v V_n}\right)^2 \le 1.0 \qquad (Eq.\ C3.3\text{–}1)$$

For beams with transverse web stiffeners, the required flexural strength, M_u, and the required shear strength, V_u, shall not exceed $\phi_b M_n$ and $\phi_v V_n$, respectively. When $M_u/(\phi_b M_{nxo}) > 0.5$ and $V_u/(\phi_v V_n) > 0.7$, then M_u and V_u shall satisfy the following interaction equation:

$$0.6\left(\frac{M_u}{\phi_b M_{nxo}}\right) + \left(\frac{V_u}{\phi_v V_n}\right) \le 1.3 \qquad (Eq.\ C3.3\text{–}2)$$

In the above:

ϕ_b = Resistance factor for bending (See Section C3.1)
ϕ_v = Resistance factor for shear (See Section C3.2)
M_n = Nominal flexural strength when bending alone exists
M_{nxo} = Nominal flexural strength about the centroidal x–axis determined in accordance with Section C3.1 excluding the provisions of Section C3.1.2
V_n = Nominal shear strength when shear alone exists

C3.4 Web Crippling Strength

These provisions are applicable to webs of flexural members subject to concentrated loads or reactions, or the components thereof, acting perpendicular to the longitudinal axis of the member, and in the plane of the web under consideration, and causing compressive stresses in the web.

To avoid crippling of unreinforced flat webs of flexural members having a flat width ratio, h/t, equal to or less than 200, the required strength for the concentrated loads and reactions shall not exceed the values of $\phi_w P_n$, with $\phi_w = 0.75$ and 0.80 for single unreinforced webs and I–sections, respectively, and P_n given in Table C3.4–1. Webs of flexural members for which h/t is greater than 200 shall be provided with adequate means of transmitting concentrated loads and/or reactions directly into the webs.

The formulas in Table C3.4–1 apply to beams when $R/t \le 6$ and to deck when $R/t \le 7$, $N/t \le 210$ and $N/h \le 3.5$.

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

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

TABLE C3.4-1
P_n

		Shapes Having Single Webs		I-Sections or Similar Sections(1)
		Stiffened or Partially Stiffened Flanges	Unstiffened Flanges	Stiffened, Partially Stiffened and Unstiffened Flanges
Opposing Loads Spaced > 1.5h(2)	End Reaction(3)	*Eq.* C3.4-1	*Eq.* C3.4-2	*Eq.* C3.4-3
	Interior Reaction(4)	*Eq.* C3.4-4	*Eq.* C3.4-4	*Eq.* C3.4-5
Opposing Loads Spaced ≤ 1.5h(5)	End Reaction(3)	*Eq.* C3.4-6	*Eq.* C3.4-6	*Eq.* C3.4-7
	Interior Reaction(4)	*Eq.* C3.4-8	*Eq.* C3.4-8	*Eq.* C3.4-9

Footnotes and Equation References to Table C3.4-1:

(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 C3.4-1:

$$t^2 kC_3C_4C_\theta[331 - 0.61(h/t)]\,[1 + 0.01(N/t)] \quad (Eq.\ C3.4\text{-}1)$$

$$t^2 kC_3C_4C_\theta[217 - 0.28(h/t)]\,[1 + 0.01(N/t)] \quad (Eq.\ C3.4\text{-}2)$$

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

$$t^2F_yC_6\left(10.0 + 1.25\sqrt{N/t}\right) \quad (Eq.\ C3.4\text{-}3)$$

$$t^2 kC_1C_2C_\theta[538 - 0.74(h/t)]\,[1 + 0.007(N/t)] \quad (Eq.\ C3.4\text{-}4)$$

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

$$t^2F_yC_5(0.88+0.12m)\left(15.0 + 3.25\sqrt{N/t}\right) \quad (Eq.\ C3.4\text{-}5)$$

$$t^2 kC_3C_4C_\theta[244 - 0.57(h/t)]\,[1 + 0.01(N/t)] \quad (Eq.\ C3.4\text{-}6)$$

$$t^2F_yC_8(0.64+0.31m)\left(10.0 + 1.25\sqrt{N/t}\right) \quad (Eq.\ C3.4\text{-}7)$$

$$t^2 kC_1C_2C_\theta[771 - 2.26(h/t)]\,[1 + 0.0013(N/t)] \quad (Eq.\ C3.4\text{-}8)$$

$$t^2F_yC_7(0.82+0.15m)\left(15.0 + 3.25\sqrt{N/t}\right) \quad (Eq.\ C3.4\text{-}9)$$

In the above-referenced formulas:

ϕ_w = Resistance factor for web crippling

P_n = Nominal strength for concentrated load or reaction per web

$$C_1 = (1.22 - 0.22k) \quad (Eq.\ C3.4\text{-}10)$$

$$C_2 = (1.06 - 0.06R/t) \le 1.0 \quad (Eq.\ C3.4\text{-}11)$$

$C_3 = (1.33 - 0.33k)$ (*Eq.* C3.4–12)

$C_4 = (1.15 - 0.15R/t) \leq 1.0$ but not less than 0.50 (*Eq.* C3.4–13)

$C_5 = (1.49 - 0.53k) \geq 0.6$ (*Eq.* C3.4–14)

$C_6 = 1 + \left(\frac{h/t}{750}\right)$ when $h/t \leq 150$ (*Eq.* C3.4–15)

$= 1.20$, when $h/t > 150$ (*Eq.* C3.4–16)

$C_7 = 1/k$, when $h/t \leq 66.5$ (*Eq.* C3.4–17)

$= \left[1.10 - \frac{h/t}{665}\right]\frac{1}{k}$, when $h/t > 66.5$ (*Eq.* C3.4–18)

$C_8 = \left[0.98 - \frac{h/t}{865}\right]\frac{1}{k}$ (*Eq.* C3.4–19)

$C_\theta = 0.7 + 0.3\,(\theta/90)^2$ (*Eq.* C3.4–20)

F_y = Design yield stress of the web, see Section A5.2.1

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

$k = F_y/33$ (*Eq.* C3.4–21)

$m = t/0.075$ (*Eq.* C3.4–22)

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 $\geq 45°$, but not more than 90°

C3.5 Combined Bending and Web Crippling Strength

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:

$$1.07\left(\frac{P_u}{\phi_w P_n}\right) + \left(\frac{M_u}{\phi_b M_{nxo}}\right) \leq 1.42 \quad (Eq.\ C3.5\text{–}1)$$

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);

$$0.82\left(\frac{P_u}{\phi_w P_n}\right) + \left(\frac{M_u}{\phi_b M_{oxo}}\right) \leq 1.32 \quad (Eq.\ C3.5\text{–}2)$$

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

In the above formulas:

ϕ_b = Resistance factor for bending (See Section 3.1)

ϕ_w = Resistance factor for web crippling (See Section C3.4)

P_u = Required strength for the concentrated load or reaction in the presence of bending moment

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

M_u = Required flexural strength at, or immediately adjacent to, the point of application of the concentrated load or reaction P_u

M_{nxo} = Nominal flexural strength about the centroidal x–axis determined in accordance with Section C3.1 excluding the provisions of Section C3.1.2

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

t = Thickness of the web or flange

λ = Slenderness factor given by Section B2.1

C4 Concentrically Loaded Compression Members

This section applies to 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 this section.

(a) The design axial strength, $\phi_c P_n$, shall be calculated as follows:

$\phi_c = 0.85$

$$P_n = A_e F_n \quad (Eq.\ C4\text{–}1)$$

where

A_e = Effective area at the stress F_n. For sections with circular holes, A_e shall be determined according to Section B2.2a, subject to the limitations of that section. If the number of holes in the effective length region times the hole diameter divided by the effective length does not exceed 0.015, A_e can be determined ignoring the holes.

F_n is determined as follows:

For $F_e > F_y/2$ $\quad F_n = F_y(1 - F_y/4F_e)$ $\quad$ (*Eq.* C4–2)

For $F_e \le F_y/2$ $\quad F_n = F_e$ $\quad$ (*Eq.* C4–3)

F_e is the least of the elastic flexural, torsional and torsional–flexural buckling stress determined according to Sections C4.1 through C4.3.

(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:

$$P_n = \frac{A\pi^2 E}{25.7(w/t)^2} \quad (Eq.\ C4\text{–}4)$$

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 required axial strength, P_u, acting simultaneously with a moment equal to $P_uL/1000$ 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.

C4.1 Sections Not Subject to Torsional or Torsional–Flexural Buckling

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:

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \qquad (Eq.\ C4.1\text{–}1)$$

where

E = Modulus of elasticity

K = Effective length factor*

L = Unbraced length of member

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

C4.2 Doubly– or Singly–Symmetric Sections Subject to Torsional or Torsional–Flexural Buckling

For sections subject to torsional or torsional–flexural buckling, F_e shall be taken as the smaller of F_e calculated according to Section C4.1 and F_e calculated as follows:

$$F_e = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\right] \qquad (Eq.\ C4.2\text{–}1)$$

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

$$F_e = \frac{\sigma_t \sigma_{ex}}{\sigma_t + \sigma_{ex}} \qquad (Eq.\ C4.2\text{–}2)$$

where σ_t and σ_{ex} are as defined in C3.1.2(a) :

$$\beta = 1 - (x_o/r_o)^2 \qquad (Eq.\ C4.2\text{–}3)$$

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

C4.3 Nonsymmetric Sections

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. Alternatively, compression members composed of such shapes may be tested in accordance with Chapter F.

C5 Combined Axial Load and Bending

The required strengths P_u, M_{ux}, and M_{uy} shall satisfy the following interaction equations:

$$\frac{P_u}{\phi_c P_n} + \frac{C_{mx} M_{ux}}{\phi_b M_{nx} \alpha_{nx}} + \frac{C_{my} M_{uy}}{\phi_b M_{ny} \alpha_{ny}} \leq 1.0 \qquad (Eq.\ C5\text{–}1)$$

$$\frac{P_u}{\phi_c P_{no}} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0 \qquad (Eq.\ C5\text{–}2)$$

* In frames where lateral stability is provided by diagonal bracing, shear walls, attachment to an adjacent structure having adequate lateral stability, or floor slabs or roof decks secured horizontally by walls or bracing systems parallel to the plane of the frame, and in trusses, the effective length factor, K, for compression members which do not depend upon their own bending stiffness for lateral stability of the frame or truss, shall be taken as unity, unless analysis shows that a smaller value may be used. In a frame which depends upon its own bending stiffness for lateral stability, the effective length, KL, of the compression members shall be determined by a rational method and shall not be less than the actual unbraced length.

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

$$\frac{P_u}{\phi_c P_n} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0 \qquad (Eq.\ C5\text{-}3)$$

where

P_u = Required axial strength

M_{ux} and M_{uy} = Required flexural strengths with respect to the centroidal axes of the effective section determined for the required axial strength alone. For angle sections, M_{uy} shall be taken either as the required flexural strength or the required flexural strength plus $P_uL/1000$, whichever results in a lower value of P_n.

P_n = Nominal axial strength determined in accordance with Section C4

P_{no} = Nominal axial strength determined in accordance with Section C4, with $F_n = F_y$

M_{nx} and M_{ny} = Nominal flexural strengths about the centroidal axes determined in accordance with Section C3

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

$$= 1\bigg/\left[1 - \frac{P_u}{\phi_c P_E}\right] \qquad (Eq.\ C5\text{-}4)$$

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

ϕ_c = 0.85

$$P_E = \frac{\pi^2 E I_b}{(K_b L_b)^2} \qquad (Eq.\ C5\text{-}5)$$

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

   $$C_m = 0.6 - 0.4\,(M_1/M_2) \qquad (Eq.\ C5\text{-}6)$$

   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, $C_m = 1.0$.

C6 Cylindrical Tubular Members

The requirements of this Section apply to cylindrical tubular members having a ratio of outside diameter to wall thickness, D/t, not greater than $0.441\ E/F_y$.

C6.1 Bending

For flexural members, the required flexural strength uncoupled from axial load, shear, and local concentrated forces or reactions shall not exceed $\phi_b M_n$, where $\phi_b = 0.95$ and M_n is calculated as follows:

For $D/t \leq 0.070\ E/F_y$

$$M_n = 1.25\ F_y S_f \qquad (Eq.\ C6.1\text{–}1)$$

For $0.070\ E/Fy < D/t \leq 0.319\ E/F_y$

$$M_n = \left[0.970 + 0.020\left(\frac{E/F_y}{D/t}\right)\right] F_y S_f \qquad (Eq.\ C6.1\text{–}2)$$

For $0.319\ E/F_y < D/t \leq 0.441\ E/F_y$

$$M_n = [0.328E/(D/t)]S_f \qquad (Eq.\ C6.1\text{–}3)$$

where

S_f = Elastic section modulus of the full, unreduced cross section

C6.2 Compression

The requirements of this Section apply to members in which the resultant of all loads and moments acting on the member is equivalent to a single force in the direction of the member axis passing through the centroid of the section.

The design axial strength, $\phi_c P_n$, shall be calculated as follows:

$\phi_c = 0.85$

$$P_n = F_n A_e \qquad (Eq.\ C6.2\text{–}1)$$

In the above equation,

For $F_e > F_y/2$

F_n = Flexural buckling stress

$$= F_y\ [1 - F_y/4F_e] \qquad (Eq.\ C6.2\text{–}2)$$

F_e = The elastic flexural buckling stress determined according to Section C4.1

$$A_e = [1 - (1 - R^2)(1 - A_o/A)]A \qquad (Eq.\ C6.2\text{–}3)$$

$$R = \sqrt{F_y / 2F_e} \qquad (Eq.\ C6.2\text{–}4)$$

$$A_o = \left[\frac{0.037}{\frac{DF_y}{tE}} + 0.667\right] A \leq A \text{ for } \frac{D}{t} \leq 0.441 \frac{E}{F_y} \qquad (Eq.\ C6.2\text{–}5)$$

A = Area of the unreduced cross section

For $F_e \leq F_y/2$

$F_n = F_e$

$A_e = A$

C6.3 Combined Bending and Compression

Combined bending and compression shall satisfy the provisions of Section C5.

D. STRUCTURAL ASSEMBLIES

D1 Built–Up Sections

D1.1 I – Sections Composed of Two Channels

The maximum permissible longitudinal spacing of welds or other connectors, s_{max}, joining two channels to form an I–section shall be

(a) For compression members:

$$s_{max} = \frac{L r_{cy}}{2 r_I} \qquad (Eq.\ D1.1\text{–}1)$$

where

L = Unbraced length of compression member

r_I = Radius of gyration of the I–section about the axis perpendicular to the direction in which buckling would occur for the given conditions of end support and intermediate bracing

r_{cy} = Radius of gyration of one channel about its centroidal axis parallel to the web

(b) For flexural members:

$$s_{max} = L/6 \qquad (Eq.\ D1.1\text{–}2)$$

In no case shall the spacing exceed the value

$$s_{max} = \frac{2gT_s}{mq} \qquad (Eq.\ D1.1\text{–}3)$$

where

L = Span of beam

T_s = Design strength of connection in tension (Section E)

g = Vertical distance between the two rows of connections nearest to the top and bottom flanges

q = Intensity of factored load on the beam (For methods of determination, see below)

m = Distance from the shear center of one channel to the mid–plane of its web. For simple channels without stiffening lips at the outer edges,

$$m = \frac{w_f^2}{2w_f + d/3} \qquad (Eq.\ D1.1\text{–}4)$$

For channels with stiffening lips at the outer edges,

$$m = \frac{w_f dt}{4I_x}\left[w_f d + 2D\left(d - \frac{4D^2}{3d}\right)\right] \qquad (Eq.\ D1.1\text{–}5)$$

w_f = Projection of flanges from the inside face of the web (For channels with flanges of unequal width, w_f shall be taken as the width of the wider flange)

d = Depth of channel or beam

D = Overall depth of lip

I_x = Moment of inertia of one channel about its centroidal axis normal to the web.

The intensity of factored load, q, is obtained by dividing the magnitude of factored concentrated loads or reactions by the length of bearing. For beams designed for a uniformly distributed load, q shall be taken equal to three times the intensity of the uniformly distributed factored load. If the length of bearing of a concentrated load or reaction is smaller than the weld spacing, s, the required design strength of the welds or connections closest to the load or reaction is

$$T_s = P_s m/2g \qquad (Eq.\ D1.1\text{–}6)$$

where P_s is a concentrated load or reaction based on factored loads.

The required maximum spacing of connections, s_{max}, depends upon the intensity of the factored load directly at the connection. Therefore, if uniform spacing of connections is used over the whole length of the beam, it shall be determined at the point of maximum local load intensity. In cases where this procedure would result in uneconomically close spacing, either one of the following methods may be adopted: (a) the connection spacing may be varied along the beam according to the variation of the load intensity; or (b) reinforcing cover plates may be welded to the flanges at points where concentrated loads occur. The design shear strength of the connections joining these plates to the flanges shall then be used for T_s, and g shall be taken as the depth of the beam.

D1.2 Spacing of Connections in Compression Elements

The spacing, s, in the line of stress, of welds, rivets, or bolts connecting a cover plate, sheet, or a non–integral stiffener in compression to another element shall not exceed

(a) that which is required to transmit the shear between the connected parts on the basis of the design strength per connection specified elsewhere herein; nor

(b) $1.16t\sqrt{(E/f_c)}$, where t is the thickness of the cover plate or sheet, and f_c is the stress at service load in the cover plate or sheet; nor

(c) three times the flat width, w, of the narrowest unstiffened compression element tributary to the connections, but need not be less than $1.11t\sqrt{(E/F_y)}$ if $w/t < 0.50\sqrt{(E/F_y)}$, or $1.33t\sqrt{(E/F_y)}$ if $w/t \geq 0.50\sqrt{(E/F_y)}$, unless closer spacing is required by (a) or (b) above.

In the case of intermittent fillet welds parallel to the direction of stress, the spacing shall be taken as the clear distance between welds, plus one–half inch. In all other cases, the spacing shall be taken as the center–to–center distance between connections.

Exception: The requirements of this Section do not apply to cover sheets which act only as sheathing material and are not considered as load–carrying elements.

D2 Mixed Systems

The design of members in mixed systems using cold–formed steel components in conjunction with other materials shall conform to this Specification and the applicable Specification of the other material.

D3 Lateral Bracing

Braces shall be designed to restrain lateral bending or twisting of a loaded beam or column, and to avoid local crippling at the points of attachment.

D3.1 Symmetrical Beams and Columns

Braces and bracing systems, including connections, shall be designed considering strength and stiffness requirements.

D3.2 Channel–Section and Z–Section Beams

The following provisions for bracing to restrain twisting of channels and Z–sections used as beams loaded in the plane of the web, apply only when (a) the top flange is connected to deck or sheathing material in such a manner as to effectively restrain lateral deflection of the connected flange*, or (b) neither flange is so connected. When both flanges are so connected, no further bracing is required.

* Where the Specification does not provide an explicit method for design, further information should be obtained from the Commentary.

D3.2.1 Anchorage of Bracing for Roof Systems Under Gravity Load With Top Flange Connected to Sheathing

For channels and Z–sections designed according to Section C3.1.1, and having deck or sheathing fastened directly to the top flanges in such a manner shown to effectively inhibit relative movement between the deck or sheathing and the purlin flange, provisions shall be made to restrain the flanges so that the maximum top flange lateral displacements with respect to the purlin reaction points do not exceed the span length divided by 360. If the top flanges of all purlins face in the same direction, anchorage of the restraint system must be capable of satisfying the requirements of Sections D3.2.1(a) and D3.2.1(b). If the top flanges of adjacent lines of purlins face in opposite directions, the provisions of Section D3.2.1(a) and D3.2.1(b) do not apply.

Anchored braces need to be connected to only one line of purlins in each purlin bay of each roof slope if provision is made to transmit forces from other purlin lines through the roof deck and its fastening system. Anchored braces shall be as close as possible to the flange which is connected to the deck or sheathing. Anchored braces shall be provided for each purlin bay.

For bracing arrangements other than those covered in Sections D3.2.1(a) and D3.2.1(b), tests in accordance with Chapter F shall be performed so that the type and/or spacing of braces selected are such that the test strength of the braced Z–section assembly is equal to or greater than its nominal flexural strength, instead of that required by Chapter F.

(a) Channel Sections

For roof systems using channel sections for purlins with all compression flanges facing in the same direction, a restraint system capable of resisting 0.05W, in addition to other loading, shall be provided where W is the factored load supported by all purlin lines being restrained. Where more than one brace is used at a purlin line, the restraint force 0.05W shall be divided equally between all braces.

(b) Z–Sections

For roof systems having a diaphragm stiffness of at least 2,000 lb/in., having four to twenty Z–purlin lines with all top flanges facing in the direction of the upward roof slope, and with restraint braces at the purlin supports, midspan or one–third points, each brace shall be designed to resist a force determined as follows:

(1) Single–Span System with Restraints at the Supports:

$$P_L = 0.5\left[\frac{0.220b^{1.50}}{n_p^{0.72}d^{0.90}t^{0.60}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{–}1)$$

(2) Single–Span System with Third–Point Restraints:

$$P_L = 0.5\left[\frac{0.474b^{1.22}}{n_p^{0.57}d^{0.89}t^{0.33}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{–}2)$$

(3) Single–Span System with Midspan Restraint:

$$P_L = \left[\frac{0.224b^{1.32}}{n_p^{0.65}d^{0.83}t^{0.50}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{–}3)$$

(4) Multiple–Span System with Restraints at the Supports:

$$P_L = C_{tr}\left[\frac{0.053b^{1.88}L^{0.13}}{n_p^{0.95}d^{1.07}t^{0.94}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{–}4)$$

with

C_{tr} = 0.63 for braces at end supports of multiple-span systems
C_{tr} = 0.87 for braces at the first interior supports
C_{tr} = 0.81 for all other braces

(5) Multiple-Span System with Third-Point Restraints:

$$P_L = C_{th}\left[\frac{0.181b^{1.15}L^{0.25}}{n_p^{0.54}d^{1.11}t^{0.29}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{–}5)$$

with

C_{th} = 0.57 for outer braces in exterior spans
C_{th} = 0.48 for all other braces

(6) Multiple-Span System with Midspan Restraints:

$$P_L = C_{ms}\left[\frac{0.116b^{1.32}L^{0.18}}{n_p^{0.70}d^{1.00}t^{0.50}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{–}6)$$

with

C_{ms} = 1.05 for braces in exterior spans
C_{ms} = 0.90 for all other braces

where

b = Flange width, in.
d = Depth of section, in.
t = Thickness, in.
L = Span length, in.
θ = Angle between the vertical and the plane of the web of the Z-section, degrees
n_p = Number of parallel purlin lines
W = Total factored load supported by the purlin lines between adjacent supports, pounds

The force, P_L, is positive when restraint is required to prevent movement of the purlin flanges in the upward roof slope direction.

For systems having less than four purlin lines, the brace force can be determined by taking 1.1 times the force found from Equations D3.2.1-1 through D3.2.1-6, with $n_p = 4$. For systems having more than twenty purlin lines, the brace force can be determined from Equations D3.2.1-1 through D3.2.1-6, with $n_p = 20$.

D3.2.2 Neither Flange Connected to Sheathing

Each intermediate brace, at the top and bottom flange, shall be designed to resist a required lateral force, P_L, determined as follows:

(a) For uniform loads, $P_L = 1.5K'$ times the factored load within a distance 0.5a each side of the brace.

(b) For concentrated loads, $P_L = 1.0K'$ times each concentrated load within a distance 0.3a each side of the brace, plus $1.4K'(1-x/a)$ times each factored concentrated load located farther than 0.3a but not farther than 1.0a from the brace.

In the above formulas:

For channels and Z–sections:

x = Distance from the concentrated load to the brace

a = Distance between center line of braces

For channels:

$$K' = m/d \qquad (Eq.\ D3.2.2\text{–}1)$$

where

m = Distance from the shear center to the mid–plane of the web, as specified in Section D1.1

d = Depth of channel

For Z–sections:

$$K' = I_{xy}/I_x \qquad (Eq.\ D3.2.2\text{–}2)$$

where

I_{xy} = Product of inertia of the full section about centroidal axes parallel and perpendicular to the web

I_x = Moment of inertia of the full section about the centroidal axis perpendicular to the web

Braces shall be designed to avoid local crippling at the points of attachment to the member.

Braces shall be attached both to the top and bottom flanges of the sections, at the ends and at intervals not greater than one–quarter of the span length, in such a manner as to prevent tipping at the ends and lateral deflection of either flange in either direction at intermediate braces. If one–third or more of the total factored load on the beam is concentrated over a length of one–twelfth or less of the span of the beam, an additional brace shall be placed at or near the center of this loaded length.

Exception: When all loads and reactions on a beam are transmitted through members which frame into the section in such a manner as to effectively restrain the section against rotation and lateral displacement, no other braces will be required.

D3.3 Laterally Unbraced Box Beams

For closed box–type sections used as beams subject to bending about the major axis, the ratio of the laterally unsupported length to the distance between the webs of the section shall not exceed 0.086 E/F_y.

D4 Wall Studs and Wall Stud Assemblies

The design strength of a stud may be computed on the basis of Section C (neglecting sheathing and using steel only) or on the basis that sheathing (attached to one or both sides of the stud) furnishes adequate lateral and rotational support to the stud in the plane of the wall, provided that the stud, sheathing, and attachments comply with the following requirements:

Both ends of the stud shall be braced to restrain rotation about the longitudinal stud axis and horizontal displacement perpendicular to the stud axis; however, the ends may or may not be free to rotate about both axes perpendicular to the stud axis. The sheathing shall be connected to the top and bottom members of the wall assembly to enhance the restraint provided to the stud and stabilize the overall assembly.

When sheathing is utilized for stability of the wall studs, the sheathing shall retain adequate strength and stiffness for the expected service life of the wall and additional bracing shall be provided as required for adequate structural integrity during construction and in the completed structure.

The equations given are based on solid-web steel studs and are applicable within the following limits:

Yield point, $F_y \leq$ 50 ksi
Section depth, $d \leq 6.0$ in.
Thickness, $t \leq 0.075$ in.
Overall length, $L \leq 16$ ft.
Stud spacing, B, not less than 12 in. nor greater than 24 in.
Studs with perforations shall be designed using the results of stub column tests and/or rational analysis.

D4.1 Wall Studs in Compression

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing* , the design axial strength, $\phi_c P_n$, shall be calculated as follows:

$\phi_c = 0.85$

$$P_n = A_e F_n \qquad (Eq.\ D4.1-1)$$

where

ϕ_c = Resistance factor for axial compression
A_e = Effective area determined at F_n
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 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 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

$$\sigma_{CR} = \sigma_{cy} + \overline{Q}_a \qquad (Eq.\ D4.1-2)$$

$$\sigma_{CR} = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_{tQ}) - \sqrt{(\sigma_{ex} + \sigma_{tQ})^2 - (4\beta\sigma_{ex}\sigma_{tQ})}\right] \qquad (Eq.\ D4.1-3)$$

(2) Z-Sections

$$\sigma_{CR} = \sigma_t + \overline{Q}_t \qquad (Eq.\ D4.1-4)$$

$$\sigma_{CR} = \frac{1}{2}\left\{(\sigma_{ex} + \sigma_{cy} + \overline{Q}_a) - \sqrt{\left[(\sigma_{ex} + \sigma_{cy} + \overline{Q}_a)^2 - 4(\sigma_{ex}\sigma_{cy} + \sigma_{ex}\overline{Q}_a - \sigma^2_{exy})\right]}\right\} \qquad (Eq.\ D4.1-5)$$

(3) I-Sections (doubly-symmetric)

$$\sigma_{CR} = \sigma_{ey} + \overline{Q}_a \qquad (Eq.\ D4.1-6)$$

$$\sigma_{CR} = \sigma_{ex} \qquad (Eq.\ D4.1-7)$$

In the above formulas:

$$\sigma_{ex} = \frac{\pi^2 E}{(L / r_x)^2} \qquad (Eq.\ D4.1-8)$$

$$\sigma_{exy} = (\pi^2 E I_{xy}) / (AL^2) \qquad (Eq.\ D4.1-9)$$

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*Studs with sheathing on one flange only, or with unidentical sheathing on both flanges, or having rotational restraint that is not neglected, or having any combination of the above, shall be designed in accordance with the same basic analysis principles used in deriving the provisions of this Section.

$$\sigma_{cy} = \frac{\pi^2 E}{(L/r_y)^2} \quad \text{(Eq. D4.1-10)}$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{L^2}\right] \quad \text{(Eq. D4.1-11)}$$

$$\sigma_{tQ} = \sigma_t + \overline{Q}_t \quad \text{(Eq. D4.1-12)}$$

$\overline{Q}$ = $\overline{q}$ B = Design shear rigidity for sheathing on both sides of the wall assembly

$\overline{q}$ = Design shear rigidity for sheathing per inch of stud spacing (see Table D4)

B = Stud spacing

$$\overline{Q}_a = \overline{Q}/A \quad \text{(Eq. D4.1-13)}$$

A = Area of full unreduced cross section

L = Length of stud

$$\overline{Q}_t = (\overline{Q}d^2)/(4Ar_o^2) \quad \text{(Eq. D4.1-14)}$$

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, $\overline{\gamma}$. The shear strain, γ, shall be determined as follows:

$$\gamma = (\pi/L)\,[C_1 + (E_1\, d/2)] \quad \text{(Eq. D4.1-15)}$$

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 and C–Sections

$$C_1 = (F_n C_o)/(\sigma_{cy} - F_n + \overline{Q}_a) \quad \text{(Eq. D4.1-16)}$$

$$E_1 = \frac{F_n\left[(\sigma_{ex} - F_n)(r_o^2 E_o - x_o D_o) - F_n x_o (D_o - x_o E_o)\right]}{(\sigma_{ex} - F_n) r_o^2 (\sigma_{tQ} - F_n) - (F_n x_o)^2} \quad \text{(Eq. D4.1-17)}$$

(2) Z–Sections

$$C_1 = \frac{F_n\left[C_o(\sigma_{ex} - F_n) - D_o \sigma_{exy}\right]}{(\sigma_{cy} - F_n + \overline{Q}_a)(\sigma_{ex} - F_n) - \sigma_{exy}^2} \quad \text{(Eq. D4.1-18)}$$

$$E_1 = (F_n E_o)/(\sigma_{tQ} - F_n) \quad \text{(Eq. D4.1-19)}$$

(3) I–Sections

$$C_1 = (F_n C_o)/(\sigma_{cy} - F_n + \overline{Q}_a) \quad \text{(Eq. D4.1-20)}$$

$$E_1 = 0$$

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 (*Eq.* D4.1–21)

D_o = L/700 in a direction perpendicular to the wall (*Eq.* D4.1–22)

E_o = L/(d × 10,000), rad., a measure of the initial twist of the stud from the initial, ideal, unbuckled shape (*Eq.* D4.1–23)

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

$$E' = 4EF_n(F_y - F_n)/F_y^2 \quad \text{(Eq. D4.1-24)}$$

$$G' = G(E'/E) \quad \text{(Eq. D4.1-25)}$$

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 (see Commentary), or from the small–scale–test values given in Table D4.

TABLE D4
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) $\bar{q} = \bar{q}_o\,(2 - s/12)$ (*Eq.* D4.1–26)

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 (see Commentary).

D4.2 Wall Studs in Bending

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing,* the design flexural strengths are $\phi_b M_{nxo}$ and $\phi_b M_{nyo}$

where

ϕ_b = 0.95 for sections with stiffened or partially stiffened compression flanges
= 0.90 for sections with unstiffened compression flanges

M_{nxo} and M_{nyo} = Nominal flexural strengths about the centroidal axes determined in accordance with Section C3.1, excluding the provisions of Section C3.1.2 (lateral buckling)

D4.3 Wall Studs with Combined Axial Load and Bending

The required axial strength and flexural strength shall satisfy the interaction equations of Section C5 with the following redefined terms:

P_n = Nominal axial strength determined according to Section D4.1

M_{nx} and M_{ny} in Equations C5–1, C5–2 and C5–3 shall be replaced by nominal flexural strengths, M_{nxo} and M_{nyo}, respectively.

* Studs with sheathing on one flange only, or with unidentical sheathing on both flanges, or having rotational restraint that is not neglected, or having any combination of the above, shall be designed in accordance with the same basic analysis principles used in deriving the provisions of this Section.

E. CONNECTIONS AND JOINTS

E1 General Provisions

Connections shall be designed to transmit the maximum forces resulting from the factored loads acting on the connected member. Proper regard shall be given to eccentricity.

E2 Welded Connections

The following LRFD design criteria govern welded connections used for cold–formed steel structural members in which the thickness of the thinnest connected part is 0.18 in. or less. For welded connections in which the thickness of the thinnest connected part is greater than 0.18 in., refer to the AISC's "Load and Resistance Factor Design Specification for Structural Steel Buildings".

Except as modified herein, arc welds on steel where at least one of the connected parts is 0.18 inch or less in thickness shall be made in accordance with the AWS D–1.3 (Reference 3 of Section A6) and its Commentary. Welders and welding procedures shall be qualified as specified in AWS D1.3. These provisions are intended to cover the welding positions as shown in Table E2.

Resistance welds shall be made in conformance with the procedures given in AWS C1.1, "Recommended Practices for Resistance Welding" or AWS C1.3, "Recommended Practice for Resistance Welding Coated Low Carbon Steels."

TABLE E2

Connection	Welding Position					
	Square Groove Butt Weld	Arc Spot Weld	Arc Seam Weld	Fillet Weld, Lap or T	Flare–Bevel Groove	Flare–V Groove Weld
Sheet to Sheet	F H V OH	— — — —	F H — —	F H V OH	F H V OH	F H V OH
Sheet to Supporting Member	— — — —	F — — —	F — — —	F H V OH	F H V OH	— — — —

(F = flat, H = horizontal, V = vertical, OH = overhead)

The required strength on each weld shall not exceed the design strength, ϕP_n,
where

ϕ = Resistance factor for arc welded connections defined in Sections E2.1 through E2.5.
P_n = Nominal strength of welds determined according to Sections E2.1 through E2.5.

E2.1 Groove Welds in Butt Joints

The design strength, ϕP_n, of a groove weld in a butt joint, welded from one or both sides, shall be determined as follows:

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

$\phi = 0.90$

$P_n = Lt_eF_y$ (*Eq.* E2.1–1)

(b) Shear on the effective area

$\phi = 0.80$

$P_n = Lt_e(0.6F_{xx})$; and (*Eq. E2.1–2)*

$\phi = 0.90$

$P_n = Lt_e\left(F_y / \sqrt{3}\right)$ (*Eq.* E2.1–3)

where

ϕ = Resistance factor for welded connections

P_n = Nominal strength of a groove weld

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

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

L = Length of weld

t_e = Effective throat dimension for groove weld

E2.2 Arc Spot Welds

Arc spot welds permitted by this 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, Figures E2.2(A) and E2.2(B), 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.

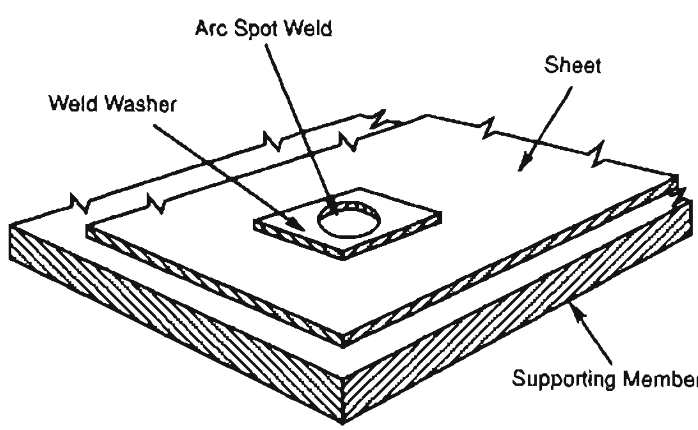


Figure E2.2A Typical Weld Washer

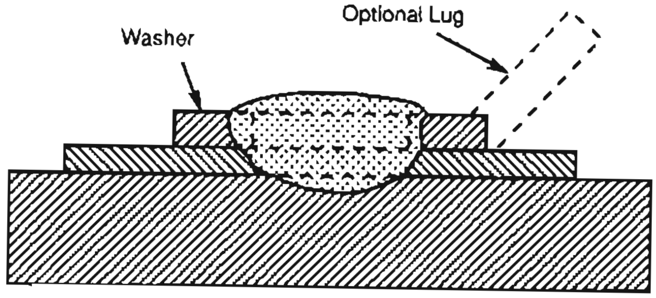


Figure E2.2B Arc Spot Weld Using Washer

Arc spot welds shall be specified by minimum effective diameter of fused area, d_e. Minimum allowable effective diameter is $^3/_8$ inch.

The design 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) $\phi = 0.60$

$P_n = 0.589\, d_e^2\, F_{xx}$; or (*Eq.* E2.2–1)

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

$\phi = 0.60$

$P_n = 2.20\, t\, d_a\, F_u$ (*Eq.* E2.2–2)

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

$\phi = 0.50$

$$P_n = 0.280\left[1 + 5.59\frac{\sqrt{E/F_u}}{d_a/t}\right]t d_a F_u \qquad (Eq.\ E2.2\text{–}3)$$

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

$\phi = 0.50$

$P_n = 1.40\, t\, d_a\, F_u$ (*Eq.* E2.2–4)

where

ϕ = Resistance factor for welded connections

P_n = Nominal 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 = $0.7d - 1.5t$ but $\le 0.55d$ (*Eq.* E2.2–5)

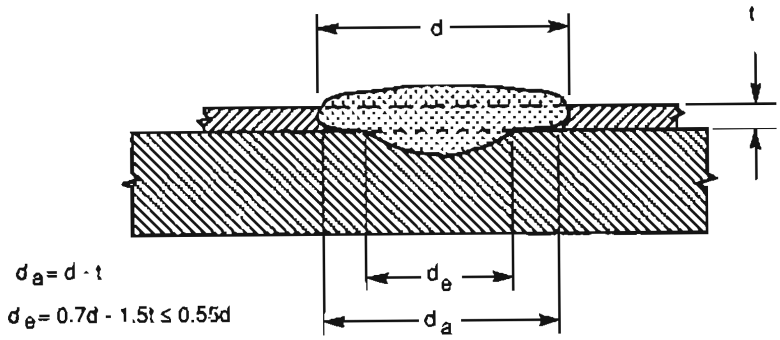
t = Total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer

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

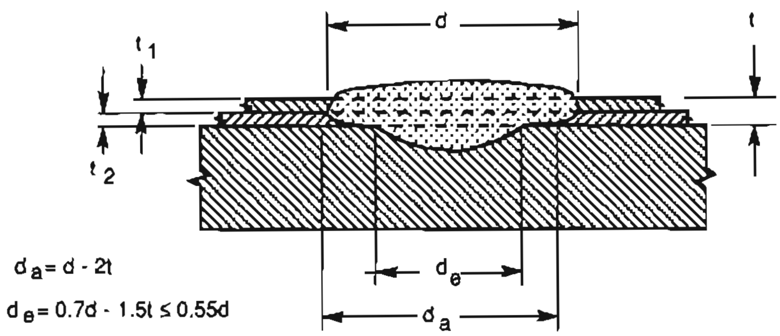
F_u = Tensile strength as specified in Section A3.1 or A3.2 or as reduced for low ductility steel.

Note: See Figures E2.2(C) and E2.2(D) for diameter definitions

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



(C) Arc Spot Weld-Single Thickness of Sheet



(D) Arc Spot Weld-Double Thickness of Sheet

Figure E2.2 C, D Arc Spot Welds

$$e_{min} = \frac{P_u}{\phi F_u t} \qquad (Eq.\ E2.2\text{–}6)$$

where

ϕ = Resistance factor for welded connections
= 0.70 when $F_u/F_{sy} \ge 1.15$
= 0.60 when $F_u/F_{sy} < 1.15$

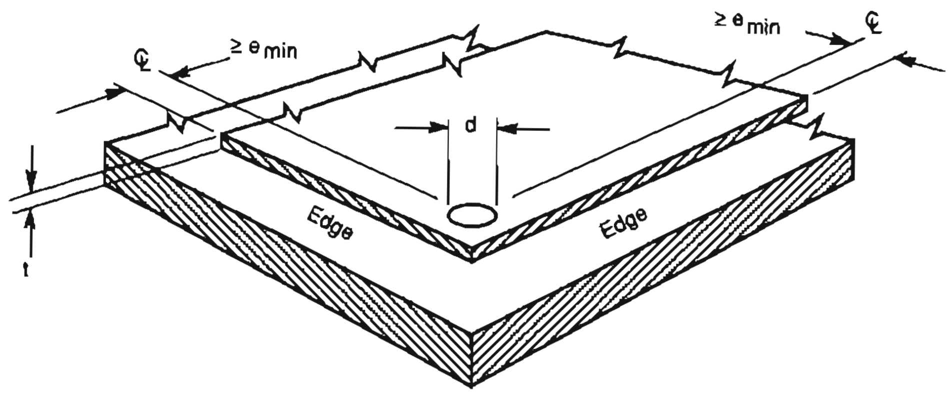
P_u = Required strength transmitted by weld

t = Thickness of thinnest connected sheet

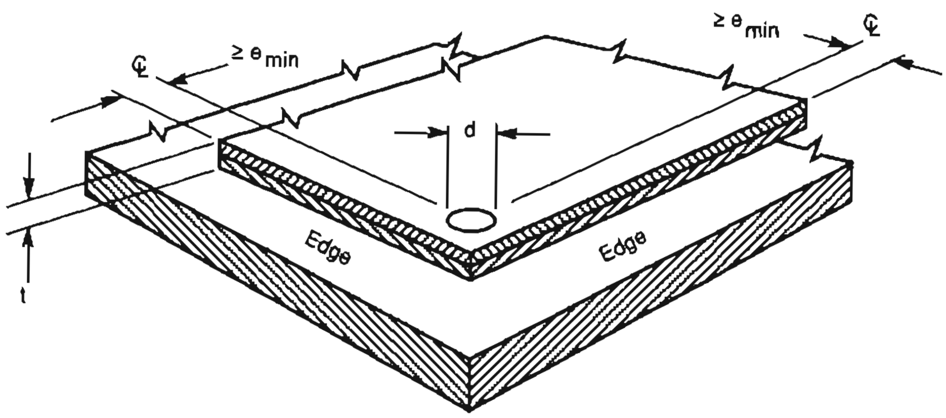
F_{sy} = Yield point as specified in Sections A3.1 or A3.2

Note: See Figures E2.2(E) and E2.2(F) for edge distances of arc welds.

In addition, the distance from the centerline of any weld to the end or boundary of the connected member shall not be less than 1.5d. In no case shall the clear distance between welds and the end of member be less than 1.0d.



(E) Single Sheet



(F) Double Sheet

Figure E2.2 E, F Edge Distances for Arc Spot Welds

The design tensile strength, ϕP_n, on each arc spot weld between sheet and supporting member, shall be determined as follows:

$\phi = 0.65$

$P_n = 0.7\, t\, d_a\, F_u$ (*Eq.* E2.2–7)

The following additional limitations for use in Eq. 2.2–7 shall apply:

$e_{min} \geq d$
$F_{xx} \geq 60$ ksi
$F_u \leq 60$ ksi
$t \geq 0.028$ in.

If it can be shown by measurement that a given weld procedure will consistently give a larger effective diameter, d_e, or average diameter, d_a, as applicable, this larger diameter may be used providing the particular welding procedure used for making those welds is followed.

E2.3 Arc Seam Welds

Arc seam welds [Figure E2.3(A)] covered by this 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 design shear strength, ϕP_n, of arc seam welds shall be determined by using the smaller of either

(a) $\phi = 0.60$

$$P_n = \left[\frac{\pi d_e^2}{4} + L d_e\right] 0.75 F_{xx} \text{ ;or} \qquad (Eq.\ E2.3\text{–}1)$$

(b) $\phi = 0.60$

$$P_n = 2.5\, t F_u (0.25L + 0.96\, d_a) \qquad (Eq.\ E2.3\text{–}2)$$

where

ϕ = Resistance factor for welded connections
P_n = Nominal 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

where

d_a = (d – t) for a single sheet, and (*Eq.* E2.3–3)
(d – 2t) for a double sheet (*Eq.* E2.3–4)

d_e = Effective width of arc seam weld at fused surfaces

d_e = 0.7d – 1.5t (*Eq.* E2.3–5)

and F_u and F_{xx} are defined in Section E2.2. The minimum edge distance shall be as determined for the arc spot weld, Section E2.2. See Figure E2.3(B).

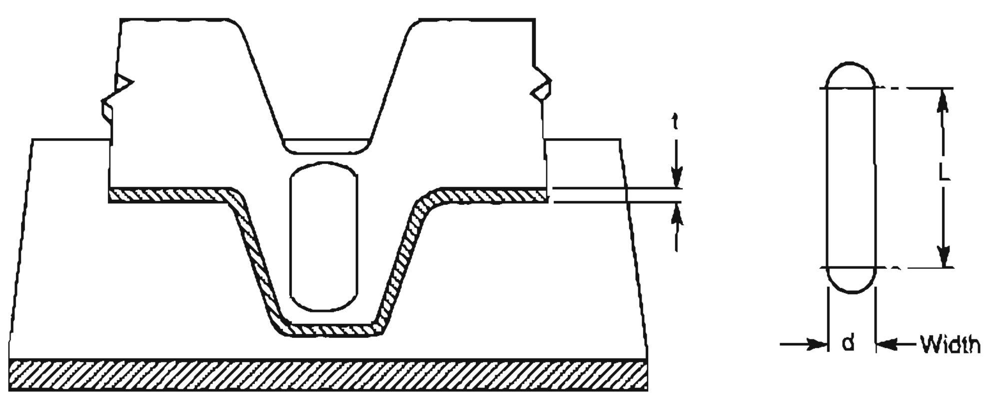


Figure E2.3A Arc Seam Welds – Sheet to Supporting Member in Flat Position

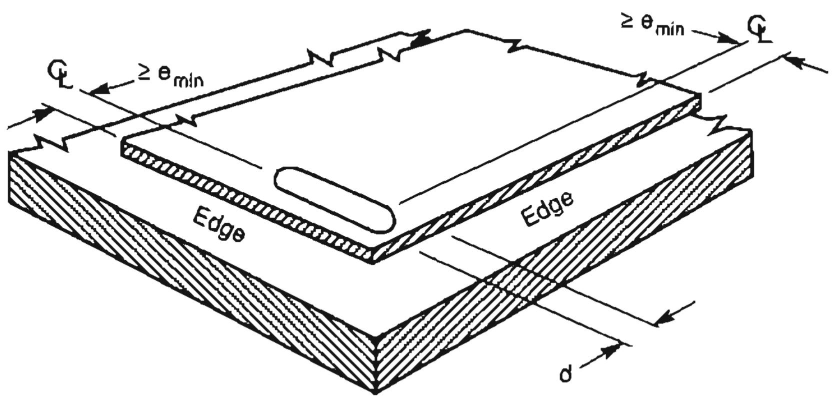


Figure E2.3B Edge Distances for Arc Seam Welds

E2.4 Fillet Welds

Fillet welds covered by this Specification apply to the welding of joints in any position, either

(a) Sheet to sheet, or
(b) Sheet to thicker steel member.

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

(a) For longitudinal loading:

For $L/t < 25$:

$\phi = 0.60$

$$P_n = \left(1 - \frac{0.01L}{t}\right) tLF_u \qquad (Eq.\ E2.4\text{–}1)$$

For $L/t \geq 25$:

$\phi = 0.55$

$$P_n = 0.75\ tLF_u \qquad (Eq.\ E2.4\text{–}2)$$

(b) For transverse loading:

$\phi = 0.60$

$$P_n = tLF_u \qquad (Eq.\ E2.4\text{–}3)$$

where t = Least value of t_1 or t_2, Figure E2.4

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

$\phi = 0.60$

$$P_n = 0.75\ t_w L F_{xx} \qquad (Eq.\ E2.4\text{–}4)$$

where

ϕ = Resistance factor for welded connedtions

P_n = Nominal strength of a fillet weld

L = Length of fillet weld

t_w = Effective throat = $0.707\ w_1$ or $0.707\ w_2$, 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 (see Figure E2.4).

F_u and F_{xx} are defined in Section E2.2.

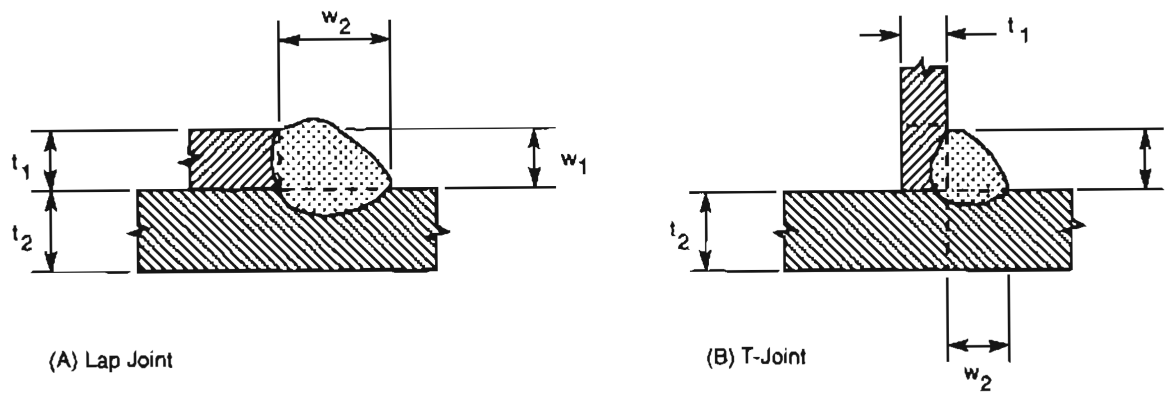


Figure E2.4 Fillet Welds

E2.5 Flare Groove Welds

Flare groove welds covered by this 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 design shear strength, ϕP_n, of a flare groove weld shall be determined as follows:

(a) For flare–bevel groove welds, transverse loading [see Figure E2.5(A)]:

$\phi = 0.55$

$P_n = 0.833tLF_u$ (*Eq.* E2.5–1)

(b) For flare groove welds, longitudinal loading [see Figures E2.5(B), E2.5(C), and E2.5(D)]:

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

$\phi = 0.55$

$P_n = 0.75tLF_u$ (*Eq.* E2.5–2)

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

$\phi = 0.55$

$P_n = 1.50tLF_u$ (*Eq.* E2.5–3)

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

$\phi = 0.60$

$P_n = 0.75t_wLF_{xx}$ (*Eq.* E2.5–4)

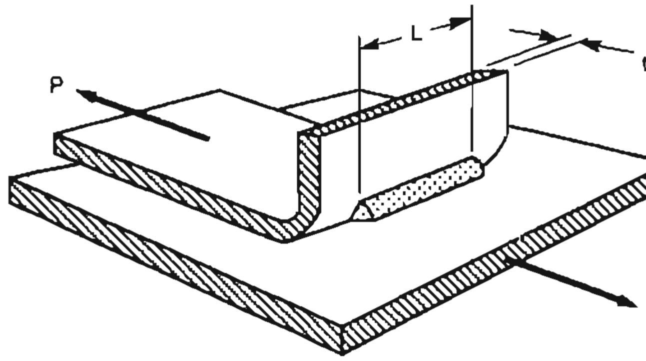


Figure E2.5A Flare–Bevel Groove Weld

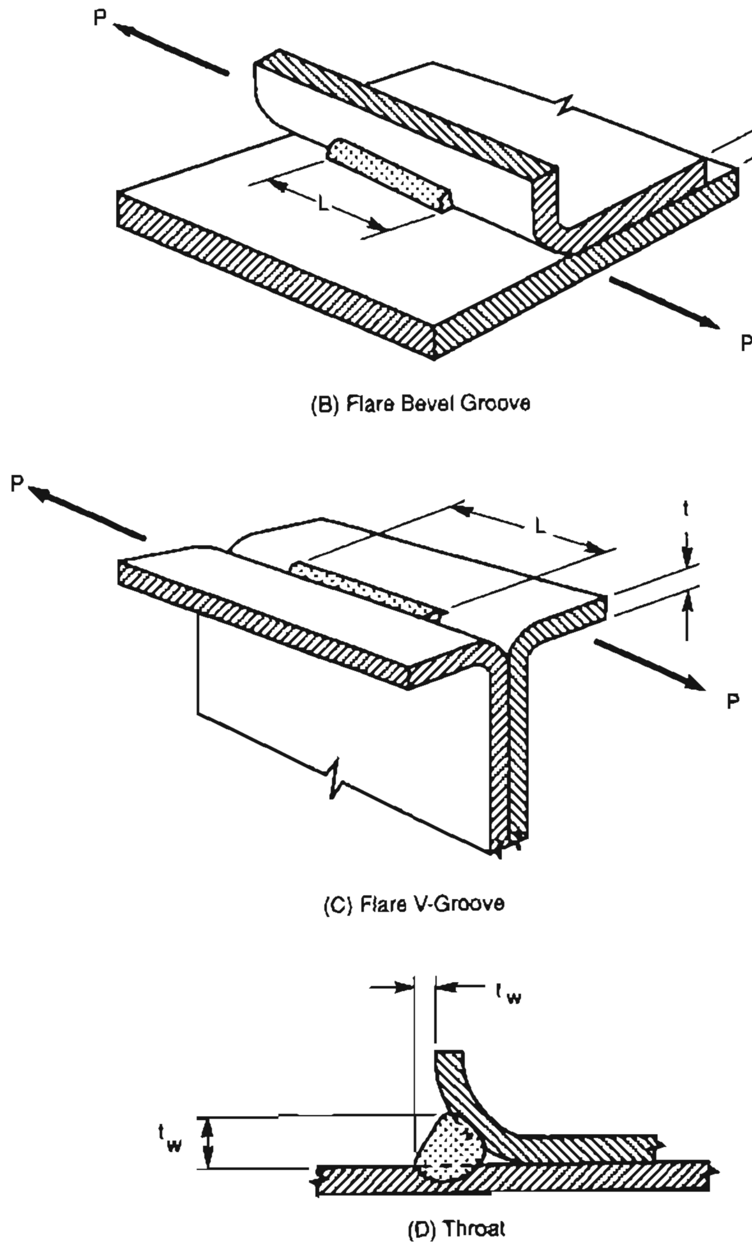


Figure E2.5 B, C, D Shear in Flare Groove Welds

E2.6 Resistance Welds

The design shear strength, ϕP_n, of spot welding shall be determined as follows:

ϕ = 0.65

P_n = Tabulated value given in Table E2.6

TABLE E2.6
Nominal Shear Strength Of Spot Welding

Thickness of Thinnest Outside Sheet, in.	Nominal Shear Strength per Spot, kips	Thickness of Thinnest Outside Sheet, in.	Nominal Shear Stength per Spot, kips
0.010	0.13	0.080	3.33
0.020	0.48	0.090	4.00
0.030	1.00	0.100	4.99
0.040	1.42	0.110	6.07
0.050	1.65	0.125	7.29
0.060	2.28	0.190	10.16
0.070	2.83	0.250	15.00

E3 Bolted Connections

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's "Load and Resistance Factor Design Specification for Structural Steel Buildings", September 1, 1986.

Bolts, nuts, and washers shall generally conform to one of the following specifications:

ASTM A194 Carbon and Alloy Steel Nuts for Bolts for High-Pressure and High-Temperature Service

ASTM A307(Type A), Carbon Steel Externally and Internally Threaded Standard Fasteners

ASTM A325 High Strength Bolts for Structural Steel Joints

ASTM A354 (Grade BD), Quenched and Tempered Alloy Steel Bolts, Studs,and Other Externally Threaded Fasteners (for diameter of bolt smaller than $^1/_2$ inch)

ASTM A449 Quenched and Tempered Steel Bolts and Studs (for diameter of bolt smaller than $^1/_2$ inch)

ASTM A490 Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints

ASTM A563 Carbon and Alloy Steel Nuts

ASTM F436 Hardened Steel Washers

ASTM F844 Washers, Steel, Plain (Flat), Unhardened for General Use

ASTM F959 Compressible Washer-Type Direct Tension Indicators for Use with Structural Fasteners

When other than the above are used, drawings shall indicate clearly the type and size of fasteners to be employed and the nominal strength assumed in design.

Bolts shall be installed and tightened to achieve satisfactory performance of the connections involved under usual service conditions.

The holes for bolts shall not exceed the sizes specified in Table E3, except that larger holes may be used in column base details or structural systems connected to concrete walls.

TABLE E3
Maximum Size of Bolt Holes, Inches

Nominal Bolt Diameter, d in.	Standard Hole Diameter, d_h in.	Oversized Hole Diameter, d_h in.	Short–Slotted Hole Dimensions in.	Long–Slotted Hole Dimensions in.
$< 1/2$	$d + 1/32$	$d + 1/16$	$(d + 1/32)$ by $(d + 1/4)$	$(d + 1/32)$ by $(2\ 1/2 d)$
$\geq 1/2$	$d + 1/16$	$d + 1/8$	$(d + 1/16)$ by $(d + 1/4)$	$(d + 1/16)$ by $(2\ 1/2 d)$

Standard holes shall be used in bolted connections, except that oversized and slotted holes may be used as approved by the designer. The length of slotted holes shall be normal to the direction of the shear load. Washers or backup plates shall be installed over oversized or short–slotted holes in an outer ply unless suitable performance is demonstrated by load tests in accordance with Section F.

E3.1 Spacing and Edge Distance

The design shear strength, ϕP_n, of the connected part along two parallel lines in the direction of applied force shall be determined as follows:

$$P_n = teF_u \qquad (Eq.\ E3.1\text{–}1)$$

(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 as specified in Section A3.1 or A3.2 or as reduced for low–ductility steel
F_{sy} = Yield point of the connected part as specified in Section A3.1 or A3.2 or as reduced for low–ductility steel

In addition, the minimum distance between centers of bolt holes shall provide sufficient clearance for bolt heads, nuts, washers and the wrench but shall not be less than 3 times the nominal bolt diameter, d. Also, the distance from the center of any standard hole to the end or other boundary of the connecting member shall not be less than $1\ 1/2$ d.

For oversized and slotted holes, the distance between edges of two adjacent holes and the distance measured from the edge of the hole to the end or other boundary of the connecting member in the line of stress shall not be less than the value of $e-(d_h/2)$, in which e is the required distance computed from the applicable equation given above, and d_h is the diameter of a standard hole defined in Table E3. In no case shall the clear distance between edges of two adjacent holes be less than 2d and the distance between the edge of the hole and the end of the member be less than d.

E3.2 Tension in Connected Part

The design 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

$$P_n = (1.0 - 0.9r + 3rd/s)\, F_u A_n \leq F_u A_n \qquad (Eq.\ E3.2\text{–}1)$$

ϕ = 0.65 for double shear connection

ϕ = 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

ϕ = 0.65

$$P_n = (1.0 - r + 2.5rd/s)\, F_u A_n \leq F_u A_n \qquad (Eq.\ E3.2\text{–}2)$$

In addition, the design tensile strength shall not exceed the following values:

ϕ = 0.95

$$P_n = F_y A_n \qquad (Eq.\ E3.2\text{–}3)$$

where

A_n = Net area of the connected part

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

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

F_u = Tensile strength of the connected part as specified in Section A3.1 or A3.2 or as reduced for low–ductility steel

F_y = Yield point of the connected part

d and t are defined in Section E3.1

E3.3 Bearing

The design bearing strength, ϕP_n, shall be determined by the values of ϕ and P_n given in Tables E3.3–1 and E3.3–2 for the applicable thickness and F_u/F_{sy} ratio of the connected part and the type of joint used in the connection.

In Tables E3.3–1 and E3.3–2, the symbols ϕ, P_n, d, F_u and t were previously defined. For conditions not shown, the design bearing strength of bolted connections shall be determined by tests.

TABLE E3.3–1
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	$3.33\ F_udt$
		< 1.15	0.65	$3.00\ F_udt$
	Single shear and outside sheets of double shear connection	No limit	0.60	$3.00\ F_udt$
≥ 3/16	See AISC LRFD Specification			

TABLE E3.3–2
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
≥ 0.036 but < 3/16	Inside sheet of double shear connection	≥ 1.15	0.65	$3.00\ F_udt$
	Single shear and outside sheets of double shear connection	≥ 1.15	0.70	$2.22\ F_udt$
≥ 3/16	See AISC LRFD Specification			

E3.4 Shear and Tension in Bolts

The required bolt strength in shear or tension shall not exceed the design strength, ϕP_n, determined as follows:

ϕ = Resistance factor given in Table E3.4–1

$$P_n = A_bF_n \qquad (Eq.\ E3.4\text{–}1)$$

where

A_b = Gross cross–sectional area of bolt

F_n is given by F_{nv} or F_{nt} in Table E3.4–1.

TABLE E3.4–1
Nominal Tensile and Shear Strength 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 BD Bolts ($^1/_4$ in. $\leq d <$ $^1/_2$ in.), when threads are not exluded from shear planes		101.0		59.0
A354 Grade BD Bolts ($^1/_4$ in. $\leq d <$ $^1/_2$ in.) when threads are excluded from shear planes		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

* Applies to bolts in holes as limited by Table E3. Washers or back–up plates shall be installed over long–slotted holes and the capacity of connections using long–slotted holes shall be determined by load tests in accordance with Section F.

The pullover strength of the connected sheet at the bolt head, nut or washer should be considered where bolt tension is involved, see Section E5.2.

When bolts are subject to a combination of shear and tension produced by factored loads, the required tension strength shall not exceed the design strength, ϕP_n, based on $\phi = 0.75$ and $P_n = A_b F'_{nt}$, where F'_{nt} is given in Table E3.4–2, in which f_v is the shear stress produced by the same factored loads. The required shear strength shall not exceed the design shear strength, $\phi A_b F_{nv}$, determined in accordance with Table E3.4–1.

TABLE E3.4–2
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.	$52 - 2.4f_v \leq 45$	

E4 Shear Rupture

At beam–end connections, where one or more flanges are coped and failure might occur along a plane through the fasteners, the required shear strength shall not exceed the design shear strength, ϕV_n.

where

ϕ = 0.75

$V_n = 0.6\, F_u A_{wn}$ (*Eq.* E4–1)

$A_{wn} = (d_{wc} - nd_h)t$ (*Eq.* E4–2)

d_{wc} = Coped web depth

n = Number of holes in the critical plane

d_h = Hole diameter

F_u = Tensile strength as specified in Section A3.1 or A3.2 or as reduced for low–ductility steel

t = Thickness of coped web

E5 Connections to Other Materials

E5.1 Bearing

Proper provisions shall be made to transfer bearing forces resulting from axial loads and moments from steel components covered by the Specification to adjacent structural components made of other materials. The required bearing strength in the contact area shall not exceed the design strength, $\phi_c P_p$.

In the absence of code regulations, the design bearing strength on concrete may be taken as $\phi_c P_p$:

On the full area of a concrete support . . . . $P_p = 0.85 f'_c A_1$
On less than the full area of a concrete support . . . . $P_p = 0.85 f'_c A_1 \sqrt{A_2 / A_1}$
where
ϕ_c = 0.60
f'_c = Specified compression strength of concrete
A_1 = Bearing area
A_2 = Full cross–sectional area of concrete support
The value of $\sqrt{A_2 / A_1}$ shall not exceed 2.

E5.2 Tension

The pull–over shear/tension forces in the steel sheet around the head of the fastener should be considered as well as the pull–out force resulting from factored axial loads and bending moments transmitted onto the fastener from various adjacent structural components in the assembly.

The nominal tensile strength of the fastener and the nominal imbedment strength of the adjacent structural component shall be determined by applicable product code approvals, or product specifications and/or product literature.

E5.3 Shear

Proper provisions shall be made to transfer shearing forces from steel components covered by this Specification to adjacent structural components made of other materials. The required shear and/or bearing strength on the steel components shall not exceed that allowed by this Specification. The design shear strength on the fasteners and other material shall not be exceeded. Imbedment requirements are to be met. Proper provision shall also be made for shearing forces in combination with other forces.

F. TESTS FOR SPECIAL CASES

(a) Tests shall be made by an independent testing laboratory or by a testing laboratory of a manufacturer.

(b) The provisions of Chapter F do not apply to cold–formed steel diaphragms.

F1 Tests for Determining Structural Performance

Where the composition or configuration of elements, assemblies, connections, or details of cold–formed steel structural members are such that calculation of their load–carrying capacity or deflection cannot be made in accordance with the provisions of this Specification, their structural performance shall be established from tests and evaluated in accordance with the following procedure.

(a) Where practicable, evaluation of the test results shall be made on the basis of the average value of test data resulting from tests of not fewer than four identical specimens, provided the deviation of any individual test result from the average value obtained from all tests does not exceed ±10 percent. If such deviation from the average value exceeds 10 percent, at least three more tests of the same kind shall be made. The average value of all tests made shall then be regarded as the predicted capacity, R_p, for the series of the tests. The mean value and the coefficient of variation of the tested–to–predicted load ratios for all tests, P_m and V_p, shall be determined for statistical analysis.

(b) The load–carrying capacity of the tested elements, assemblies, connections, or members shall satisfy Eq. F1–1.

$$\phi R_P \geq \Sigma \gamma_i Q_i \qquad (Eq.\ F1\text{–}1)$$

where

$\Sigma \gamma_i Q_i$ = Required resistance based on the most critical load combination determined in accordance with Section A5.1.4. γ_i and Q_i are load factors and load effects, respectively.

R_P = Average value of all test results

ϕ = Resistance factor

$$= 1.5(M_m F_m P_m)\exp\left(-\beta_o\sqrt{V_M^2 + V_F^2 + C_P V_P^2 + V_Q^2}\right)^* \qquad (Eq.\ F1\text{–}2)$$

M_m = Mean value of the material factor listed in Table F1 for the type of component involved

F_m = Mean value of the fabrication factor listed in Table F1 for the type of component involved

P_m = Mean value of the tested–to–predicted load ratios determined in Section F1(a)

β_o = Target reliability index
= 2.5 for structural members and 3.5 for connections

V_M = Coefficient of variation of the material factor listed in Table F1 for the type of component involved

V_F = Coefficient of variation of the fabrication factor listed in Table F1 for the type of component involved

C_P = Correction factor

$$= (n-1)/(n-3) \qquad (Eq.\ F1\text{–}3)$$

V_P = Coefficient of variation of the tested–to–predicted load ratios determined in Section F1(a)

n = Number of tests

V_Q = Coefficient of variation of the load effect
= 0.21

* For beams having tension flange through–fastened to deck or sheathing and with compression flange laterally unbraced, ϕ shall be determined with a coefficient of 1.6 in lieu of 1.5, β_o = 1.5, and V_Q = 0.43.

TABLE F1
Statistical Data for the Determination of Resistance Factor

Type of Component	M_m	V_M	F_m	V_F
Transverse Stiffeners	1.10	0.10	1.00	0.05
Shear Stiffeners	1.00	0.06	1.00	0.05
Tension Members	1.10	0.10	1.00	0.05
Flexural Members				
Bending Strength	1.10	0.10	1.00	0.05
Lateral Buckling Strength	1.00	0.06	1.00	0.05
One Flange Through–Fastened to Deck or Sheathing	1.10	0.10	1.00	0.05
Shear Strength	1.10	0.10	1.00	0.05
Combined Bending and Shear	1.10	0.10	1.00	0.05
Web Crippling Strength	1.10	0.10	1.00	0.05
Combined Bending and Web Crippling	1.10	0.10	1.00	0.05
Concentrically Loaded Compression Members	1.10	0.10	1.00	0.05
Combined Axial Load and Bending	1.05	0.10	1.00	0.05
Cylindrical Tubular Members				
Bending Strength	1.10	0.10	1.00	0.05
Axial Compression	1.10	0.10	1.00	0.05
Wall Studs and Wall Stud Assemblies				
Wall Studs in Compression	1.10	0.10	1.00	0.05
Wall Studs in Bending	1.10	0.10	1.00	0.05
Wall Studs with Combined Axial Load and Bending	1.05	0.10	1.00	0.05

TABLE F1 (Continued)
Statistical Data for the Determination of Resistance Factor

Type of Component	M_m	V_M	F_m	V_F
Welded Connections				
Arc Spot Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Failure	1.10	0.08	1.00	0.15
Arc Seam Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Tearing	1.10	0.10	1.00	0.10
Fillet Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Failure	1.10	0.08	1.00	0.15
Flare Groove Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Failure	1.10	0.10	1.00	0.10
Resistance Welds	1.10	0.10	1.00	0.10
Bolted Connections				
Minimum Spacing and Edge Distance	1.10	0.08	1.00	0.05
Tension Strength on Net Section	1.10	0.08	1.00	0.05
Bearing Strength	1.10	0.08	1.00	0.05

The listing in Table F1 does not exclude the use of other documented statistical data if they are established from sufficient results on material properties and fabrication.*

For steels not listed in Section A3.1, the values of M_m and V_M shall be determined by the statistical analysis for the materials used.

When distortions interfere with the proper functioning of the specimen in actual use, the load effects based on the critical load combination at the occurrence of the acceptable distortion shall also satisfy Eq. F1–1, except that the resistance factor ϕ is taken as unity and that the load factor for dead load may be taken as 1.0.

(c) If the yield point of the steel from which the tested sections are formed is larger than the specified value, the test results shall be adjusted down to the specified minimum yield point of the steel which the manufacturer intends to use. The test results shall not be adjusted upward if the yield point of the test specimen is less than the minimum specified yield point. Similar adjustments shall be made on the basis of tensile strength instead of yield point where tensile strength is the critical factor.

Consideration must also be given to any variation or differences which may exist between the design thickness and the thickness of the specimens used in the tests.

F2 Tests for Confirming Structural Performance

For structural members, connections, and assemblies whose capacities can be computed according to this Specification or its specific references, confirmatory tests may be made to demonstrate the load–carrying capacity not less than the nominal resistance, R_n, specified in this Specification or its specific references for the type of behavior involved.

F3 Tests for Determining Mechanical Properties

F3.1 Full Section

Tests for determination of mechanical properties of full sections to be used in Section A5.2.2 shall be made as specified below:

(a) Tensile testing procedures shall agree with Standard Methods and Definitions for Mechanical Testing of Steel Products, ASTM A370. Compressive yield point determinations shall be made by means of compression tests of short specimens of the section.

(b) The compressive yield stress shall be taken as the smaller value of either the maximum compressive strength of the sections divided by the cross section area or the stress defined by one of the following methods:

(1) For sharp yielding steel, the yield point shall be determined by the autographic diagram method or by the total strain under load method.

(2) For gradual yielding steel, the yield point shall be determined by the strain under load method or by the 0.2 percent offset method.

When the total strain under load method is used, there shall be evidence that the yield point so determined agrees within 5 percent with the yield point which would be determined by the 0.2 percent offset method

(c) Where the principal effect of the loading to which the member will be subjected in service will be to produce bending stresses, the yield point shall be determined for the flanges only. In determining such yield points, each specimen shall consist of one complete flange plus a portion of the web of such flat width ratio that the value of ρ for the specimen is unity.

* See Reference 36 of the Commentary

(d) For acceptance and control purposes, two full section tests shall be made from each lot of not more than 50 tons nor less than 30 tons of each section, or one test from each lot of less than 30 tons of each section. For this purpose a lot may be defined as that tonnage of one section that is formed in a single production run of material from one heat.
(e) At the option of the manufacturer, either tension or compression tests may be used for routine acceptance and control purposes, provided the manufacturer demonstrates that such tests reliably indicate the yield point of the section when subjected to the kind of stress under which the member is to be used.

F3.2 Flat Elements of Formed Sections

Tests for determining mechanical properties of flat elements of formed sections and representative mechanical properties of virgin steel to be used in Section A5.2.2 shall be made in accordance with the following provisions:

The yield point of flats, F_{yf}, shall be established by means of a weighted average of the yield points of standard tensile coupons taken longitudinally from the flat portions of a representative cold–formed member. The weighted average shall be the sum of the products of the average yield point for each flat portion times its cross sectional area, divided by the total area of flats in the cross section. The exact number of such coupons will depend on the shape of the member, i.e., on the number of flats in the cross section. At least one tensile coupon shall be taken from the middle of each flat. If the actual virgin yield point exceeds the specified minimum yield point, the yield point of the flats, F_{yf}, shall be adjusted by multiplying the test values by the ratio of the specified minimum yield point to the actual virgin yield point.

F3.3 Virgin Steel

The following provisions apply to steel produced to other than the ASTM Specifications listed in Section A3.1 when used in sections for which the increased yield point of the steel after cold forming shall be computed from the virgin steel properties according to Section A5.2.2. For acceptance and control purposes, at least four tensile specimens shall be taken from each lot as defined in Section F3.1(d) for the establishment of the representative values of the virgin tensile yield point and ultimate strength. Specimens shall be taken longitudinally from the quarter points of the width near the outer end of the coil.

American Iron and Steel Institute
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SPECIFICATION

LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD–FORMED STEEL STRUCTURAL MEMBERS

MARCH 16, 1991 EDITION

LRFD Cold-Formed Steel Design Manual - Part I

AMERICAN IRON AND STEEL INSTITUTE
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PREFACE

The American Iron and Steel Institute allowable stress design specification has long been used for the design of cold–formed steel structural members. The Load and Resistance Factor Design (LRFD) Specification has recently been developed from a research project sponsored by AISI at the University of Missouri–Rolla under the direction of Wei–Wen Yu with consultation of T.V. Galambos and initial contribution of M.K. Ravindra. In this LRFD Specification, separate load and resistance factors are applied to specified loads and nominal resistance to ensure that the probability of reaching a limit state is acceptably small. These factors reflect the uncertainties of analysis, design, loading, material properties and fabrication. They are derived on the basis of the first order probabilistic methodology as used for the development of the AISC Load and Resistance Factor Design Specification for Structural Steel Buildings.

This Specification contains six chapters of the LRFD recommendations for cold–formed steel structural members and connections. The background information for the design criteria is discussed in the Commentary and other related references.

AISI acknowledges the devoted efforts of the members of the Committee on Specifications for the Design of Cold–Formed Steel Structural Members. This group, comprised of consulting engineers, researchers, designers from companies manufacturing cold–formed steel members, components, assemblies, and complete structures, and specialists from the steel producing industry, has met two to three times per year since its establishment in 1973. Its current members, who have made extensive contributions of time and effort in developing and reaching consensus on this LRFD Specification are:

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The activities of the Committee are sponsored by AISI's Light Construction Subcommittee of the Construction Marketing Committee. The Specification is issued under the auspices of AISI's Committee on Construction Codes and Standards.

Users of the Specification are invited to continue to offer their valuable comments and suggestions. The cooperation of all involved, the users as well as the writers, is needed to continue to keep the Specification up to date and a useful tool for the designer.

American Iron and Steel Institute
March 16, 1991

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SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
A	Full unreduced cross–sectional area of the member	C3.1.1, C3.1.2, C4, C6.2, D4.1
A_b	$b_1t + A_s$, for transverse stiffeners at interior support and under concentrated load, and $b_2t + A_s$, for transverse stiffeners at end support	B6.1
A_b	Gross cross–sectional area of bolt	E3.4
A_c	$18t^2 + A_s$, for transverse stiffeners at interior support and under concentrated load, and $10t^2 + A_s$, for transverse stiffeners at end support	B6.1
A_e	Effective area at the stress F_n	C4, C6.2, D4.1
A_n	Net area of cross section	C2, E3.2
A_s	Cross–sectional area of transverse stiffeners	B4, B4.1, B4.2, B6.1
A'_s	Effective area of stiffener	B4, B4.1, B4.2
A_{st}	Gross area of shear stiffener	B6.2
A_{wn}	Net web area	E4
A_1	Bearing area	E5.1
A_2	Full cross sectional area of concrete support	E5.1
a	Shear panel length of the unreinforced web element. For a reinforced web element, the distance between transverse stiffeners	B6.2, C3.2
a	Length of bracing interval	D3.2
B	Stud spacing	D4, D4.1
B_c	Term for determining the tensile yield point of corners	A5.2.2
b	Effective design width of compression element	B2.1, B2.2, B2.3, B3.1, B3.2, B4.1, B4.2, B5
b	Flange width, Z–section	D3.2.1
b_d	Effective width for deflection calculation	B2.1, B2.2
b_e	Effective design width of sub–element or element	A1.2, B2.3, B5
b_o	See Figure B4.1	B4, B4.1, B5
C	For flexural members, ratio of the total corner cross –sectional area of the controlling flange to the full cross –sectional area of the controlling flange	A5.2.2
C_b	Bending coefficient dependent on moment gradient	C3.1.2
C_m	End moment coefficient in interaction formula	C5

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
C_{ms}	Coefficient for lateral bracing of Z–section	D3.2.1
C_{mx}	End moment coefficient in interaction formula	C5
C_{my}	End moment coefficient in interaction formula	C5
C_p	Correction Factor	F1
C_s	Coefficient for lateral torsional buckling	C3.1.2
C_{TF}	End moment coefficient in interaction formula	C3.1.2
C_{th}	Coefficient for lateral bracing of Z–sections	D3.2.1
C_{tr}	Coefficient for lateral bracing of Z–sections	D3.2.1
C_v	Shear stiffener coefficient	B6.2
C_w	Torsional warping constant of the cross–section	C3.1.2
C_y	Compression strain factor	C3.1.1
C_o	Initial column imperfection	D4.1
C_1	Term used to compute shear strain in wall board	B4, B4.1, D4.2
C_2	Coefficient as defined in Figure B4–2	B4, B4.2
c_f	Amount of curling	B1.1b
D	Outside diameter of cylindrical tube	C6, C6.1, C6.2, D4.2
D	Overall depth of lip	B1.1, B4, D1.1
D	Shear stiffener coefficient	B6.2
D	Nominal dead load	A5.1.4
D_o	Initial column imperfection	D4.1
d	Depth of section	B1.1b, B4, C3.1.1, C3.1.3, D1.1, D3.2.1, D4, D4.1
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d	Visible diameter of outer surface of arc spot weld	E2.2
d	Diameter of bolt	E3, E3.1, E3.2, E3.4
d_a	Average diameter of the arc spot weld at mid–thickness of t	E2.2
d_a	Average width of seam weld	E2.3
d_e	Effective diameter of fused area	E2.2
d_e	Effective width of arc seam weld at fused surfaces	E2, E2.3
d_h	Diameter of standard hole	B2.2, E3.1, E4
d_s	Reduced effective width of stiffener	B4, B4.2
d'_s	Actual effective width of stiffener	B4, B4.2
d_{wc}	Coped web depth	E4
E	Modulus of elasticity of steel (29.5×10^3 ksi)	B1.1b, B2.1, B6.1, C3.1.1, C3.2, C3.5, C4, C4.1, C5, C6.1, D1.2, D4.1, D4.2 , E2.2

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
E	Nominal earthquake load	A5.1.4
E_o	Initial column imperfection; a measure of the initial twist of the stud from the initial, ideal, unbuckled location	D4.1
E_1	Term used to compute shear strain in wallboard	D4.1
E'	Inelastic modulus of elasticity	D4.1
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 toward which the force is directed	E3.1
e_{min}	Minimum allowable distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed	E2.2
e_y	Yield strain = F_y/E	C3.1.1
F	Loads due to fluids	A5.1.4
F_e	Elastic buckling stress	C4, C4.1, C4.2, C4.3, C6.2, D4.1
F_m	Mean value of the fabrication factor	F1
F_n	Nominal buckling stress	C4, C6.2, D4.1
F_{nt}	Nominal tensile strength of bolts	E3.4
F_{nv}	Nominal shear strength of bolts	E3.4
F'_{nt}	Nominal tensile strength for bolts subject to combination of shear and tension	E3.4
F_{sy}	Yield point as specified in Sections A3.1 or A3.2	A3.1, A3.2, A3.3.2, E2.2, E3.1, E3.2
F_u	Tensile strength as specified in Sections A3.1 or A3.2, or as reduced for low ductility steel	A3.1, A3.2, A3.3, A3.3.2, E2.2, E2.3, E2.4,E2.5, E3.1, E3.2, E3.3, E4
F_{uv}	Tensile strength of virgin steel specified by Section A3 or established in accordance with Section F3.3	A3, A5.2.2, E2.2, F3.3
F_{wy}	Yield point for design of transverse stiffeners	B6.1
F_{xx}	Strength level designation in AWS electrode classification	E2.2, E2.3, E2.4, E2.5

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
F_y	Yield point used for design, not to exceed the specified yield point or established in accordance with Section F3, or as increased for cold work of forming in Section A5.2.2 or as reduced for low ductility steels in Section A3.3.2	A1.2, A3.3, A5.2.1, A5.2.2, B2.1, B5, B6.1, C2, C3.1, C3.1.1, C3.1.3, C3.2, C3.5, C4, C6.1, C6.2, D1.2, D4, D4.2, E2
F_{ya}	Average yield point of section	A5.2.2
F_{yc}	Tensile yield point of corners	A5.2.2
F_{yf}	Weighted average tensile yield point of the flat portions	A5.2.2, F3.2
F_{ys}	Yield point of stiffener steel	B6.1
F_{yv}	Tensile yield point of virgin steel specified by Section A3 or established in accordance with Section F3.3	A3, A5.2.2, F3.3
f	Stress in the compression element computed on the basis of the effective design width	B2.1, B2.2, B3.2, B4, B4.1
f_{av}	Average computed stress in the full, unreduced flange width	B1.1b
f_c	Stress at service load in the cover plate or sheet	D1.2
f'_c	Specified compression stress of concrete	E5.1
f_d	Computed compressive stress in the element being considered. Calculations are based on the effective section at the load for which deflections are determined.	B2.1, B2.2, B3.1, B4.1, B4.2
f_{d1}, f_{d2}	Computed stresses f_1 and f_2 as shown in Figure B2.3–1. Calculations are based on the effective section at the load for which deflections are deteremined	B2.3
f_{d3}	Computed stress f_3 in edge stiffener, as shown in Figure B4–2. Calculations are based on the effective section at the load for which deflections are determined	B3.2
f_v	Computed shear stress on a bolt	E4
f_1, f_2	Web stresses defined by Figure B2.3–1	B2.3
f_3	Edge stiffener stress defined by Figure B4–2	B3.2
G	Shear modulus of steel (11,300 ksi)	C3.1.1, D4.1
G'	Inelastic shear modulus	D4.1
g	Vertical distance between two rows of connections nearest to the top and bottom flanges	D1.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
H	Loads due to the weight and lateral pressure of soil and water in soil	A5.1.4
h	Depth of flat portion of web measured along the plane of web	B1.2, B6.2, C3.2, C3.4
I_a	Adequate moment of inertia of stiffener so that each component element will behave as a stiffened element	B1.1, B4, B4.1, B4.2
I_b	Moment of inertia of the full unreduced section about the bending axis	C5
I_s	Actual moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened	B1.1, B4, B4.1, B4.2, B5
I_{sf}	Moment of inertia of the full area of the multiple stiffened element, including the intermediate stiffeners, about its own centroidal axis parallel to the element to be stiffened	B5
I_x, I_y	Moment of inertia of full section about principal axis	D1.1, D3.2.2
I_{xy}	Product of inertia of full section about major and minor centroidal axes	D3.2.2, D4.1
I_{yc}	Moment of inertia of the compression portion of a section about the centroidal axis of the entire section parallel to the web, using the full unreduced section	C3.1.2
J	St. Venant torsion constant	C3.1.2
j	Section property for torsional–flexural buckling	C3.1.2
K	Effective length factor	C3.1.2, C4, C4.1, C5
K'	A constant	D3.2.2
K_b	Effective length factor in the plane of bending	C5
K_t	Effective length factor for torsion	C3.1.2
K_x	Effective length factor for bending about x–axis	C3.1.2
K_y	Effective length factor for bending about y–axis	C3.1.2
k	Plate buckling coefficient	B2.1, B2.3, B3.1, B3.2, B4 B4.1, B4.2
k_v	Shear buckling coefficient	B6.2, C3.2
L	Full span for simple beams, distance between inflection points for continuous beams, twice the length of cantilever beams	B1.1c, D3.2.1

SYMBOLS AND DEFINITIONS

Symbols	Definition	Section
L	Length of seam weld not including the circular ends	E2.3
L	Length of fillet weld	E2.4, E2.5
L	Unbraced length of member	C3.1.2, C4.1, D1.1, D4, D4.1
L	Nominal live load	A5.1.4
L_r	Nominal roof live load	A5.1.4
L_{st}	Length of transverse stiffener	B6.1
L_t	Unbraced length of compression member for torsion	C3.1.2
L_x	Unbraced length of compression member for bending about x–axis	C3.1.2
L_y	Unbraced length of compression member for bending about y–axis	C3.1.2
M_c	Critical moment	C3.1.2
M_e	Elastic critical moment	C3.1.2
M_m	Mean value of the material factor	F1
M_n	Nominal flexural strength	C3.1, C3.1.1, C3.1.2, C3.1.3, C6.1
M_{nx},M_{ny}	Nominal flexural strengths about the centroidal axes determined in accordance with Section C3	C5
M_{nxo}, M_{nyo}	Nominal flexural strengths about the centroidal axes determined in accordance with Section C3.1 excluding the provisions of Section C3.1.2	C3.3, C3.5, D4.2, D4.3
M_u	Required flexural strength	C3.3, C3.5
M_{ux}	Required flexural strength about x–axis	C5
M_{uy}	Required flexural strength about y–axis	C5
M_y	Moment causing a maximum strain e_y	B2.1, C3.1
M_1	Smaller end moment	C3.1.2, C5
M_2	Larger end moment	C3.1.2, C5
m	Distance from the shear center of one channel to the mid–plane of its web	D1.1, D3.2.2
N	Actual length of bearing	D3.6
n	Number of holes	E4
n	Number of tests	F1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
n_p	Number of parallel purlin lines	D3.2.1
P	Loads, forces, and effects due to ponding	A5.1.4
P_E	$\pi^2 EI_b/(K_b L_b)^2$	C5
P_L	Force to be resisted by intermediate beam brace	D3.2.1
P_m	Mean value of the tested–to–predicted load ratios	F1
P_n	Nominal axial strength of member	C4, C6.2
P_n	Nominal strength of connection component	E2, E2.2, E2.3, E2.4, E2.5
P_{no}	Nominal axial strength of member determined in accordance with Section C4 for L = 0	C5
P_p	Nominal bearing capacity on concrete	E5.1
P_s	Concentrated load or reaction based on factored loads	D1.1
P_u	Required axial strength	C5
$\overline{Q}$	Design shear rigidity for sheathing on both sides of the wall assembly	D4.1
Q_i	Load effect	F1
q	Uniformly distributed factored load in the plane of the web	D1.1
$\overline{q}$	Design shear rigidity for sheathing per inch of stud spacing	D4.1
$\overline{q}_o$	Factor used to determine design shear rigidity	D4.1
R	Reduction Factor	C3.1.3
R	Coefficient	C4, C6.2
R	Inside bend radius	A5.2.2, C3.4
R_n	Nominal resistance	A1.2, F1
R_P	Average value of all test results	F1
R_r	Nominal roof rain load	A5.1.4
r	Radius of gyration of full unreduced cross section	C3.1.1, C4, C4.1
r	Force transmitted by the bolt or bolts at the section considered, divided by the tension force in the member at that section	E3.2
r_{cy}	Radius of gyration of one channel about its centroidal axis parallel to web	D1.1
r_I	Radius of gyration of I–section about the axis perp–pendicular to the direction in which buckling would occur for the given conditions of end support and intermediate bracing	D1.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
r_o	Polar radius of gyration of cross section about the shear center	C3.1.1, C4.2, D4.1
r_x, r_y	Radius of gyration of cross section about centroidal principal axis	C3.1.1
S	$1.28\sqrt{E/f}$	B4, B4.1
S	Nominal snow load	A5.1.4
S_c	Elastic section modulus of the effective section calculated at a stress M_c/S_f in the extreme compression fiber	C3.1.1, C3.1.2, C4
S_e	Elastic section modulus of the effective section calculated with extreme compression or tension fiber at F_y	C3.1.1, C3.1.3
S_f	Elastic section modulus of full, unreduced section for the extreme compression fiber	C3.1.1, C3.1.2, C6.1
s_{max}	Maximum permissible longitudinal spacing of welds or other connectors joining two channels to form an I–section	D1.1
s	Fastener spacing	D1.2, D4.1
s	Spacing in line of stress of welds, rivets, or bolts connecting a compression coverplate or sheet to a non–integral stiffener or other element	E3.2
s	Weld spacing	D1.1
T_n	Nominal tensile strength	C2
T_s	Design strength of connection in tension	D1.1
t	Base steel thickness of any element or section	A1.2, A3.4, A5.2.1, B1.1, B1.1b, B1.2, B2.1, B4, B4.1, B4.2, B5, B6.1, C3.1.1., C3.2, C3.4, C3.5, C4, C6.1, C6.2, D1.2, D4, E2.4, E2.5
t	Total thickness of the two welded sheets	E2.2
t	Thickness of thinnest connected part	E2.2, E3.1, E4
t_s	Equivalent thickness of a multiple–stiffened element	B5, B6.1
t_w	Effective throat of weld	E2.4, E2.5
V_F	Coefficient of variation of the fabrication factor	F1
V_M	Coefficient of variation of the material factor	F1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
V_n	Nominal shear strength	B6.2, C3.2, C3.3
V_P	Coefficient of variation of the tested–to–predicted load ratios	F1
V_Q	Coefficient of variation of the load effect	F1
V_u	Required shear strength	C3.3
W	Factored load supported by all purlin lines being restrained	D3.2.1
W	Nominal wind load	A5.1.4
w	Flat width of element exclusive of radii	A1.2, B1.1, B2.1,B2.2, B3.1, B4, B4.1, B4.2, B5, C3.1.1, C4, D1.2
w	Flat width of the beam flange which contacts the bearing plate	C3.5
w_f	Width of flange projection beyond the web or half the distance between webs for box– or U–type sections	B1.1c
w_f	Projection of flanges from inside face of web	B1.1b, D1.1
w_1	Leg on weld	E2.4
w_2	Leg on weld	E2.4
x	Distance from concentrated load to brace	D3.2
x_o	Distance from shear center to centroid along the principal x–axis	C3.1.1, C4.2, D4.1
Y	Yield point of web steel divided by yield point of stiffener steel	B6.2
$1/\alpha_{nx}$, $1/\alpha_{ny}$	Magnification factors	C5
β	Coefficient	C4.2, D4.1
β_o	Target reliability index	F1
γ	Actual shear strain in the sheathing	D4.1
$\bar{\gamma}$	Permissible shear strain of the sheathing	D4.1
γ_i	Load factor	F1
θ	Angle between web and bearing surface >45° but no more than 90°	C3.4
θ	Angle between the vertical and the plane of the web of the Z–section, degrees	D3.2.1
σ	Stress related to shear strain in sheathing	D4.1
σ_{CR}	Theoretical elastic buckling stress	D4.1

SYMBOLS AND DEFINITIONS

Symbol	Definition	Section
σ_{ex}	$(\pi^2E)/(K_xL_x/r_x)^2$	C3.1.2, C4.2
	$(\pi^2E)/(L/r_x)^2$	D4.1
σ_{exy}	$(\pi^2EI_{xy})/(AL^2)$	D4.1
σ_{ey}	$(\pi^2E)/(K_yL_y/r_y)^2$	C3.1.2
	$(\pi^2E)/(L/r_y)^2$	D4.1
σ_t	Torsional buckling stress	C3.1.1, C4.2, D4.1
ρ	Reduction factor	B2.1
λ, λ_c	Slenderness factors	B2.1, C3.5
ψ	f_2/f_1	B2.3
ϕ	Resistance factor	A5.1.5, E2, E2.1, E2.2, E2.3, E2.4, E2.5, E2.6, E3.1, E3.2, E3.3, E3.4, E4, F1
ϕ_b	Resistance factor for bending strength	A5.1.5, C3, C3.1.1, C3.1.2, C3.1.3, C3.3, C3.5, C5, C6.1, C6.3, D4.2, D4.3
ϕ_c	Resistance factor for concentrically loaded compression member	A3.3.1, A5.1.5, B6.1, C4, C5, C6.2, C6.3, D4.1, D4.3
ϕ_c	Resistance factor for bearing strength	E5.1
ϕ_t	Resistance factor for tension member	C2
ϕ_v	Resistance factor for shear strength	B6.2, C3.2, C3.3
ϕ_w	Resistance factor for web crippling strength	C3.4, C3.5

LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD–FORMED STEEL STRUCTURAL MEMBERS

A. GENERAL PROVISIONS

A1 Limits of Applicability and Terms

A1.1 Scope and Limits of Applicability

This Load and Resistance Factor Design Specification is an alternate to the Specification for the Design of Cold–Formed Steel Structural Members of the American Iron and Steel Institute.

This Specification shall apply to the design of structural 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. It may also be used for structures other than buildings provided appropriate allowances are made for thermal and/or dynamic effects.

A1.2 Terms

Where the following terms appear in this Specification they shall have the meaning herein indicated:

(a) *Stiffened or Partially Stiffened Compression Elements.* A stiffened or partially stiffened compression element is a flat compression element (i.e., a plane compression flange of a flexural member or a plane web or flange of a compression member) of which both edges parallel to the direction of stress are stiffened either by a web, flange, stiffening lip, intermediate stiffener, or the like.

(b) *Unstiffened Compression Elements.* An unstiffened compression element is a flat compression element which is stiffened at only one edge parallel to the direction of stress.

(c) *Multiple–Stiffened Elements.* A multiple–stiffened element is an element that is stiffened between webs, or between a web and a stiffened edge, by means of intermediate stiffeners which are parallel to the direction of stress. A *sub–element* is the portion between adjacent stiffeners or between web and intermediate stiffener or between edge and intermediate stiffener.

(d) *Flat–Width–to–Thickness Ratio.* The flat width of an element measured along its plane, divided by its thickness.

(e) *Effective Design Width.* Where the flat width of an element is reduced for design purposes, the reduced design width is termed the effective width or effective design width.

(f) *Thickness.* The thickness, t, of any element or section shall be the base steel thickness, exclusive of coatings.

(g) *Torsional–Flexural Buckling.* Torsional–flexural buckling is a mode of buckling in which compression members can bend and twist simultaneously.

(h) *Point–Symmetric Section.* A point–symmetric section is a section symmetrical about a point (centroid) such as a Z–section having equal flanges.

(i) *Yield Point.* Yield point, F_y or F_{sy}, as used in this Specification shall mean yield point or yield strength.

(j) *Stress.* Stress as used in this Specification means force per unit area.

(k) *Confirmatory Test.* A confirmatory test is a test made, when desired, on members, connections, and assemblies designed according to the provisions of Sections A through E of this Specification or its specific references, in order to compare actual versus calculated performance.

(l) *Performance Test.* A performance test is a test made on structural members, connections, and assemblies whose performance cannot be determined by the provisions of Sections A through E of this Specification or its specific references.

(m) *Virgin Steel.* Virgin steel refers to steel as received from the steel producer or warehouse before being cold worked as a result of fabricating operations.

(n) *Virgin Steel Properties.* Virgin steel properties refer to mechanical properties of virgin steel such as yield point, tensile strength, and elongation.

(o) *Specified Minimum Yield Point.* The specified minimum yield point is the lower limit of yield point which must be equalled or exceeded in a specification test to qualify a lot of steel for use in a cold–formed steel structural member designed at that yield point.

(p) *Cold–Formed Steel Structural Members.* Cold–formed steel structural members are shapes which are manufactured by press–braking blanks sheared from sheets, cut lengths of coils or plates, or by roll forming cold– or hot–rolled coils or sheets; both forming operations being performed at ambient room temperature, that is, without manifest addition of heat such as would be required for hot forming.

(q) *LRFD (Load and Resistance Factor Design).* A method of proportioning structural components (members, connectors, connecting elements and assemblages) such that no applicable limit state is exceeded when the structure is subjected to all appropriate load combinations.

(r) *Design Strength.* Factored resistance or strength (force, moment, as appropriate), ϕR_n, provided by the structural component.

(s) *Required Strength.* Load effect (force, moment, as appropriate) acting on the structural component determined by structural analysis from the factored loads (using most appropriate critical load combinations).

A1.3 Units of Symbols and Terms

The Specification is written so that any compatible system of units may be used except where explicitly stated otherwise in the text of these provisions.

A2 Non–Conforming Shapes and Construction

The provisions of the Specification are not intended to prevent the use of alternate shapes or constructions not specifically prescribed herein. Such alternates shall meet the provisions of Section F of the Specification and be approved by the appropriate building code authority.

A3 Material

A3.1 Applicable Steels

This Specification requires the use of steel of structural quality as defined in general by the provisions of the following specifications of the American Society for Testing and Materials:

ASTM A36/A36M, Structural Steel

ASTM A242/A242M, High–Strength Low–Alloy Structural Steel

ASTM A441M, High–Strength Low–Alloy Structural Manganese Vanadium Steel

ASTM A446/A446M (Grades A, B, C, D, & F) Steel, Sheet, Zinc–Coated (Galvanized) by the Hot–Dip Process, Structural (Physical) Quality

ASTM A500, Cold–Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

ASTM A529/A529M, Structural Steel with 42 ksi Minimum Yield Point (1/2 in. Maximum Thickness)

ASTM A570/A570M Steel, Sheet and Strip, Carbon, Hot–Rolled, Structural Quality

ASTM A572/A572M, High–Strength Low–Alloy Columbium–Vanadium Steels of Structural Quality

ASTM A588/A588M, High–Strength Low–Alloy Structural Steel with 50 ksi Minimum Yield Point to 4 in. Thick

ASTM A606 Steel, Sheet and Strip, High Strength, Low Alloy, Hot–Rolled and Cold–Rolled, with Improved Atmospheric Corrosion Resistance

ASTM A607 Steel Sheet and Strip, High Strength, Low Alloy, Columbium or Vanadium, or both, Hot–Rolled and Cold–Rolled

ASTM A611 (Grades A, B, C, & D) Steel, Sheet, Carbon, Cold–Rolled, Structural Quality

ASTM A715 (Grades 50 and 60) Sheet Steel and Strip, High–Strength, Low–Alloy, Hot–Rolled, With Improved Formability

ASTM A792 (Grades 33, 37, 40 & 50) Steel Sheet, Aluminum–Zinc Alloy–Coated by the Hot–Dip Process, General Requirements

A3.2 Other Steels

The listing in Section A3.1 does not exclude the use of steel up to and including one inch in thickness ordered or produced to other than the listed specifications provided such steel conforms to the chemical and mechanical requirements of one of the listed specifications or other published specification which establishes its properties and suitability, and provided it is subjected by either the producer or the purchaser to analyses, tests and other controls to the extent and in the manner prescribed by one of the listed specifications and Section A3.3.

A3.3 Ductility

Steels not listed in Section A3.1 and used for structural members and connections shall comply with one of the following ductility requirements:

A3.3.1 The ratio of tensile strength to yield point shall not be less than 1.08, and the total elongation shall not be less than 10 percent for a two–inch gage length or 7 percent for an eight–inch gage length standard specimen tested in accordance with ASTM A370. If these requirements cannot be met, the following criteria shall be satisfied: (1) local elongation in a 1/2 inch gage length across the fracture shall not be less than 20%, (2) uniform elongation outside the fracture shall not be less than 3%*. When material ductility is determined on the basis of the local and uniform elongation criteria, the use of such material is restricted to the design of purlins and girts** in

* Further information on the test procedures should be obtained from the Commentary.

** Horizontal structural members which support roof deck or panel covering and applied loads principally by bending.

accordance with Sections C3.1.1(a), C3.1.2, and C3.1.3. For purlins and girts subject to combined axial load and bending moment (Section C5), $P_u/\phi_c P_n$ shall not exceed 0.15.

A3.3.2 Steels conforming to ASTM A446 Grade E and A611 Grade E and other steels which do not meet the provisions of Section A3.3.1 may be used for particular configurations provided (1) the yield strength, F_y, used for design in Chapters B, C and D is taken as 75 percent of the specified minimum yield point or 60 ksi, whichever is less and (2) the tensile strength, F_u, used for design in Chapter E is taken as 75 percent of the specified minimum tensile stress or 62 ksi, whichever is less. Alternatively, the suitability of such steels for the configuration shall be demonstrated by load tests in accordance with Section F1. Design strengths based on these tests shall not exceed the strengths calculated according to Chapters B through E, using the specified minimum yield point, F_{sy}, for F_y and the specified minimum tensile strength, F_u.

Design strengths based on existing use shall not exceed the strengths calculated according to Chapters B through E, using the specified minimum yield point, F_{sy}, for F_y and the specified minimum tensile strength, F_u.

A3.4 Delivered Minimum Thickness

The uncoated minimum steel thickness of the cold–formed product as delivered to the job site shall not at any location be less than 95 percent of the thickness, t, used in its design; however, lesser thicknesses shall be permitted at bends, such as corners, due to cold–forming effects.

A4 Loads

A4.1 Dead Load

The dead load to be assumed in design shall consist of the weight of steelwork and all material permanently fastened thereto or supported thereby.

A4.2 Live or Snow Load

The live or snow load shall be that stipulated by the applicable code or specification under which the structure is being designed or that dictated by the conditions involved.

A4.3 Impact Load

For structures carrying live loads which induce impact, the assumed live load shall be increased sufficiently to provide for impact.

A4.4 Wind or Earthquake Loads

Wind or earthquake load shall be that stipulated by the applicable code or specification under which the structure is being designed or that dictated by the conditions involved.

A4.5 Ponding

Unless a roof surface is provided with sufficient slope toward points of free drainage or adequate individual drains to prevent the accumulation of rainwater, the roof system shall be investigated by rational analysis to assure stability under ponding conditions.

A5 Structural Analysis and Design

A5.1 Design Basis

This Specification is based on the Load and Resistance Factor Design concept. Load and Resistance Factor Design is a method of proportioning cold–formed steel structural components (i.e., members, connectors and connections) such that any applicable limit state is not exceeded when the structure is subjected to any appropriate load combination.

Two types of limit states are to be considered: 1) the limit state of the strength required to resist the extreme loads during the intended life of the structure, and 2) the limit state of the ability of the structure to perform its intended function during its life. These limit states will be called the Limit State of Strength and the Limit State of Serviceability, respectively, in these criteria.

A5.1.1 Limit State – Strength

The design meets this Specification when the required strengths, as determined from the assigned nominal loads which are multiplied by appropriate load factors, are smaller than or equal to the design strength of each structural component.

The design strength is equal to ϕR_n, where ϕ is a resistance factor and R_n is the nominal strength determined according to the formulas given in Chapter C for members, in Chapter D for structural assemblies and in Chapter E for connections. Values of resistance factors ϕ are given in Section A5.1.5 for the appropriate limit states governing member and connection strength.

A5.1.2 Limit State – Serviceability

Serviceability is satisfactory if a nominal structural response (e.g. live load deflection) due to the applicable nominal loads is less than or equal to the appropriate acceptable or allowable value of this response.

A5.1.3 Nominal Loads

The nominal loads shall be the minimum design loads stipulated by the applicable code under which the structure is designed or dictated by the conditions involved. In the absence of a code, the loads and load combinations shall be those stipulated in the American Society of Civil Engineers Standard, ANSI/ASCE 7–88, Minimum Design Loads for Buildings and Other Structures. For design purposes, the loads stipulated by the applicable code shall be taken as nominal loads.

A5.1.4 Load Factors and Load Combinations*

The structure and its components must be designed for the appropriate most critical load combination. The following load combinations of the factored nominal loads shall be used in the computation of the required strengths:

1. $1.4\,D + L$
2. $1.2\,D + 1.6\,L + 0.5(L_r \text{ or } S \text{ or } R_r)$
3. $1.2\,D + (1.4\,L_r \text{ or } 1.6\,S \text{ or } 1.6\,R_r) + (0.5\,L \text{ or } 0.8\,W)$

* For roof and floor construction, recommended load combinations for dead load, weight of wet concrete, and construction load including equipment, workmen and formwork are given in Section A5.1 of the Commentary.

4. $1.2\,D + 1.3\,W + 0.5\,L + 0.5(L_r \text{ or } S \text{ or } R_r)$
5. $1.2\,D + 1.5\,E + (0.5\,L \text{ or } 0.2\,S)$
6. $0.9\,D - (1.3\,W \text{ or } 1.5\,E)$

where D = nominal dead load
E = nominal earthquake load
L = nominal live load
L_r = nominal roof live load
R_r = nominal roof rain load
S = nominal snow load
W = nominal wind load (Exception: For wind load on individual purlins, girts, wall panels and roof decks, multiply the load factor for W by 0.9)

Exception: The load factor for L in combinations (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.

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 change, shrinkage, moisture changes, creep in component materials, movement due to differential settlement, or combinations thereof.

A5.1.5 Resistance Factors

The resistance factors to be used for determining the design strengths, ϕR_n, of structural members and connections shall be taken as follows:

Type of Strength	Resistance Factor, ϕ
(a) Stiffeners	
Transverse stiffeners	0.85
Shear stiffeners*	0.90
(b) Tension members	0.95
(c) Flexural members	
Bending strength	
For sections with stiffened or partially stiffened compression flanges	0.95
For sections with unstiffened compression flanges	0.90
Laterally unbraced beams	0.90
Beams having one flange through–fastened to deck or sheathing (C– or Z–sections)	0.90
Web design	
Shear strength*	0.90
Web Crippling	
For single unreinforced webs	0.75
For I–sections	0.80

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

Type of Strength	Resistance Factor, ϕ
(d) Concentrically loaded compression members	0.85
(e) Combined axial load and bending	
ϕ_c for compression	0.85
ϕ_b for bending	
Using Section C3.1.1	0.90 – 0.95
Using Section C3.1.2	0.90
(f) Cylindrical tubular members	
Bending strength	0.95
Axial compression	0.85
(g) Wall studs and wall stud assemblies	
Wall studs in compression	0.85
Wall studs in bending	
For sections with stiffened or partially stiffened compression flanges	0.95
For sections with unstiffened compression flanges	0.90
(h) Welded connections	
Groove welds	
Tension or compression	0.90
Shear (welds)	0.80
Shear (base metal)	0.90
Arc spot welds	
Welds	0.60
Connected part	0.50 – 0.65
Minimum edge distance	0.60 – 0.70
Arc seam welds	
Welds	0.60
Connected part	0.60
Fillet welds	
Longitudinal loading (connected part)	0.55 – 0.60
Transverse loading (connected part)	0.60
Welds	0.60
Flare groove welds	
Transverse loading (connected part)	0.55
Longitudinal loading (connected part)	0.55
Welds	0.60
Resistance Welds	0.65
(i) Bolted connections	
Minimum spacing and edge distance	0.60 – 0.70
Tension strength on net section	
With washers	
Double shear connection	0.65
Single shear connection	0.55
Without washers	0.65
Bearing strength	
See Tables E3.3–1 and E3.3–2	0.55 – 0.70
Shear strength of bolts	0.65
Tensile strength of bolts	0.75
(j) Shear rupture	0.75
(k) Connections to other materials (Bearing)	0.60

A5.2 Yield Point and Strength Increase from Cold Work of Forming

A5.2.1 Yield Point

The yield point used in design, F_y, shall not exceed the specified minimum yield point of steels as listed in Section A3.1 or A3.2, as established in accordance with Chapter F, or as increased for cold work of forming in Section A5.2.2, or as reduced for low ductility steels in Section A3.3.2.

A5.2.2 Strength Increase from Cold Work of Forming

Strength increase from cold work of forming shall be permitted by substituting F_{ya} for F_y, where F_{ya} is the average yield point of the full section. Such increase shall be limited to Sections C3.1 (excluding Section C3.1.1(b)), C4, C5, C6 and D4. The limitations and methods for determining F_{ya} are as follows:

(a) For axially loaded compression members and flexural members whose proportions are such that the quantity ρ for load capacity is unity as determined according to Section B2 for each of the component elements of the section, the design yield stress, F_{ya}, of the steel shall be determined on the basis of one of the following methods:

(1) full section tensile tests [see paragraph (a) of Section F3.1]

(2) stub column tests [see paragraph (b) of Section F3.1]

(3) computed as follows:

$$F_{ya} = CF_{yc} + (1 - C)\, F_{yf} \qquad (Eq.\ A5.2.2\text{–}1)$$

where

F_{ya} = Average yield point of the steel in the full section of compression members or full flange sections of flexural members

C = For compression members, ratio of the total corner cross–sectional area to the total cross–sectional area of the full section; for flexural members, ratio of the total corner cross–sectional area of the controlling flange to the full cross–sectional area of the controlling flange

F_{yf} = Weighted average tensile yield point of the flat portions established in accordance with Section F3.2 or virgin steel yield point if tests are not made

F_{yc} = $B_c F_{yv}/(R/t)^m$, tensile yield point of corners. This formula is applicable only when $F_{uv}/F_{yv} \geq 1.2$, $R/t \leq 7$, and minimum included angle $\leq 120°$ (*Eq.* A5.2.2–2)

B_c = $3.69\,(F_{uv}/F_{yv}) - 0.819\,(F_{uv}/F_{yv})^2 - 1.79$ (*Eq.* A5.2.2–3)

m = $0.192\,(F_{uv}/F_{yv}) - 0.068$ (*Eq.* A5.2.2–4)

R = Inside bend radius.

F_{yv} = Tensile yield point of virgin steel* specified by Section A3 or established in accordance with Section F3.3

F_{uv} = Ultimate tensile strength of virgin steel* specified by Section A3 or established in accordance with Section F3.3

(b) For axially loaded tension members the yield point of the steel shall be determined by either method (1) or method (3) prescribed in paragraph (a) of this Section.

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* Virgin steel refers to the condition (i.e., coiled or straight) of the steel prior to the cold–forming operation.

(c) The effect of any welding on mechanical properties of a member shall be determined on the basis of tests of full section specimens containing within the gage length, such welding as the manufacturer intends to use. Any necessary allowance for such effect shall be made in the structural use of the member.

A5.3 Durability

A structure shall be designed to perform its required functions during its expected life for durability considerations.

A6 Reference Documents

The following documents are referenced in this Specification:

1. American Society of Civil Engineers, ANSI/ASCE 7–88, "Minimum Design Loads in Buildings and Other Structures," American Society of Civil Engineers (ASCE), 345 East 47th Street, New York, N.Y. 10017
2. American Institute of Steel Construction, "Load and Resistance Factor Design Specification for Structural Steel Buildings", American Institute of Steel Construction (AISC), One East Wacker Drive, Suite 3100, Chicago, Illinois 60601–2001, September 1, 1986
3. American Welding Society, AWS D1.3–89, "Structural Welding Code – Sheet Steel," American Welding Society (AWS), 550 N.W. LeJeune Road, Miami, Florida 33135
4. American Welding Society, AWS C1.1–66, "Recommended Practices for Resistance Welding," American Welding Society (AWS), 550 N.W. LeJeune Road, Miami, Florida 33135
5. American Welding Society, AWS C1.3–70, "Recommended Practices for Resistance Welding Coated Low Carbon Steels," American Welding Society (AWS), 550 N.W. LeJeune Road, Miami, Florida 33135
6. American Society for Testing and Materials (ASTM), 1916 Race Street, Philadelphia, Pennsylvania 19013:

   ASTM A36/A36M–84a, Structural Steel

   ASTM A194–88, Carbon and Alloy Steel Nuts for Bolts for High–Pressure and High–Temperature Service

   ASTM A242/A242M–85, High–Strength Low–Alloy Structural Steel

   ASTM A307–84 (Type A), Carbon Steel Externally and Internally Threaded Standard Fasteners

   ASTM A325–84, High Strength Bolts for Structural Steel Joints

   ASTM A354–84 (Grade BD), Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners (for diameter of bolt smaller than 1/2 inch)

   ASTM A370–77 Mechanical Testing of Steel Products

   ASTM A441M–85, High–Strength Low–Alloy Structural Manganese Vanadium Steel

   ASTM A446/A446M–85 (Grades A, B, C, D, & F) Steel, Sheet, Zinc–Coated (Galvanized) by the Hot–Dip Process, Structural (Physical) Quality

   ASTM A449–84a, Quenched and Tempered Steel Bolts and Studs (for diameter of bolt smaller than 1/2 inch)

ASTM A490–84, Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints.

ASTM A500–84, Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes

ASTM A529/A529M–85, Structural Steel with 42 ksi Minimum Yield Point ($^1/_2$ in. Maximum Thickness)

ASTM A563–88a, Carbon and Alloy Steel Nuts

ASTM A570/A570M–85 Steel, Sheet and Strip, Carbon, Hot–Rolled, Structural Quality

ASTM A572/A572M–85, High–Strength Low–Alloy Columbium–Vanadium Steels of Structural Quality

ASTM A588/A588M–85, High–Strength Low–Alloy Structural Steel with 50 ksi Minimum Yield Point to 4 in. Thick

ASTM A606–85 Steel, Sheet and Strip, High Strength, Low Alloy, Hot–Rolled and Cold–Rolled, with Improved Atmospheric Corrosion Resistance

ASTM A607–85 Steel Sheet and Strip, High Strength, Low Alloy, Columbium or Vanadium, or both, Hot–Rolled and Cold–Rolled

ASTM A611–85 (Grades A, B, C, & D) Steel, Sheet, Carbon, Cold–Rolled, Structural Quality

ASTM A715–85 (Grades 50 & 60) Sheet Steel and Strip, High–Strength, Low–Alloy, Hot–Rolled, With Improved Formability

ASTM A792–85a (Grades 33, 37, 40 & 50) Steel Sheet, Aluminum–Zinc Alloy–Coated by the Hot–Dip Process, General Requirements

ASTM F436–86, Hardened Steel Washers

ASTM F844–83(1988), Washers, Steel, Plain (Flat), Unhardened for General Use

ASTM F959–85, Compressible Washer–Type Direct Tension Indicators for Use with Structural Fasteners

B. ELEMENTS

B1 Dimensional Limits and Considerations

B1.1 Flange Flat-Width-to-Thickness Considerations

(a) *Maximum Flat-Width-to-Thickness Ratios*

Maximum allowable overall flat-width-to-thickness ratios, w/t, disregarding intermediate stiffeners and taking as t the actual thickness of the element, shall be as follows:

(1) Stiffened compression element having *one* longitudinal edge connected to a web or flange element, the other stiffened by:

Simple lip	60
Any other kind of stiffener having $I_s > I_a$ and $D/w < 0.8$ according to Section B4.2	90

(2) Stiffened compression element with *both* longitudinal edges connected to other stiffened elements — 500

(3) Unstiffened compression element and elements with an edge stiffener having $I_s < I_a$ and $D/w \leq 0.8$ according to Section B4.2 — 60

Note: Unstiffened compression elements that have w/t ratios exceeding approximately 30 and stiffened compression elements that have w/t ratios exceeding approximately 250 are likely to develop noticeable deformation at the full design strength, without affecting the ability of the member to develop required strength.

Stiffened elements having w/t ratios larger than 500 can be used with adequate design strength to sustain the required loads; however, substantial deformations of such elements usually will invalidate the design formulas of this Specification.

(b) *Flange Curling*

Where the flange of a flexural member is unusually wide and it is desired to limit the maximum amount of curling or movement of the flange toward the neutral axis, the following formula applies to compression and tension flanges, either stiffened or unstiffened:

$$w_f = \sqrt{0.061 t d E / f_{av}} \sqrt[4]{(100 c_f / d)} \qquad (Eq.\ B1.1\text{-}1)$$

where

w_f = Width of flange projecting beyond the web; or half of the distance between webs for box- or U-type beams

t = Flange thickness

d = Depth of beam

c_f = Amount of curling*

f_{av} = Average stress in the full, unreduced flange width. (Where members are designed by the effective design width procedure, the average stress equals the maximum stress multiplied by the ratio of the effective design width to the actual width.)

(c) *Shear Lag Effects – Short Spans Supporting Concentrated Loads*

Where the span of the beam is less than $30w_f$ (w_f as defined below) and it carries one concentrated load, or several loads spaced farther apart than $2w_f$, the effective design width of any flange, whether in tension or compression, shall be limited to the following:

TABLE B1.1(c)

SHORT, WIDE FLANGES

MAXIMUM ALLOWABLE RATIO OF EFFECTIVE DESIGN WIDTH TO ACTUAL WIDTH

L/w_f	Ratio	L/w_f	Ratio
30	1.00	14	0.82
25	0.96	12	0.78
20	0.91	10	0.73
18	0.89	8	0.67
16	0.86	6	0.55

where

L = Full span for simple beams; or the distance between inflection points for continuous beams; or twice the length of cantilever beams.

w_f = Width of flange projection beyond the web for I–beam and similar sections or half the distance between webs of box or U–type sections.

For flanges of I–beams and similar sections stiffened by lips at the outer edges, w_f shall be taken as the sum of the flange projection beyond the web plus the depth of the lip.

B1.2 Maximum Web Depth–to–Thickness Ratio

The ratio, h/t, of the webs of flexural members shall not exceed the following limitations:

(a) For unreinforced webs: $(h/t)_{max} = 200$

(b) For webs which are provided with transverse stiffeners satisfying the requirements of Section B6.1:

(1) When using bearing stiffeners only, $(h/t)_{max} = 260$

(2) When using bearing stiffeners and intermediate stiffeners, $(h/t)_{max} = 300$

In the above,

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

* The amount of curling that can be tolerated will vary with different kinds of sections and must be established by the designer. Amount of curling in the order of 5 percent of the depth of the section is usually not considered excessive.

t = Web thickness

Where a web consists of two or more sheets, the h/t ratio shall be computed for the individual sheets.

B2 Effective Widths of Stiffened Elements

B2.1 Uniformly Compressed Stiffened Elements

(a) *Load Capacity Determination*

The effective widths, b, of uniformly compressed elements shall be determined from the following formulas:

$b = w$ when $\lambda \leq 0.673$ (*Eq.* B2.1–1)

$b = \rho w$ when $\lambda > 0.673$ (*Eq.* B2.1–2)

where

w = Flat width as shown in Figure B2.1–1

$\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq.* B2.1–3)

λ is a slenderness factor determined as follows:

$$\lambda = \frac{1.052}{\sqrt{k}}\left(\frac{w}{t}\right)\sqrt{\frac{f}{E}}$$ (*Eq.* B2.1–4)

where

t = Thickness of the uniformly compressed stiffened elements, and

f for load capacity determination is as follows:

For flexural members:

(1) If Procedure I of Section C3.1.1 is used, $f = F_y$ if the initial yielding is in compression in the element considered.
If the initial yielding is in tension, the compressive stress, f, in the element considered shall be determined on the basis of the effective section at M_y (moment causing initial yield).

(2) If Procedure II of Section C3.1.1 is used then f is the stress in the element considered at M_n determined on the basis of the effective section.

(3) If Section C3.1.2 is used, then f is the stress $\frac{M_c}{S_f}$ as described in that Section in determining S_c.

For compression members f is taken equal to F_n as determined in Section C4 or D4.1 as applicable.

E = Modulus of elasticity

k = Plate buckling coefficient

= 4 for stiffened elements supported by a web on each longitudinal edge. Values for different types of elements are given in the applicable sections.

(b) *Deflection Determination*

The effective widths, b_d, used in computing deflection shall be determined from the following formulas:

$b_d = w$ when $\lambda \leq 0.673$ (*Eq.* B2.1–5)

$b_d = \rho w$ when $\lambda > 0.673$ (*Eq.* B2.1–6)

where

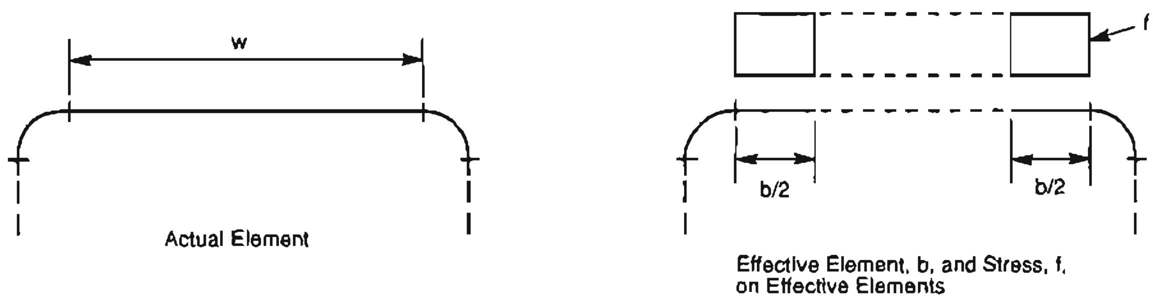


Figure B2.1-1 Stiffened Elements

w = Flat width

ρ = Reduction factor determined by either of the following two procedures:

(1) Procedure I.
A low estimate of the effective width may be obtained from Eqs. B2.1-3 and B2.1-4 where f_d is substituted for f where f_d is the computed compressive stress in the element being considered.

(2) Procedure II.
For stiffened elements supported by a web on each longitudinal edge an improved estimate of the effective width can be obtained by calculating ρ as follows:

$\rho = 1$ when $\lambda \leq 0.673$ (*Eq.* B2.1-7)

$\rho = (1.358 - 0.461/\lambda)/\lambda$ when $0.673 < \lambda < \lambda_c$ (*Eq.* B2.1-8)

$\rho = (0.41 + 0.59\sqrt{F_y / f_d} - 0.22/\lambda)/\lambda$ when $\lambda \geq \lambda_c$ (*Eq.* B2.1-9)

ρ shall not exceed 1.0 for all cases.

where

$\lambda_c = 0.256 + 0.328\,(w/t)\sqrt{F_y / E}$ (*Eq.* B2.1-10)

and λ is as defined by Eq. B2.1-4 except that f_d is substituted for f.

B2.2 Uniformly Compressed Stiffened Elements with Circular Holes

(a) *Load Capacity Determination*

The effective width, b, of stiffened elements with uniform compression having circular holes shall be determined as follows:

for $0.50 \geq \dfrac{d_h}{w} \geq 0$, and $\dfrac{w}{t} \leq 70$

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

$b = w - d_h$ when $\lambda \leq 0.673$ (*Eq.* B2.2-1)

$$b = \frac{w\left[1 - \dfrac{(0.22)}{\lambda} - \dfrac{(0.8d_h)}{w}\right]}{\lambda} \quad \text{when } \lambda > 0.673 \qquad (Eq.\ \mathrm{B2.2\text{-}2})$$

b shall not exceed $w - d_h$

where

w = Flat width

d_h = Diameter of holes

λ is as defined in Section B2.1.

(b) *Deflection Determination*

The effective width, b_d, used in deflection calculations shall be equal to b determined in accordance with Procedure I of Section B2.2a except that f_d is substituted for f, where f_d is the computed compressive stress in the element being considered.

B2.3 Effective Widths of Webs and Stiffened Elements with Stress Gradient

(a) *Load Capacity Determination*

The effective widths, b_1 and b_2, as shown in Figure B2.3–1 shall be determined from the following formulas:

$$b_1 = b_e/(3 - \psi) \qquad (Eq.\ B2.3\text{–}1)$$

For $\psi \leq -0.236$

$$b_2 = b_e/2 \qquad (Eq.\ B2.3\text{–}2)$$

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

For $\psi > -0.236$

$$b_2 = b_e - b_1 \qquad (Eq.\ B2.3\text{–}3)$$

where

b_e = Effective width b determined in accordance with Section B2.1 with f_1 substituted for f and with k determined as follows:

$$k = 4 + 2(1 - \psi)^3 + 2(1 - \psi) \qquad (Eq.\ B2.3\text{–}4)$$

$\psi = f_2/f_1$

f_1, f_2 = Stresses shown in Figure B2.3–1 calculated on the basis of effective section.

f_1 is compression (+) and f_2 can be either tension (–) or compression. In case f_1 and f_2 are both compression, $f_1 \geq f_2$

(b) *Deflection Determination*

The effective widths in computing deflections at a given load shall be determined in accordance with Section B2.3a except that f_{d1} and f_{d2} are substituted for f_1 and f_2, where f_{d1}, f_{d2} = Computed stresses f_1 and f_2 as shown in Figure B2.3–1. Calculations are based on the effective section at the load for which deflections are determined.

B3 Effective Widths of Unstiffened Elements

B3.1 Uniformly Compressed Unstiffened Elements

(a) *Load Capacity Determination*

Effective widths, b, of unstiffened compression elements with uniform compression shall be determined in accordance with Section B2.1a with the exception that k shall be taken as 0.43 and w as defined in Figure B3.1–1.

(b) *Deflection Determination*

The effective widths used in computing deflections shall be determined in accordance with Procedure I of Section B2.1b except that f_d is substituted for f and k = 0.43.

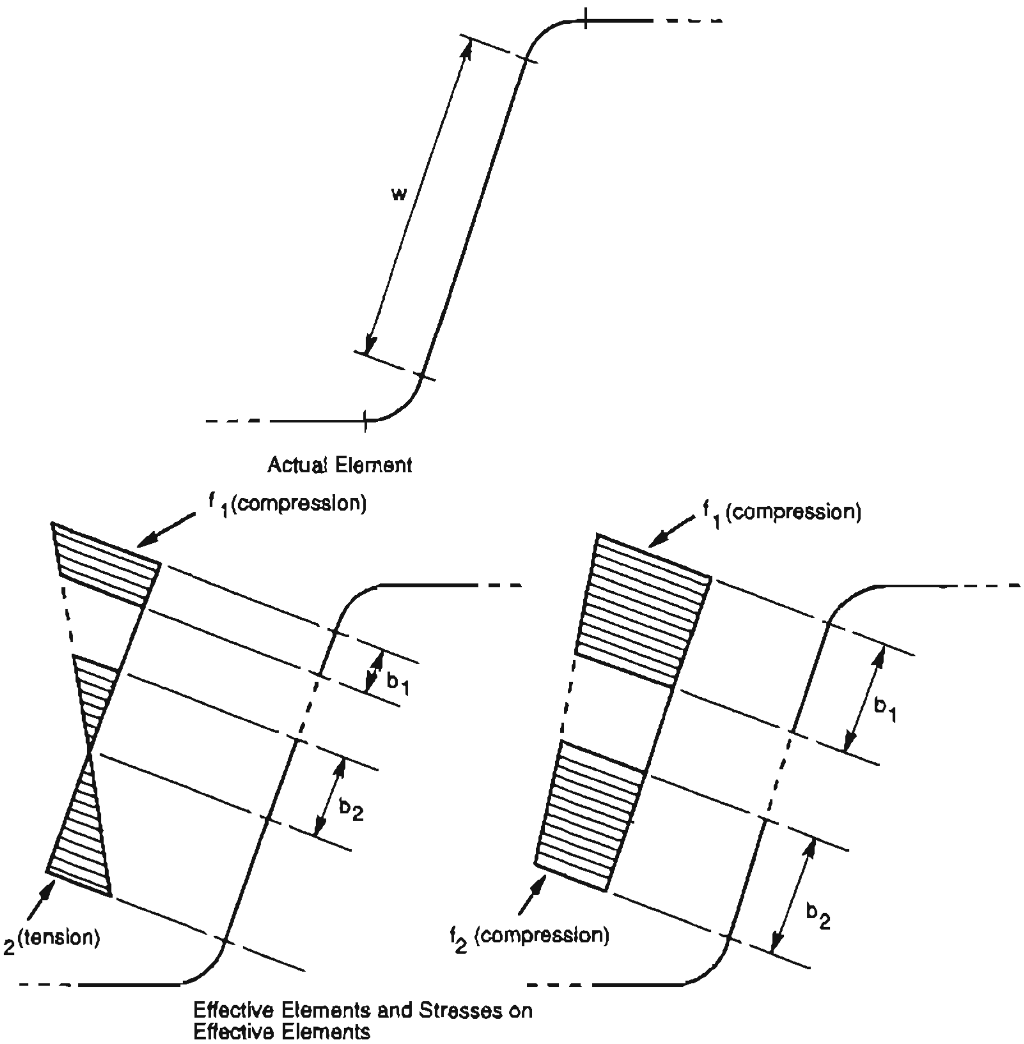


Figure B2.3–1 Stiffened Elements with Stress Gradient and Webs

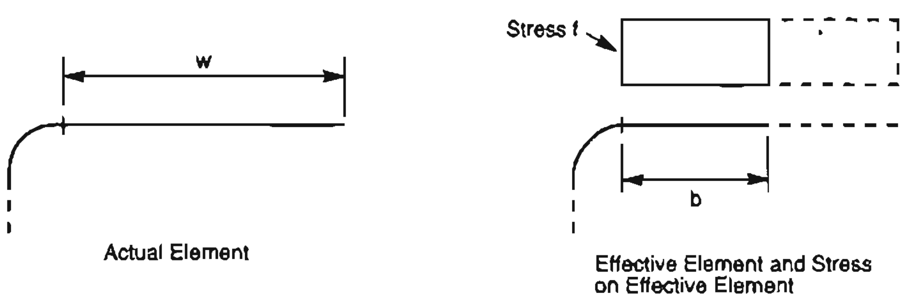


Figure B3.1–1 Unstiffened Element with Uniform Compression

B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

(a) *Load Capacity Determination*
Effective widths, b, of unstiffened compression elements and edge stiffeners with stress gradient shall be determined in accordance with Section B2.1a with $f = f_3$ as in Figure B4–2 in the element and $k = 0.43$.

(b) *Deflection Determination*
Effective widths, b, of unstiffened compression elements and edge stiffeners with

stress gradient shall be determined in accordance with Procedure I of Section B2.1b except that f_{d3} is substituted for f and k = 0.43.

B4 Effective Widths of Elements with an Edge Stiffener or One Intermediate Stiffener

The following notation is used in this section.

S = $1.28\sqrt{E/f}$ (*Eq.* B4–1)

k = Buckling coefficient

b_o = Dimension defined in Figure B4–1

d, w, D = Dimensions defined in Figure B4–2

d_s = Reduced effective width of the stiffener as specified in this section. d_s, calculated according to Section B4.2, is to be used in computing the overall effective section properties (see Figure B4–2)

d'_s = Effective width of the stiffener calculated according to Section B3.1 (see Figure B4–2)

C_1, C_2 = Coefficients defined in Figure B4–2

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

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

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

For the stiffener shown in Figure B4–2:

I_s = $(d^3 t \sin^2\theta)/12$ (*Eq.* B4–2)

A'_s = $d'_s t$ (*Eq.* B4–3)

B4.1 Uniformly Compressed Elements with an Intermediate Stiffener

(a) *Load Capacity Determination*

Case I: $b_o/t \le S$ (*Eq.* B4.1–1)

$I_a = 0$ (no intermediate stiffener needed) (*Eq.* B4.1–2)

$b = w$ (*Eq.* B4.1–3)

$A_s = A'_s$ (*Eq.* B4.1–4)

Case II: $S < b_o/t < 3S$ (*Eq.* B4.1–5)

$I_a/t^4 = [50(b_o/t)/S] - 50$ (*Eq.* B4.1–6)

b and A_s are calculated according to Section B2.1a where

$k = 3(I_s/I_a)^{1/2} + 1 \le 4$ (*Eq.* B4.1–7)

$A_s = A'_s(I_s/I_a) \le A'_s$ (*Eq.* B4.1–8)

Case III: $b_o/t \ge 3S$

$I_a/t^4 = [128(b_o/t)/S] - 285$ (*Eq.* B4.1–9)

b and A_s are calculated according to Section B2.1a where

$k = 3(I_s/I_a)^{1/3} + 1 \le 4$ (*Eq.* B4.1–10)

$A_s = A'_s(I_s/I_a) \le A'_s$ (*Eq.* B4.1–11)

(b) *Deflection Determination*

Effective widths shall be determined as in Section B4.1a except that f_d is substituted for f.

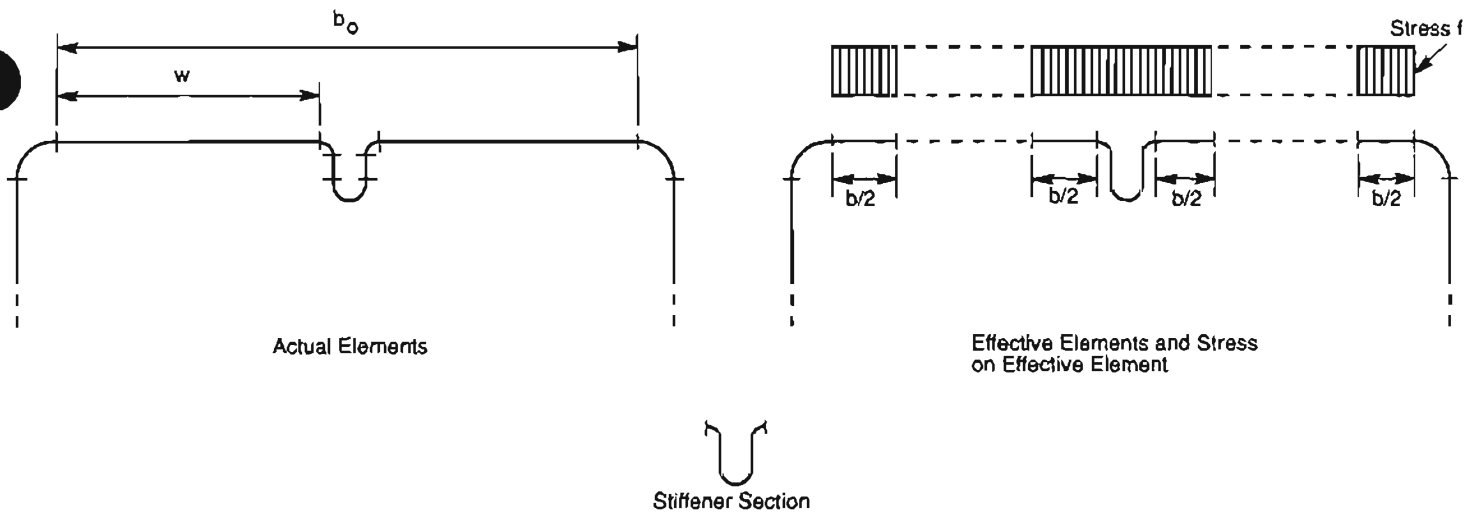


Figure B4–1 Elements with Intermediate Stiffener

B4.2 Uniformly Compressed Elements with an Edge Stiffener

(a) *Load Capacity Determination*

Case I: $w/t \leq S/3$ (*Eq.* B4.2–1)

$I_a = 0$ (no edge stiffener needed) (*Eq.* B4.2–2)

$b = w$ (*Eq.* B4.2–3)

$d_s = d'_s$ for simple lip stiffener (*Eq.* B4.2–4)

$A_s = A'_s$ for other stiffener shapes (*Eq.* B4.2–5)

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

$I_a/t^4 = 399\{[(w/t)/S] - 0.33\}^3$ (*Eq.* B4.2–6)

$n = 1/2$

$C_2 = I_s/I_a \leq 1$ (*Eq.* B4.2–7)

$C_1 = 2 - C_2$ (*Eq.* B4.2–8)

b shall be calculated according to Section B2.1 where

$k = [4.82 - 5(D/w)](I_s/I_a)^n + 0.43 \leq 5.25 - 5(D/w)$ (*Eq.* B4.2–9)

for $0.8 \geq D/w > 0.25$

$k = 3.57(I_s/I_a)^n + 0.43 \leq 4.0$ (*Eq.* B4.2–10)

for $(D/w) \leq 0.25$

$d_s = d'_s\,(I_s/I_a) \leq d'_s$ (*Eq.* B4.2–11)

for simple lip stiffener

$A_s = A'_s\,(I_s/I_a) \leq A'_s$ (*Eq.* B4.2–12)

for other stiffener shape

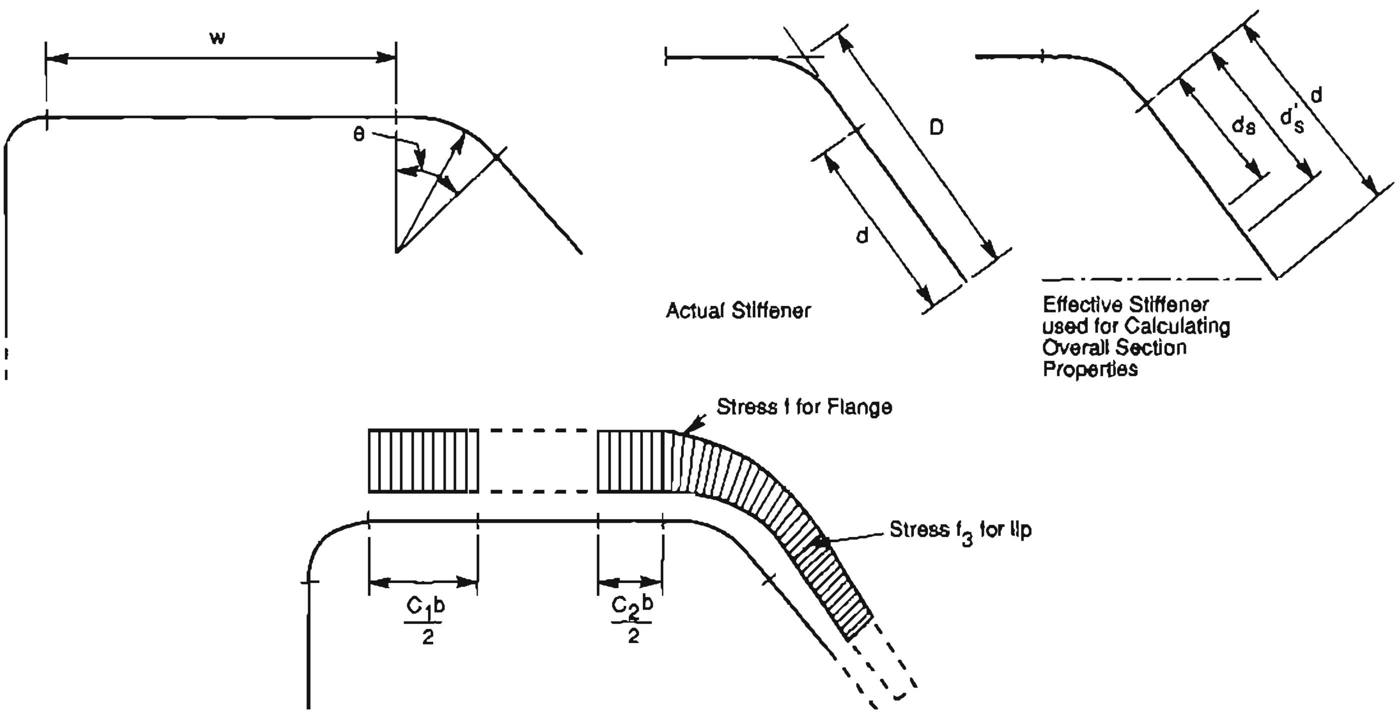
Case III: $w/t \geq S$

$I_a/t^4 = [115\,(w/t)/S] + 5$ (*Eq.* B4.2–13)

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

(b) *Deflection Determination*

Effective widths shall be determined as in Section B4.2a except that f_d is substituted for f.



Effective Element and Stress on Effective Element

Figure B4–2 Elements with Edge Stiffener

B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener

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

$$I_{min} = \left[3.66\sqrt{\left((w/t)^2 - (0.136E)/F_y\right)}\right]t^4 \qquad (Eq.\ B5\text{–}1)$$

but not less than $18.4\ t^4$

where

w/t = Width–thickness ratio of the larger stiffened sub–element

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

(a) If the spacing of intermediate stiffeners between two webs is such that for the sub–element between stiffeners $b < w$ as determined in Section B2.1, only two intermediate stiffeners (those nearest each web) shall be considered effective.

(b) If the spacing of intermediate stiffeners between a web and an edge stiffener is such that for the sub–element between stiffeners $b < w$ as determined in Section B2.1, only one intermediate stiffener, that nearest the web, shall be considered effective.

(c) If intermediate stiffeners are spaced so closely that for the elements between stiffeners $b = w$ as determined in Section B2.1, all the stiffeners may be considered effective. In computing the flat–width to thickness ratio of the entire multiple–stiffened element, such element shall be considered as replaced by an "equivalent element" without intermediate stiffeners whose width, b_o, is the full width between webs or from web to edge stiffener, and whose equivalent thickness, t_s, is determined as follows:

$$t_s = \sqrt[3]{12 I_{sf} / b_o} \qquad (Eq.\ B5\text{–}2)$$

where

I_{sf} = Moment of inertia of the full area of the multiple–stiffened element, including the intermediate stiffeners, about its own centroidal axis. The moment of inertia of the entire section shall be calculated assuming the "equivalent element" to be located at the centroidal axis of the multiple stiffened element, including the intermediate stiffener. The actual extreme fiber distance shall be used in computing the section modulus.

(d) If $w/t > 60$, the effective width, b_e, of the sub–element or element shall be determined from the following formula:

$$\frac{b_e}{t} = \frac{b}{t} - 0.10\left[\frac{w}{t} - 60\right] \qquad (Eq.\ B5\text{–}3)$$

where

w/t = flat–width ratio of sub–element or element

b = effective design width determined in accordance with the provisions of Section B2.1

b_e = effective design width of sub–element or element to be used in design computations

For computing the effective structural properties of a member having compression sub–elements or element subject to the above reduction in effective width, the area of stiffeners (edge stiffener or intermediate stiffeners) shall be considered reduced to an effective area as follows:

For $60 < w/t < 90$:

$$A_{ef} = \alpha A_{st} \qquad (Eq.\ B5\text{–}4)$$

where

$$\alpha = (3 - 2b_e / w) - \frac{1}{30}\left[1 - \frac{b_e}{w}\right]\left[\frac{w}{t}\right] \qquad (Eq.\ B5\text{–}5)$$

For $w/t \geq 90$:

$$A_{ef} = (b_e/w)\ A_{st} \qquad (Eq.\ B5\text{–}6)$$

In the above expressions, A_{ef} and A_{st} refer only to the area of the stiffener section, exclusive of any portion of adjacent elements.

The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener, and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener.

B6 Stiffeners

B6.1 Transverse Stiffeners

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. Required strengths for the concentrated loads or reactions shall not exceed the design strength, $\phi_c P_n$, where $\phi_c = 0.85$ and P_n is the smaller value given by (a) and (b) as follows:

(a) $P_n = F_{wy}A_c$ (*Eq.* B6.1–1)

(b) P_n = Nominal axial strength evaluated according to Section C4(a) with A_e replaced by A_b

where

$A_c = 18t^2 + A_s$, for transverse stiffeners at interior support and under concentrated load (*Eq.* B6.1–2)

$A_c = 10t^2 + A_s$, for transverse stiffeners at end support (*Eq.* B6.1–3)

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

$A_b = b_1t + A_s$, for transverse stiffeners at interior support and under concentrated load (*Eq.* B6.1–4)

$A_b = b_2t + A_s$, for transverse stiffeners at end support (*Eq.* B6.1–5)

A_s = Cross sectional area of transverse stiffeners

$b_1 = 25t\,[0.0024(L_{st}/t) + 0.72] \leq 25t$ (*Eq.* B6.1–6)

$b_2 = 12t\,[0.0044(L_{st}/t) + 0.83] \leq 12t$ (*Eq.* B6.1–7)

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.

B6.2 Shear Stiffeners

Where shear stiffeners are required, the spacing shall be such that the required shear strength shall not exceed the design 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 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

$$I_{smin} = 5ht^3[h/a - 0.7(a/h)] \geq (h/50)^4 \qquad (Eq.\ B6.2\text{–}1)$$

The gross area of shear stiffeners shall be not less than

$$A_{st} = \frac{1-C_v}{2}\left[\frac{a}{h} - \frac{(a/h)^2}{(a/h)+\sqrt{1+(a/h)^2}}\right]YDht \qquad (Eq.\ B6.2\text{–}2)$$

where

$$C_v = \frac{45{,}000k_v}{F_y(h/t)^2} \quad \text{when } C_v \leq 0.8 \qquad (Eq.\ B6.2\text{–}3)$$

$$C_v = \frac{190}{h/t}\left(\sqrt{\frac{k_v}{F_y}}\right) \quad \text{when } C_v > 0.8 \qquad (Eq.\ B6.2\text{–}4)$$

$$k_v = 4.00 + \frac{5.34}{(a/h)^2} \quad \text{when } a/h \leq 1.0 \qquad (Eq.\ B6.2\text{–}5)$$

$$k_v = 5.34 + \frac{4.00}{(a/h)^2} \quad \text{when } a/h > 1.0 \qquad (Eq.\ B6.2\text{–}6)$$

a = Distance between transverse stiffeners

$$Y = \frac{\text{Yield point of web steel}}{\text{Yield point of stiffener steel}}$$

D = 1.0 for stiffeners furnished in pairs

D = 1.8 for single–angle stiffeners

D = 2.4 for single–plate stiffeners

t and h are as defined in Section B1.2

B6.3 Non–Conforming Stiffeners

The design strength of members with transverse stiffeners that do not meet the requirements of Section B6.1 or B6.2, such as stamped or rolled–in transverse stiffeners shall be determined by tests in accordance with Chapter F of this Specification.

C. MEMBERS

C1 Properties of Sections

Properties of sections (cross–sectional area, moment of inertia, section modulus, radius of gyration, etc.) shall be determined in accordance with conventional methods of structural design. Properties shall be based on the full cross section of the members (or net sections where the use of net section is applicable) except where the use of a reduced cross section, or effective design width, is required.

C2 Tension Members

For axially loaded tension members, the design tensile strength, $\phi_t T_n$, shall be determined as follows:

$\phi_t = 0.95$

$$T_n = A_n F_y \qquad (Eq.\ C2\text{–}1)$$

where

T_n = Nominal strength of member when loaded in tension

ϕ_t = Resistance factor for tension

A_n = Net area of the cross section

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

For tension members using bolted connections, the design tensile strength shall also be limited by Section E3.2.

C3 Flexural Members

C3.1 Strength for Bending Only

The design flexural strength, $\phi_b M_n$, shall be the smallest of the values calculated according to Sections C3.1.1, C3.1.2, and C3.1.3.

C3.1.1 Nominal Section Strength

The design flexural strength, $\phi_b M_n$, shall be determined with $\phi_b = 0.95$ for sections with stiffened or partially stiffened compression flanges and 0.90 for sections with unstiffened compression flanges, and the nominal section strength, M_n, 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:

$$M_n = S_e F_y \qquad (Eq.\ C3.1.1\text{–}1)$$

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

The nominal flexural strength, M_n, shall not exceed either 1.25 S_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 $w/t \leq \lambda_1$

$$C_y = 3 - 2\left(\frac{w/t - \lambda_1}{\lambda_2 - \lambda_1}\right) \text{ for } \lambda_1 < \frac{w}{t} < \lambda_2$$

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

where

$$\lambda_1 = \frac{1.11}{\sqrt{F_y/E}} \qquad (Eq.\ C3.1.1–2)$$

$$\lambda_2 = \frac{1.28}{\sqrt{F_y/E}} \qquad (Eq.\ C3.1.1–3)$$

(b) Unstiffened compression elements
$C_y = 1$
(c) Multiple–stiffened compression elements and compression elements with edge stiffeners
$C_y = 1$

When applicable, effective design widths shall be used in calculating section properties. M_n shall be calculated considering equilibrium of stresses, assuming an ideally elastic–plastic stress–strain curve which is the same in tension as in compression, assuming small deformation and assuming that plane sections remain plane during bending. Combined bending and web crippling shall be checked by provisions of Section C3.5.

C3.1.2 Lateral Buckling Strength

The design strength of the laterally unbraced segments of singly–, doubly–, and point–symmetric sections* subject to lateral buckling, ϕ_bM_n shall be determined with $\phi_b = 0.90$ and M_n calculated as follows:

$$M_n = S_c \frac{M_c}{S_f} \qquad (Eq.\ C3.1.2–1)$$

where

* The provisions of this Section apply to I–, Z–, C– and other singly–symmetric section flexural members (not including multiple–web deck, U– and closed box–type members, and curved or arch members). The provisions of this Section do not apply to laterally unbraced compression flanges of otherwise laterally stable sections. Refer to C3.1.3 for C– and Z–purlins in which the tension flange is attached to sheathing.

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 singly-, doubly-, and point-symmetric sections:

For $M_e > 0.5M_y$

$$M_c = M_y\left(1 - \frac{M_y}{4M_e}\right) \quad (Eq.\ C3.1.2\text{-}2)$$

For $M_e \le 0.5M_y$

$$M_c = M_e \quad (Eq.\ C3.1.2\text{-}3)$$

where

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

$= S_f F_y$ *(Eq.* C3.1.2-4)

M_e = Elastic critical moment computed by the following equations:

M_e $= C_b r_o A\sqrt{\sigma_{ey}\sigma_t}$ for bending about the symmetry axis. For singly-symmetric sections, x-axis is the axis of symmetry oriented such that the shear center has a negative x-coordinate. For point-symmetric sections, use 0.5 M_e. *(Eq.* C3.1.2-5)

Alternatively, M_e can be calculated using the formula for doubly-symmetric I-sections or point-symmetric sections given in (b)

M_e $= C_s A\sigma_{ex}\left[j + C_s\sqrt{j^2 + r_o^2(\sigma_t / \sigma_{ex})}\right] / C_{TF}$ for bending about the centroidal axis perpendicular to the symmetry axis for singly-symmetric sections only *(Eq.* C3.1.2-6)

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

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

$$\sigma_{ex} = \frac{\pi^2 E}{(K_x L_x / r_x)^2} \quad (Eq.\ C3.1.2\text{-}7)$$

$$\sigma_{ey} = \frac{\pi^2 E}{(K_y L_y / r_y)^2} \quad (Eq.\ C3.1.2\text{-}8)$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \quad (Eq.\ C3.1.2\text{-}9)$$

A = Full cross-sectional area

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

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

where

M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length, taken about the strong axis of the member, and where M_1/M_2, the ratio of end moments, is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite sign (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, and for members subject to combined axial load and bending moment (Section C5), C_b shall be taken as unity.

E = Modulus of elasticity

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

$$= \sqrt{r_x^2 + r_y^2 + x_o^2} \qquad (Eq.\ C3.1.2\text{–}10)$$

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

G = Shear modulus

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

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

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

J = St. Venant torsion constant of the cross section

C_w = Torsional warping constant of the cross section

$$j = \frac{1}{2I_y}\left[\int_A x^3 dA + \int_A xy^2 dA\right] - x_o \qquad (Eq.\ C3.1.2\text{–}11)$$

(b) For I– or Z–sections bent about the centroidal axis perpendicular to the web (x–axis):

In lieu of (a), the following equations may be used to evaluate M_c:

For $M_e \geq 2.78M_y$

$$M_c = M_y \qquad (Eq.\ C3.1.2\text{–}12)$$

For $2.78M_y > M_e > 0.56M_y$

$$M_c = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_e}\right) \qquad (Eq.\ C3.1.2\text{–}13)$$

For $M_e \leq 0.56M_y$

$$M_c = M_e \qquad (Eq.\ C3.1.2\text{–}14)$$

where

M_e = Elastic critical moment determined either as defined in (a) above or as follows:

$$= \frac{\pi^2 E C_b d I_{yc}}{L^2} \text{ for doubly-symmetric I-sections} \quad (Eq.\ C3.1.2\text{–}15)$$

$$= \frac{\pi^2 E C_b d I_{yc}}{2L^2} \text{ for point-symmetric Z-sections} \quad (Eq.\ C3.1.2\text{–}16)$$

d = Depth of section

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 (a).

C3.1.3 Beams Having One Flange Through–Fastened to Deck or Sheathing

This section does not apply to a continuous beam for the region between inflection points adjacent to a support, or to a cantilever beam.

The design flexural strength, $\phi_b M_n$, of a Channel or Z–section loaded in a plane parallel to the web, with the tension flange attached to deck or sheathing and with the compression flange laterally unbraced shall be determined with $\phi_b = 0.90$ and the nominal flexural strength, M_n, calculated as follows:

$$M_n = R S_e F_y \quad (Eq.\ C3.1.3\text{–}1)$$

where

R = 0.40 for simple span C sections
= 0.50 for simple span Z sections
= 0.60 for continuous span C sections
= 0.70 for continuous span Z sections

S_e and F_y are defined in Section C3.1.1

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

(1) Member depth less than 11.5 inches
(2) The flanges are edge stiffened compression elements
(3) $60 \leq$ depth/thickness ≤ 170
(4) $2.8 \leq$ depth/flange width ≤ 4.5
(5) $16 \leq$ flat width/thickness of flange ≤ 43
(6) For continuous span systems, the lap length at each interior support in each direction (distance from center of support to end of lap) shall not be less than:
    1.5d for Zee sections
    3.0d for Channel sections
(7) Member span length no greater than 33 feet
(8) For continuous span systems, the longest member span shall not be more than 20% greater than the shortest span
(9) Both flanges are prevented from moving laterally at the supports
(10) Roof or wall panels shall be steel sheets, minimum of 0.019 in. coated thickness, having a minimum rib depth of 1 in., spaced a maximum of 12 in. on

centers and attached in a manner to effectively inhibit relative movement between the panel and purlin flange

(11) Insulation shall be glass fiber blanket 0 to 6 inches thick compressed between the member and panel in a manner consistent with the fastener being used

(12) Fastener type: minimum No. 12 self–drilling or self–tapping sheet metal screws or 3/16 – in. rivets, washers 1/2 in. diameter

(13) Fasteners shall not be standoff type screws

(14) Fasteners shall be spaced not greater than 12 in. on centers and placed near the center of the beam flange

If variables fall outside any of the above stated limits, the user must perform full scale tests in accordance with Section F1 of the Specification, or apply another rational analysis procedure. In any case, the user is permitted to perform tests, in accordance with Section F1, as an alternate to the procedure described in this section.

C3.2 Strength for Shear Only

The design shear strength, $\phi_v V_n$, at any section shall be calcuated as follows:

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

$\phi_v = 1.0$

$$V_n = 0.577F_y ht \qquad (Eq. \text{C3.2.–1})$$

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

$\phi_v = 0.90$

$$V_n = 0.64t^2\sqrt{k_v F_y E} \qquad (Eq. \text{ C3.2–2})$$

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

$\phi_v = 0.90$

$$V_n = 0.905Ek_v t^3/h \qquad (Eq. \text{ C3–2.3})$$

where

ϕ_v = Resistance factor for shear

V_n = Nominal shear strength of beam

t = Web thickness

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

k_v = Shear buckling coefficient determined as follows:

1. For unreinforced webs, $k_v = 5.34$
2. For beam webs with transverse stiffeners satisfying the requirements of Section B6

when $a/h \le 1.0$

$$k_v = 4.00 + \frac{5.34}{(a/h)^2} \qquad (Eq. \text{ C3.2–4})$$

when $a/h > 1.0$

$$k_v = 5.34 + \frac{4.00}{(a/h)^2} \qquad (Eq. \text{ C3.2–5})$$

where

a = the shear panel length for unreinforced web element

= distance between transverse stiffeners for reinforced web elements.

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

C3.3 Strength for Combined Bending and Shear

For beams with unreinforced webs, the required flexural strength, M_u, and the required shear strength, V_u, shall satisfy the following interaction equation:

$$\left(\frac{M_u}{\phi_b M_{nxo}}\right)^2 + \left(\frac{V_u}{\phi_v V_n}\right)^2 \leq 1.0 \qquad (Eq.\ C3.3\text{-}1)$$

For beams with transverse web stiffeners, the required flexural strength, M_u, and the required shear strength, V_u, shall not exceed $\phi_b M_n$ and $\phi_v V_n$, respectively. When $M_u/(\phi_b M_{nxo}) > 0.5$ and $V_u/(\phi_v V_n) > 0.7$, then M_u and V_u shall satisfy the following interaction equation:

$$0.6\left(\frac{M_u}{\phi_b M_{nxo}}\right) + \left(\frac{V_u}{\phi_v V_n}\right) \leq 1.3 \qquad (Eq.\ C3.3\text{-}2)$$

In the above:

ϕ_b = Resistance factor for bending (See Section C3.1)
ϕ_v = Resistance factor for shear (See Section C3.2)
M_n = Nominal flexural strength when bending alone exists
M_{nxo} = Nominal flexural strength about the centroidal x-axis determined in accordance with Section C3.1 excluding the provisions of Section C3.1.2
V_n = Nominal shear strength when shear alone exists

C3.4 Web Crippling Strength

These provisions are applicable to webs of flexural members subject to concentrated loads or reactions, or the components thereof, acting perpendicular to the longitudinal axis of the member, and in the plane of the web under consideration, and causing compressive stresses in the web.

To avoid crippling of unreinforced flat webs of flexural members having a flat width ratio, h/t, equal to or less than 200, the required strength for the concentrated loads and reactions shall not exceed the values of $\phi_w P_n$, with $\phi_w = 0.75$ and 0.80 for single unreinforced webs and I-sections, respectively, and P_n given in Table C3.4-1. Webs of flexural members for which h/t is greater than 200 shall be provided with adequate means of transmitting concentrated loads and/or reactions directly into the webs.

The formulas in Table C3.4-1 apply to beams when $R/t \leq 6$ and to deck when $R/t \leq 7$, $N/t \leq 210$ and $N/h \leq 3.5$.

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

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

TABLE C3.4-1
P_n

		Shapes Having Single Webs		I-Sections or Similar Sections(1)
		Stiffened or Partially Stiffened Flanges	Unstiffened Flanges	Stiffened, Partially Stiffened and Unstiffened Flanges
Opposing Loads Spaced > 1.5h(2)	End Reaction(3)	*Eq.* C3.4-1	*Eq.* C3.4-2	*Eq.* C3.4-3
	Interior Reaction(4)	*Eq.* C3.4-4	*Eq.* C3.4-4	*Eq.* C3.4-5
Opposing Loads Spaced ≤ 1.5h(5)	End Reaction(3)	*Eq.* C3.4-6	*Eq.* C3.4-6	*Eq.* C3.4-7
	Interior Reaction(4)	*Eq.* C3.4-8	*Eq.* C3.4-8	*Eq.* C3.4-9

Footnotes and Equation References to Table C3.4-1:

(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 C3.4-1:

$t^2 kC_3C_4C_\theta[331 - 0.61(h/t)]\ [1 + 0.01(N/t)]$ (*Eq.* C3.4-1)

$t^2 kC_3C_4C_\theta[217 - 0.28(h/t)]\ [1 + 0.01(N/t)]$ (*Eq.* C3.4-2)

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

$t^2F_yC_6\left(10.0 + 1.25\sqrt{N/t}\right)$ (*Eq.* C3.4-3)

$t^2 kC_1C_2C_\theta[538 - 0.74(h/t)]\ [1 + 0.007(N/t)]$ (*Eq.* C3.4-4)

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

$t^2F_yC_5(0.88+0.12m)\left(15.0 + 3.25\sqrt{N/t}\right)$ (*Eq.* C3.4-5)

$t^2 kC_3C_4C_\theta[244 - 0.57(h/t)]\ [1 + 0.01(N/t)]$ (*Eq.* C3.4-6)

$t^2F_yC_8(0.64+0.31m)\left(10.0 + 1.25\sqrt{N/t}\right)$ (*Eq.* C3.4-7)

$t^2 kC_1C_2C_\theta[771 - 2.26(h/t)]\ [1 + 0.0013(N/t)]$ (*Eq.* C3.4-8)

$t^2F_yC_7(0.82+0.15m)\left(15.0 + 3.25\sqrt{N/t}\right)$ (*Eq.* C3.4-9)

In the above-referenced formulas:

ϕ_w = Resistance factor for web crippling

P_n = Nominal strength for concentrated load or reaction per web

$C_1 = (1.22 - 0.22k)$ (*Eq.* C3.4-10)

$C_2 = (1.06 - 0.06R/t) \le 1.0$ (*Eq.* C3.4-11)

$C_3 = (1.33 - 0.33k)$ (*Eq.* C3.4–12)

$C_4 = (1.15 - 0.15R/t) \leq 1.0$ but not less than 0.50 (*Eq.* C3.4–13)

$C_5 = (1.49 - 0.53k) \geq 0.6$ (*Eq.* C3.4–14)

$C_6 = 1 + \left(\frac{h/t}{750}\right)$ when $h/t \leq 150$ (*Eq.* C3.4–15)

$= 1.20$, when $h/t > 150$ (*Eq.* C3.4–16)

$C_7 = 1/k$, when $h/t \leq 66.5$ (*Eq.* C3.4–17)

$= \left[1.10 - \frac{h/t}{665}\right]\frac{1}{k}$, when $h/t > 66.5$ (*Eq.* C3.4–18)

$C_8 = \left[0.98 - \frac{h/t}{865}\right]\frac{1}{k}$ (*Eq.* C3.4–19)

$C_\theta = 0.7 + 0.3\,(\theta/90)^2$ (*Eq.* C3.4–20)

F_y = Design yield stress of the web, see Section A5.2.1

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

$k = F_y/33$ (*Eq.* C3.4–21)

$m = t/0.075$ (*Eq.* C3.4–22)

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 $\geq 45°$, but not more than $90°$

C3.5 Combined Bending and Web Crippling Strength

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:

$$1.07\left(\frac{P_u}{\phi_w P_n}\right) + \left(\frac{M_u}{\phi_b M_{nxo}}\right) \leq 1.42 \quad \text{(Eq. C3.5–1)}$$

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);

$$0.82\left(\frac{P_u}{\phi_w P_n}\right) + \left(\frac{M_u}{\phi_b M_{nxo}}\right) \leq 1.32 \quad \text{(Eq. C3.5–2)}$$

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

In the above formulas:

ϕ_b = Resistance factor for bending (See Section 3.1)

ϕ_w = Resistance factor for web crippling (See Section C3.4)

P_u = Required strength for the concentrated load or reaction in the presence of bending moment

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

M_u = Required flexural strength at, or immediately adjacent to, the point of application of the concentrated load or reaction P_u

M_{nxo} = Nominal flexural strength about the centroidal x–axis determined in accordance with Section C3.1 excluding the provisions of Section C3.1.2

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

t = Thickness of the web or flange

λ = Slenderness factor given by Section B2.1

C4 Concentrically Loaded Compression Members

This section applies to 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 this section.

(a) The design axial strength, $\phi_c P_n$, shall be calculated as follows:

$\phi_c = 0.85$

$P_n = A_e F_n$ *(Eq. C4–1)*

where

A_e = Effective area at the stress F_n. For sections with circular holes, A_e shall be determined according to Section B2.2a, subject to the limitations of that section. If the number of holes in the effective length region times the hole diameter divided by the effective length does not exceed 0.015, A_e can be determined ignoring the holes.

F_n is determined as follows:

For $F_e > F_y/2$ $\quad F_n = F_y(1 - F_y/4F_e)$ *(Eq. C4–2)*

For $F_e \le F_y/2$ $\quad F_n = F_e$ *(Eq. C4–3)*

F_e is the least of the elastic flexural, torsional and torsional–flexural buckling stress determined according to Sections C4.1 through C4.3.

(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:

$$P_n = \frac{A\pi^2 E}{25.7(w/t)^2} \qquad (Eq.\ C4\text{–}4)$$

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 required axial strength, P_u, acting simultaneously with a moment equal to $P_u L/1000$ 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.

C4.1 Sections Not Subject to Torsional or Torsional-Flexural Buckling

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:

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \qquad (Eq.\ C4.1\text{-}1)$$

where

E = Modulus of elasticity

K = Effective length factor*

L = Unbraced length of member

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

C4.2 Doubly- or Singly-Symmetric Sections Subject to Torsional or Torsional-Flexural Buckling

For sections subject to torsional or torsional-flexural buckling, F_e shall be taken as the smaller of F_e calculated according to Section C4.1 and F_e calculated as follows:

$$F_e = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\right] \qquad (Eq.\ C4.2\text{-}1)$$

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

$$F_e = \frac{\sigma_t \sigma_{ex}}{\sigma_t + \sigma_{ex}} \qquad (Eq.\ C4.2\text{-}2)$$

where σ_t and σ_{ex} are as defined in C3.1.2(a) :

$$\beta = 1 - (x_o/r_o)^2 \qquad (Eq.\ C4.2\text{-}3)$$

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

C4.3 Nonsymmetric Sections

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. Alternatively, compression members composed of such shapes may be tested in accordance with Chapter F.

C5 Combined Axial Load and Bending

The required strengths P_u, M_{ux}, and M_{uy} shall satisfy the following interaction equations:

$$\frac{P_u}{\phi_c P_n} + \frac{C_{mx} M_{ux}}{\phi_b M_{nx} \alpha_{nx}} + \frac{C_{my} M_{uy}}{\phi_b M_{ny} \alpha_{ny}} \le 1.0 \qquad (Eq.\ C5\text{-}1)$$

$$\frac{P_u}{\phi_c P_{no}} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \le 1.0 \qquad (Eq.\ C5\text{-}2)$$

* In frames where lateral stability is provided by diagonal bracing, shear walls, attachment to an adjacent structure having adequate lateral stability, or floor slabs or roof decks secured horizontally by walls or bracing systems parallel to the plane of the frame, and in trusses, the effective length factor, K, for compression members which do not depend upon their own bending stiffness for lateral stability of the frame or truss, shall be taken as unity, unless analysis shows that a smaller value may be used. In a frame which depends upon its own bending stiffness for lateral stability, the effective length, KL, of the compression members shall be determined by a rational method and shall not be less than the actual unbraced length.

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

$$\frac{P_u}{\phi_c P_n}+\frac{M_{ux}}{\phi_b M_{nx}}+\frac{M_{uy}}{\phi_b M_{ny}} \le 1.0 \qquad (Eq.\ C5\text{–}3)$$

where

P_u = Required axial strength

M_{ux} and M_{uy} = Required flexural strengths with respect to the centroidal axes of the effective section determined for the required axial strength alone. For angle sections, M_{uy} shall be taken either as the required flexural strength or the required flexural strength plus $P_uL/1000$, whichever results in a lower value of P_n.

P_n = Nominal axial strength determined in accordance with Section C4

P_{no} = Nominal axial strength determined in accordance with Section C4, with $F_n = F_y$

M_{nx} and M_{ny} = Nominal flexural strengths about the centroidal axes determined in accordance with Section C3

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

$$= 1\Big/\left[1-\frac{P_u}{\phi_c P_E}\right] \qquad (Eq.\ C5\text{–}4)$$

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

ϕ_c = 0.85

$$P_E = \frac{\pi^2 E I_b}{(K_b L_b)^2} \qquad (Eq.\ C5\text{–}5)$$

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

   $$C_m = 0.6 - 0.4\,(M_1/M_2) \qquad (Eq.\ C5\text{–}6)$$

   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, $C_m = 1.0$.

C6 Cylindrical Tubular Members

The requirements of this Section apply to cylindrical tubular members having a ratio of outside diameter to wall thickness, D/t, not greater than $0.441\ E/F_y$.

C6.1 Bending

For flexural members, the required flexural strength uncoupled from axial load, shear, and local concentrated forces or reactions shall not exceed $\phi_b M_n$, where $\phi_b = 0.95$ and M_n is calculated as follows:

For $D/t \leq 0.070\ E/F_y$

$$M_n = 1.25\ F_y S_f \qquad (Eq.\ C6.1\text{–}1)$$

For $0.070\ E/F_y < D/t \leq 0.319\ E/F_y$

$$M_n = \left[0.970 + 0.020\left(\frac{E/F_y}{D/t}\right)\right] F_y S_f \qquad (Eq.\ C6.1\text{–}2)$$

For $0.319\ E/F_y < D/t \leq 0.441\ E/F_y$

$$M_n = [0.328E/(D/t)]S_f \qquad (Eq.\ C6.1\text{–}3)$$

where

S_f = Elastic section modulus of the full, unreduced cross section

C6.2 Compression

The requirements of this Section apply to members in which the resultant of all loads and moments acting on the member is equivalent to a single force in the direction of the member axis passing through the centroid of the section.

The design axial strength, $\phi_c P_n$, shall be calculated as follows:

ϕ_c = 0.85

$$P_n = F_n A_e \qquad (Eq.\ C6.2\text{–}1)$$

In the above equation,

For $F_e > F_y/2$

F_n = Flexural buckling stress

$$= F_y\ [1 - F_y/4F_e] \qquad (Eq.\ C6.2\text{–}2)$$

F_e = The elastic flexural buckling stress determined according to Section C4.1

$$A_e = [1 - (1 - R^2)(1 - A_o/A)]A \qquad (Eq.\ C6.2\text{–}3)$$

$$R = \sqrt{F_y / 2F_e} \qquad (Eq.\ C6.2\text{–}4)$$

$$A_o = \left[\frac{0.037}{\frac{DF_y}{tE}} + 0.667\right] A \leq A \text{ for } \frac{D}{t} \leq 0.441 \frac{E}{F_y} \qquad (Eq.\ C6.2\text{–}5)$$

A = Area of the unreduced cross section

For $F_e \leq F_y/2$

$F_n = F_e$

$A_e = A$

C6.3 Combined Bending and Compression

Combined bending and compression shall satisfy the provisions of Section C5.

D. STRUCTURAL ASSEMBLIES

D1 Built–Up Sections

D1.1 I – Sections Composed of Two Channels

The maximum permissible longitudinal spacing of welds or other connectors, s_{max}, joining two channels to form an I–section shall be

(a) For compression members:

$$s_{max} = \frac{L r_{cy}}{2 r_I} \qquad (Eq.\ D1.1\text{–}1)$$

where

L = Unbraced length of compression member

r_I = Radius of gyration of the I–section about the axis perpendicular to the direction in which buckling would occur for the given conditions of end support and intermediate bracing

r_{cy} = Radius of gyration of one channel about its centroidal axis parallel to the web

(b) For flexural members:

$$s_{max} = L/6 \qquad (Eq.\ D1.1\text{–}2)$$

In no case shall the spacing exceed the value

$$s_{max} = \frac{2 g T_s}{mq} \qquad (Eq.\ D1.1\text{–}3)$$

where

L = Span of beam

T_s = Design strength of connection in tension (Section E)

g = Vertical distance between the two rows of connections nearest to the top and bottom flanges

q = Intensity of factored load on the beam (For methods of determination, see below)

m = Distance from the shear center of one channel to the mid–plane of its web. For simple channels without stiffening lips at the outer edges,

$$m = \frac{w_f^2}{2w_f + d/3} \qquad (Eq.\ D1.1\text{–}4)$$

For channels with stiffening lips at the outer edges,

$$m = \frac{w_f d t}{4 I_x}\left[w_f d + 2D\left(d - \frac{4D^2}{3d}\right)\right] \qquad (Eq.\ D1.1\text{–}5)$$

w_f = Projection of flanges from the inside face of the web (For channels with flanges of unequal width, w_f shall be taken as the width of the wider flange)

d = Depth of channel or beam

D = Overall depth of lip

I_x = Moment of inertia of one channel about its centroidal axis normal to the web.

The intensity of factored load, q, is obtained by dividing the magnitude of factored concentrated loads or reactions by the length of bearing. For beams designed for a uniformly distributed load, q shall be taken equal to three times the intensity of the uniformly distributed factored load. If the length of bearing of a concentrated load or reaction is smaller than the weld spacing, s, the required design strength of the welds or connections closest to the load or reaction is

$$T_s = P_s m / 2g \qquad (Eq.\ D1.1\text{–}6)$$

where P_s is a concentrated load or reaction based on factored loads.

The required maximum spacing of connections, s_{max}, depends upon the intensity of the factored load directly at the connection. Therefore, if uniform spacing of connections is used over the whole length of the beam, it shall be determined at the point of maximum local load intensity. In cases where this procedure would result in uneconomically close spacing, either one of the following methods may be adopted: (a) the connection spacing may be varied along the beam according to the variation of the load intensity; or (b) reinforcing cover plates may be welded to the flanges at points where concentrated loads occur. The design shear strength of the connections joining these plates to the flanges shall then be used for T_s, and g shall be taken as the depth of the beam.

D1.2 Spacing of Connections in Compression Elements

The spacing, s, in the line of stress, of welds, rivets, or bolts connecting a cover plate, sheet, or a non–integral stiffener in compression to another element shall not exceed

(a) that which is required to transmit the shear between the connected parts on the basis of the design strength per connection specified elsewhere herein; nor

(b) $1.16t\sqrt{(E/f_c)}$, where t is the thickness of the cover plate or sheet, and f_c is the stress at service load in the cover plate or sheet; nor

(c) three times the flat width, w, of the narrowest unstiffened compression element tributary to the connections, but need not be less than $1.11t\sqrt{(E/F_y)}$ if $w/t < 0.50\sqrt{(E/F_y)}$, or $1.33t\sqrt{(E/F_y)}$ if $w/t \geq 0.50\sqrt{(E/F_y)}$, unless closer spacing is required by (a) or (b) above.

In the case of intermittent fillet welds parallel to the direction of stress, the spacing shall be taken as the clear distance between welds, plus one–half inch. In all other cases, the spacing shall be taken as the center–to–center distance between connections.

Exception: The requirements of this Section do not apply to cover sheets which act only as sheathing material and are not considered as load–carrying elements.

D2 Mixed Systems

The design of members in mixed systems using cold–formed steel components in conjunction with other materials shall conform to this Specification and the applicable Specification of the other material.

D3 Lateral Bracing

Braces shall be designed to restrain lateral bending or twisting of a loaded beam or column, and to avoid local crippling at the points of attachment.

D3.1 Symmetrical Beams and Columns

Braces and bracing systems, including connections, shall be designed considering strength and stiffness requirements.

D3.2 Channel–Section and Z–Section Beams

The following provisions for bracing to restrain twisting of channels and Z–sections used as beams loaded in the plane of the web, apply only when (a) the top flange is connected to deck or sheathing material in such a manner as to effectively restrain lateral deflection of the connected flange*, or (b) neither flange is so connected. When both flanges are so connected, no further bracing is required.

* Where the Specification does not provide an explicit method for design, further information should be obtained from the Commentary.

D3.2.1 Anchorage of Bracing for Roof Systems Under Gravity Load With Top Flange Connected to Sheathing

For channels and Z–sections designed according to Section C3.1.1, and having deck or sheathing fastened directly to the top flanges in such a manner shown to effectively inhibit relative movement between the deck or sheathing and the purlin flange, provisions shall be made to restrain the flanges so that the maximum top flange lateral displacements with respect to the purlin reaction points do not exceed the span length divided by 360. If the top flanges of all purlins face in the same direction, anchorage of the restraint system must be capable of satisfying the requirements of Sections D3.2.1(a) and D3.2.1(b). If the top flanges of adjacent lines of purlins face in opposite directions, the provisions of Section D3.2.1(a) and D3.2.1(b) do not apply.

Anchored braces need to be connected to only one line of purlins in each purlin bay of each roof slope if provision is made to transmit forces from other purlin lines through the roof deck and its fastening system. Anchored braces shall be as close as possible to the flange which is connected to the deck or sheathing. Anchored braces shall be provided for each purlin bay.

For bracing arrangements other than those covered in Sections D3.2.1(a) and D3.2.1(b), tests in accordance with Chapter F shall be performed so that the type and/or spacing of braces selected are such that the test strength of the braced Z–section assembly is equal to or greater than its nominal flexural strength, instead of that required by Chapter F.

(a) Channel Sections

For roof systems using channel sections for purlins with all compression flanges facing in the same direction, a restraint system capable of resisting 0.05W, in addition to other loading, shall be provided where W is the factored load supported by all purlin lines being restrained. Where more than one brace is used at a purlin line, the restraint force 0.05W shall be divided equally between all braces.

(b) Z–Sections

For roof systems having a diaphragm stiffness of at least 2,000 lb/in., having four to twenty Z–purlin lines with all top flanges facing in the direction of the upward roof slope, and with restraint braces at the purlin supports, midspan or one–third points, each brace shall be designed to resist a force determined as follows:

(1) Single–Span System with Restraints at the Supports:

$$P_L = 0.5\left[\frac{0.220b^{1.50}}{n_p^{0.72}d^{0.90}t^{0.60}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{–}1)$$

(2) Single–Span System with Third–Point Restraints:

$$P_L = 0.5\left[\frac{0.474b^{1.22}}{n_p^{0.57}d^{0.89}t^{0.33}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{–}2)$$

(3) Single–Span System with Midspan Restraint:

$$P_L = \left[\frac{0.224b^{1.32}}{n_p^{0.65}d^{0.83}t^{0.50}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{–}3)$$

(4) Multiple–Span System with Restraints at the Supports:

$$P_L = C_{tr}\left[\frac{0.053b^{1.88}L^{0.13}}{n_p^{0.95}d^{1.07}t^{0.94}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{-}4)$$

with

C_{tr} = 0.63 for braces at end supports of multiple-span systems
C_{tr} = 0.87 for braces at the first interior supports
C_{tr} = 0.81 for all other braces

(5) Multiple-Span System with Third-Point Restraints:

$$P_L = C_{th}\left[\frac{0.181b^{1.15}L^{0.25}}{n_p^{0.54}d^{1.11}t^{0.29}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{-}5)$$

with

C_{th} = 0.57 for outer braces in exterior spans
C_{th} = 0.48 for all other braces

(6) Multiple-Span System with Midspan Restraints:

$$P_L = C_{ms}\left[\frac{0.116b^{1.32}L^{0.18}}{n_p^{0.70}d^{1.00}t^{0.50}} - \sin\theta\right]W \qquad (Eq.\ D3.2.1\text{-}6)$$

with

C_{ms} = 1.05 for braces in exterior spans
C_{ms} = 0.90 for all other braces

where

b = Flange width, in.
d = Depth of section, in.
t = Thickness, in.
L = Span length, in.
θ = Angle between the vertical and the plane of the web of the Z-section, degrees
n_p = Number of parallel purlin lines
W = Total factored load supported by the purlin lines between adjacent supports, pounds

The force, P_L, is positive when restraint is required to prevent movement of the purlin flanges in the upward roof slope direction.

For systems having less than four purlin lines, the brace force can be determined by taking 1.1 times the force found from Equations D3.2.1-1 through D3.2.1-6, with $n_p = 4$. For systems having more than twenty purlin lines, the brace force can be determined from Equations D3.2.1-1 through D3.2.1-6, with $n_p = 20$.

D3.2.2 Neither Flange Connected to Sheathing

Each intermediate brace, at the top and bottom flange, shall be designed to resist a required lateral force, P_L, determined as follows:

(a) For uniform loads, P_L = 1.5K′ times the factored load within a distance 0.5a each side of the brace.

(b) For concentrated loads, P_L = 1.0K′ times each concentrated load within a distance 0.3a each side of the brace, plus 1.4K′ (1−x/a) times each factored concentrated load located farther than 0.3a but not farther than 1.0a from the brace.

In the above formulas:

For channels and Z–sections:

x = Distance from the concentrated load to the brace

a = Distance between center line of braces

For channels:

$$K' = m/d \qquad (Eq.\ D3.2.2\text{–}1)$$

where

m = Distance from the shear center to the mid–plane of the web, as specified in Section D1.1

d = Depth of channel

For Z–sections:

$$K' = I_{xy}/I_x \qquad (Eq.\ D3.2.2\text{–}2)$$

where

I_{xy} = Product of inertia of the full section about centroidal axes parallel and perpendicular to the web

I_x = Moment of inertia of the full section about the centroidal axis perpendicular to the web

Braces shall be designed to avoid local crippling at the points of attachment to the member.

Braces shall be attached both to the top and bottom flanges of the sections, at the ends and at intervals not greater than one–quarter of the span length, in such a manner as to prevent tipping at the ends and lateral deflection of either flange in either direction at intermediate braces. If one–third or more of the total factored load on the beam is concentrated over a length of one–twelfth or less of the span of the beam, an additional brace shall be placed at or near the center of this loaded length.

Exception: When all loads and reactions on a beam are transmitted through members which frame into the section in such a manner as to effectively restrain the section against rotation and lateral displacement, no other braces will be required.

D3.3 Laterally Unbraced Box Beams

For closed box–type sections used as beams subject to bending about the major axis, the ratio of the laterally unsupported length to the distance between the webs of the section shall not exceed $0.086\ E/F_y$.

D4 Wall Studs and Wall Stud Assemblies

The design strength of a stud may be computed on the basis of Section C (neglecting sheathing and using steel only) or on the basis that sheathing (attached to one or both sides of the stud) furnishes adequate lateral and rotational support to the stud in the plane of the wall, provided that the stud, sheathing, and attachments comply with the following requirements:

Both ends of the stud shall be braced to restrain rotation about the longitudinal stud axis and horizontal displacement perpendicular to the stud axis; however, the ends may or may not be free to rotate about both axes perpendicular to the stud axis. The sheathing shall be connected to the top and bottom members of the wall assembly to enhance the restraint provided to the stud and stabilize the overall assembly.

When sheathing is utilized for stability of the wall studs, the sheathing shall retain adequate strength and stiffness for the expected service life of the wall and additional bracing shall be provided as required for adequate structural integrity during construction and in the completed structure.

The equations given are based on solid–web steel studs and are applicable within the following limits:

Yield point, $F_y \leq$ 50 ksi
Section depth, $d \leq 6.0$ in.
Thickness, $t \leq 0.075$ in.
Overall length, $L \leq 16$ ft.
Stud spacing, B, not less than 12 in. nor greater than 24 in.
Studs with perforations shall be designed using the results of stub column tests and/or rational analysis.

D4.1 Wall Studs in Compression

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing* , the design axial strength, $\phi_c P_n$, shall be calculated as follows:

$\phi_c = 0.85$

$$P_n = A_e F_n \qquad (Eq.\ D4.1\text{–}1)$$

where

ϕ_c = Resistance factor for axial compression
A_e = Effective area determined at F_n
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 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 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

$$\sigma_{CR} = \sigma_{ey} + \overline{Q}_a \qquad (Eq.\ D4.1\text{–}2)$$

$$\sigma_{CR} = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_{tQ}) - \sqrt{(\sigma_{ex} + \sigma_{tQ})^2 - (4\beta\sigma_{ex}\sigma_{tQ})}\right] \qquad (Eq.\ D4.1\text{–}3)$$

(2) Z–Sections

$$\sigma_{CR} = \sigma_t + \overline{Q}_t \qquad (Eq.\ D4.1\text{–}4)$$

$$\sigma_{CR} = \frac{1}{2}\left\{(\sigma_{ex} + \sigma_{ey} + \overline{Q}_a) - \sqrt{\left[(\sigma_{ex} + \sigma_{ey} + \overline{Q}_a)^2 - 4(\sigma_{ex}\sigma_{ey} + \sigma_{ex}\overline{Q}_a - \sigma^2_{exy})\right]}\right\} \qquad (Eq.\ D4.1\text{–}5)$$

(3) I–Sections (doubly–symmetric)

$$\sigma_{CR} = \sigma_{ey} + \overline{Q}_a \qquad (Eq.\ D4.1\text{–}6)$$

$$\sigma_{CR} = \sigma_{ex} \qquad (Eq.\ D4.1\text{–}7)$$

In the above formulas:

$$\sigma_{ex} = \frac{\pi^2 E}{(L / r_x)^2} \qquad (Eq.\ D4.1\text{–}8)$$

$$\sigma_{exy} = (\pi^2 E I_{xy}) / (AL^2) \qquad (Eq.\ D4.1\text{–}9)$$

*Studs with sheathing on one flange only, or with unidentical sheathing on both flanges, or having rotational restraint that is not neglected, or having any combination of the above, shall be designed in accordance with the same basic analysis principles used in deriving the provisions of this Section.

$$\sigma_{ey} = \frac{\pi^2 E}{(L/r_y)^2} \qquad (Eq.\ D4.1\text{–}10)$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{L^2}\right] \qquad (Eq.\ D4.1\text{–}11)$$

$$\sigma_{tQ} = \sigma_t + \overline{Q}_t \qquad (Eq.\ D4.1\text{–}12)$$

$\overline{Q}$ = $\overline{q}$ B = Design shear rigidity for sheathing on both sides of the wall assembly

$\overline{q}$ = Design shear rigidity for sheathing per inch of stud spacing (see Table D4)

B = Stud spacing

$$\overline{Q}_a = \overline{Q}/A \qquad (Eq.\ D4.1\text{–}13)$$

A = Area of full unreduced cross section

L = Length of stud

$$\overline{Q}_t = \left(\overline{Q}d^2\right)/\left(4Ar_o^2\right) \qquad (Eq.\ D4.1\text{–}14)$$

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, $\overline{\gamma}$. The shear strain, γ, shall be determined as follows:

$$\gamma = (\pi/L)\,[C_1 + (E_1\, d/2)] \qquad (Eq.\ D4.1\text{–}15)$$

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 and C–Sections

$$C_1 = (F_n C_o)/(\sigma_{ey} - F_n + \overline{Q}_a) \qquad (Eq.\ D4.1\text{–}16)$$

$$E_1 = \frac{F_n\left[(\sigma_{ex} - F_n)(r_o^2 E_o - x_o D_o) - F_n x_o (D_o - x_o E_o)\right]}{(\sigma_{ex} - F_n) r_o^2 (\sigma_{tQ} - F_n) - (F_n x_o)^2} \qquad (Eq.\ D4.1\text{–}17)$$

(2) Z–Sections

$$C_1 = \frac{F_n\left[C_o(\sigma_{ex} - F_n) - D_o \sigma_{exy}\right]}{(\sigma_{ey} - F_n + \overline{Q}_a)(\sigma_{ex} - F_n) - \sigma_{exy}^2} \qquad (Eq.\ D4.1\text{–}18)$$

$$E_1 = (F_n E_o)/(\sigma_{tQ} - F_n) \qquad (Eq.\ D4.1\text{–}19)$$

(3) I–Sections

$$C_1 = (F_n C_o)/(\sigma_{ey} - F_n + \overline{Q}_a) \qquad (Eq.\ D4.1\text{–}20)$$

$$E_1 = 0$$

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

$$C_o = L/350 \text{ in a direction parallel to the wall} \qquad (Eq.\ D4.1\text{–}21)$$

$$D_o = L/700 \text{ in a direction perpendicular to the wall} \qquad (Eq.\ D4.1\text{–}22)$$

E_o = L/(d × 10,000), rad., a measure of the initial twist of the stud from the initial, ideal, unbuckled shape (Eq. D4.1–23)

If $F_n > 0.5\ F_y$, 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

$$E' = 4EF_n(F_y - F_n)/F_y^2 \qquad (Eq.\ D4.1\text{–}24)$$

$$G' = G\,(E'/E) \qquad (Eq.\ D4.1\text{–}25)$$

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 (see Commentary), or from the small–scale–test values given in Table D4.

TABLE D4
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) $\bar{q} = \bar{q}_o\,(2 - s/12)$ (*Eq.* D4.1–26)

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 (see Commentary).

D4.2 Wall Studs in Bending

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing,* the design flexural strengths are $\phi_b M_{nxo}$ and $\phi_b M_{nyo}$

where

ϕ_b = 0.95 for sections with stiffened or partially stiffened compression flanges
= 0.90 for sections with unstiffened compression flanges

M_{nxo} and M_{nyo} = Nominal flexural strengths about the centroidal axes determined in accordance with Section C3.1, excluding the provisions of Section C3.1.2 (lateral buckling)

D4.3 Wall Studs with Combined Axial Load and Bending

The required axial strength and flexural strength shall satisfy the interaction equations of Section C5 with the following redefined terms:

P_n = Nominal axial strength determined according to Section D4.1

M_{nx} and M_{ny} in Equations C5–1, C5–2 and C5–3 shall be replaced by nominal flexural strengths, M_{nxo} and M_{nyo}, respectively.

* Studs with sheathing on one flange only, or with unidentical sheathing on both flanges, or having rotational restraint that is not neglected, or having any combination of the above, shall be designed in accordance with the same basic analysis principles used in deriving the provisions of this Section.

E. CONNECTIONS AND JOINTS

E1 General Provisions

Connections shall be designed to transmit the maximum forces resulting from the factored loads acting on the connected member. Proper regard shall be given to eccentricity.

E2 Welded Connections

The following LRFD design criteria govern welded connections used for cold–formed steel structural members in which the thickness of the thinnest connected part is 0.18 in. or less. For welded connections in which the thickness of the thinnest connected part is greater than 0.18 in., refer to the AISC's "Load and Resistance Factor Design Specification for Structural Steel Buildings".

Except as modified herein, arc welds on steel where at least one of the connected parts is 0.18 inch or less in thickness shall be made in accordance with the AWS D–1.3 (Reference 3 of Section A6) and its Commentary. Welders and welding procedures shall be qualified as specified in AWS D1.3. These provisions are intended to cover the welding positions as shown in Table E2.

Resistance welds shall be made in conformance with the procedures given in AWS C1.1, "Recommended Practices for Resistance Welding" or AWS C1.3, "Recommended Practice for Resistance Welding Coated Low Carbon Steels."

TABLE E2

Connection	Welding Position					
	Square Groove Butt Weld	Arc Spot Weld	Arc Seam Weld	Fillet Weld, Lap or T	Flare–Bevel Groove	Flare–V Groove Weld
Sheet to Sheet	F H V OH	— — — —	F H — —	F H V OH	F H V OH	F H V OH
Sheet to Supporting Member	— — — —	F — — —	F — — —	F H V OH	F H V OH	— — — —

(F = flat, H = horizontal, V = vertical, OH = overhead)

The required strength on each weld shall not exceed the design strength, ϕP_n, where

ϕ = Resistance factor for arc welded connections defined in Sections E2.1 through E2.5.

P_n = Nominal strength of welds determined according to Sections E2.1 through E2.5.

E2.1 Groove Welds in Butt Joints

The design strength, ϕP_n, of a groove weld in a butt joint, welded from one or both sides, shall be determined as follows:

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

$\phi = 0.90$

$P_n = Lt_eF_y$ (*Eq.* E2.1–1)

(b) Shear on the effective area

$\phi = 0.80$

$P_n = Lt_e(0.6F_{xx})$; and (*Eq. E2.1–2*)

$\phi = 0.90$

$P_n = Lt_e\left(F_y / \sqrt{3}\right)$ (*Eq.* E2.1–3)

where

ϕ = Resistance factor for welded connections

P_n = Nominal strength of a groove weld

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

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

L = Length of weld

t_e = Effective throat dimension for groove weld

E2.2 Arc Spot Welds

Arc spot welds permitted by this 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, Figures E2.2(A) and E2.2(B), 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.

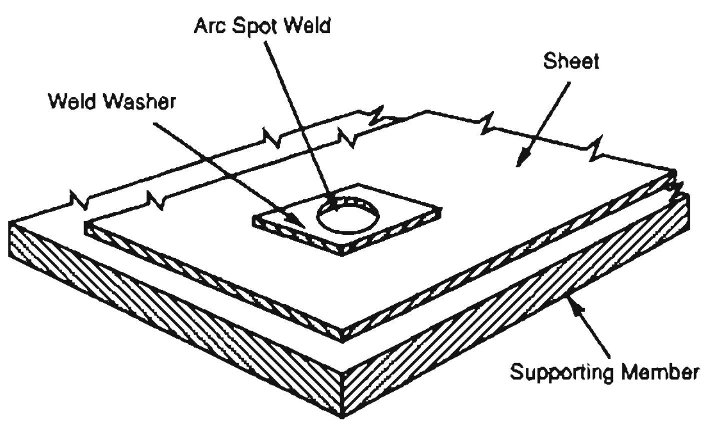


Figure E2.2A Typical Weld Washer

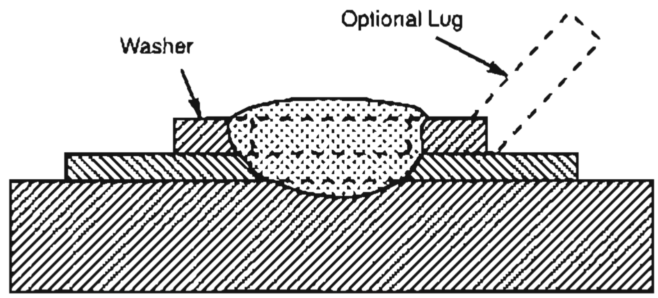


Figure E2.2B Arc Spot Weld Using Washer

Arc spot welds shall be specified by minimum effective diameter of fused area, d_e. Minimum allowable effective diameter is $^3/_8$ inch.

The design 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) $\phi = 0.60$

$P_n = 0.589\, d_e^2\, F_{xx}$; or (*Eq.* E2.2–1)

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

$\phi = 0.60$

$P_n = 2.20\, t\, d_a\, F_u$ (*Eq.* E2.2–2)

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

$\phi = 0.50$

$$P_n = 0.280\left[1 + 5.59\frac{\sqrt{E/F_u}}{d_a/t}\right] t d_a F_u \qquad (Eq.\ E2.2\text{–}3)$$

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

$\phi = 0.50$

$P_n = 1.40\, t\, d_a\, F_u$ (*Eq.* E2.2–4)

where

ϕ = Resistance factor for welded connections

P_n = Nominal 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 = $0.7d - 1.5t$ but $\le 0.55d$ (*Eq.* E2.2–5)

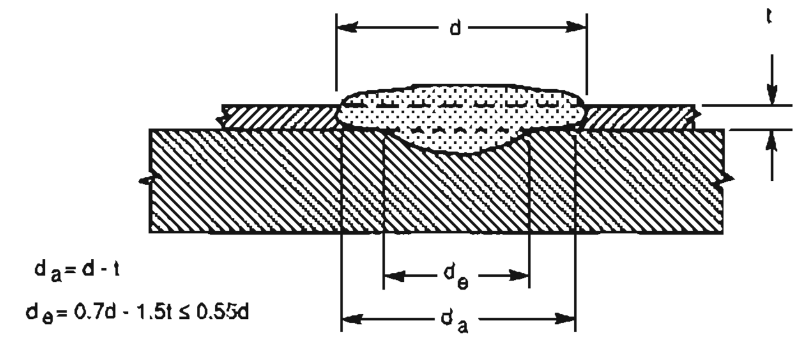
t = Total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer

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

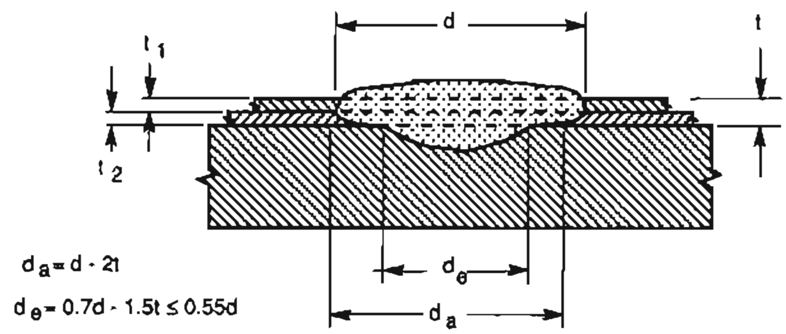
F_u = Tensile strength as specified in Section A3.1 or A3.2 or as reduced for low ductility steel.

Note: See Figures E2.2(C) and E2.2(D) for diameter definitions

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



(C) Arc Spot Weld-Single Thickness of Sheet



(D) Arc Spot Weld-Double Thickness of Sheet

Figure E2.2 C, D Arc Spot Welds

$$e_{min} = \frac{P_u}{\phi F_u t} \qquad (Eq.\ E2.2\text{–}6)$$

where

ϕ = Resistance factor for welded connections
= 0.70 when $F_u/F_{sy} \ge 1.15$
= 0.60 when $F_u/F_{sy} < 1.15$

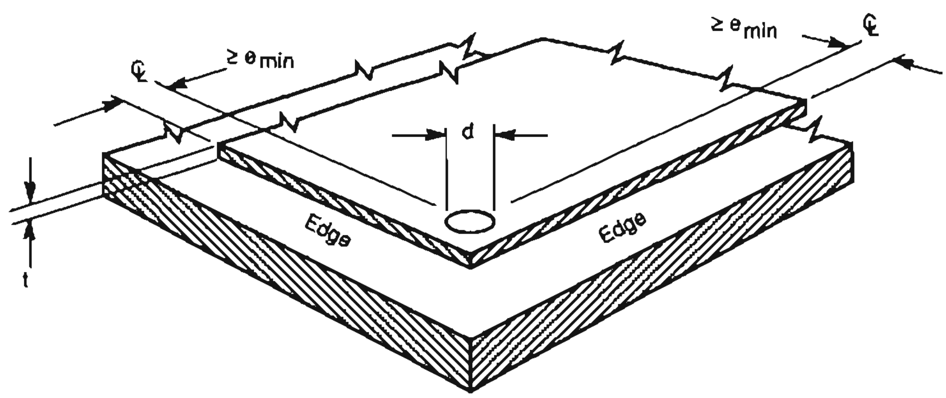
P_u = Required strength transmitted by weld

t = Thickness of thinnest connected sheet

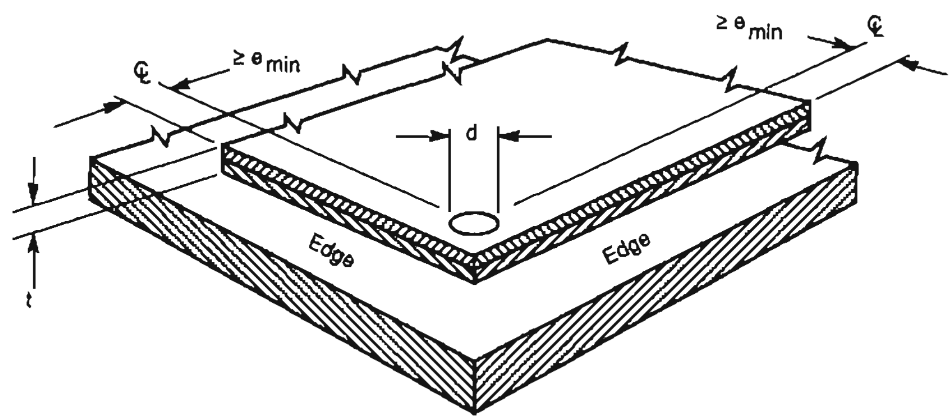
F_{sy} = Yield point as specified in Sections A3.1 or A3.2

Note: See Figures E2.2(E) and E2.2(F) for edge distances of arc welds.

In addition, the distance from the centerline of any weld to the end or boundary of the connected member shall not be less than 1.5d. In no case shall the clear distance between welds and the end of member be less than 1.0d.



(E) Single Sheet



(F) Double Sheet

Figure E2.2 E, F Edge Distances for Arc Spot Welds

The design tensile strength, ϕP_n, on each arc spot weld between sheet and supporting member, shall be determined as follows:

ϕ = 0.65

$$P_n = 0.7\, t\, d_a\, F_u \qquad (Eq.\ E2.2\text{–}7)$$

The following additional limitations for use in Eq. 2.2–7 shall apply:

$e_{min} \geq d$
$F_{xx} \geq 60$ ksi
$F_u \leq 60$ ksi
$t \geq 0.028$ in.

If it can be shown by measurement that a given weld procedure will consistently give a larger effective diameter, d_e, or average diameter, d_a, as applicable, this larger diameter may be used providing the particular welding procedure used for making those welds is followed.

E2.3 Arc Seam Welds

Arc seam welds [Figure E2.3(A)] covered by this 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 design shear strength, ϕP_n, of arc seam welds shall be determined by using the smaller of either

(a) ϕ = 0.60

$$P_n = \left[\frac{\pi d_e^2}{4} + Ld_e\right]0.75F_{xx} \text{ ; or} \qquad (Eq.\ E2.3\text{-}1)$$

(b) ϕ = 0.60

$$P_n = 2.5\, tF_u(0.25L + 0.96\, d_a) \qquad (Eq.\ E2.3\text{-}2)$$

where

ϕ = Resistance factor for welded connections
P_n = Nominal 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

where

$d_a = (d - t)$ for a single sheet, and (*Eq.* E2.3-3)
$(d - 2t)$ for a double sheet (*Eq.* E2.3-4)

d_e = Effective width of arc seam weld at fused surfaces
$d_e = 0.7d - 1.5t$ (*Eq.* E2.3-5)

and F_u and F_{xx} are defined in Section E2.2. The minimum edge distance shall be as determined for the arc spot weld, Section E2.2. See Figure E2.3(B).

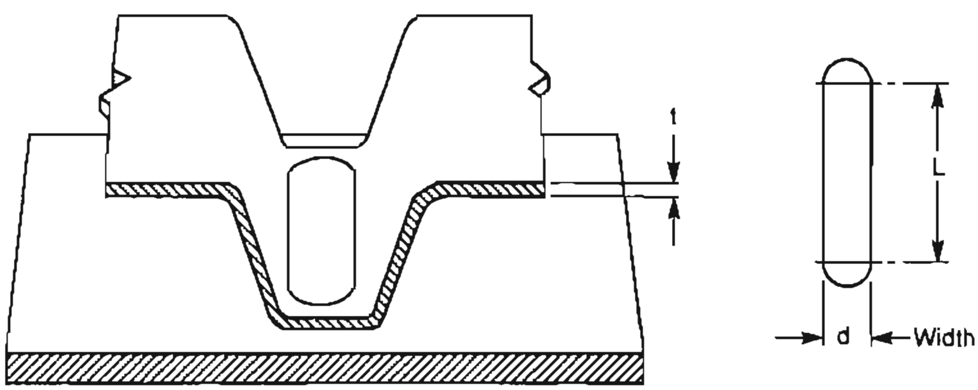


Figure E2.3A Arc Seam Welds - Sheet to Supporting Member in Flat Position

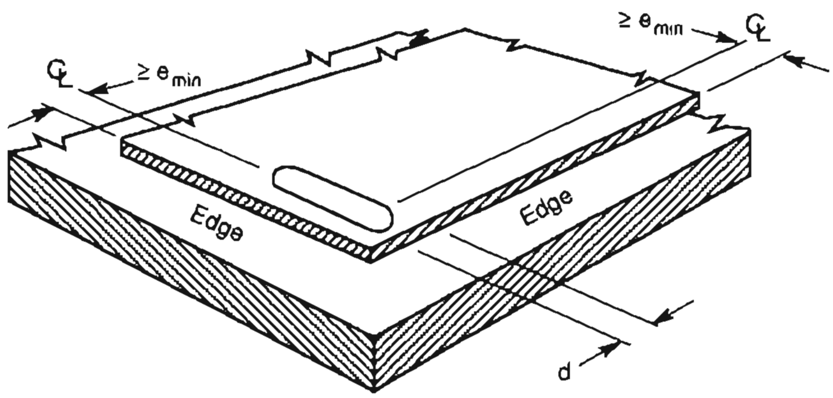


Figure E2.3B Edge Distances for Arc Seam Welds

E2.4 Fillet Welds

Fillet welds covered by this Specification apply to the welding of joints in any position, either

(a) Sheet to sheet, or
(b) Sheet to thicker steel member.

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

(a) For longitudinal loading:
For $L/t < 25$:
$\phi = 0.60$

$$P_n = \left(1 - \frac{0.01L}{t}\right) tLF_u \quad (Eq.\ E2.4\text{–}1)$$

For $L/t \geq 25$:
$\phi = 0.55$

$$P_n = 0.75\, tLF_u \quad (Eq.\ E2.4\text{–}2)$$

(b) For transverse loading:
$\phi = 0.60$

$$P_n = tLF_u \quad (Eq.\ E2.4\text{–}3)$$

where t = Least value of t_1 or t_2, Figure E2.4

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

$\phi = 0.60$

$$P_n = 0.75\, t_w LF_{xx} \quad (Eq.\ E2.4\text{–}4)$$

where

ϕ = Resistance factor for welded connedtions
P_n = Nominal strength of a fillet weld
L = Length of fillet weld
t_w = Effective throat = $0.707\, w_1$ or $0.707\, w_2$, 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 (see Figure E2.4).
F_u and F_{xx} are defined in Section E2.2.

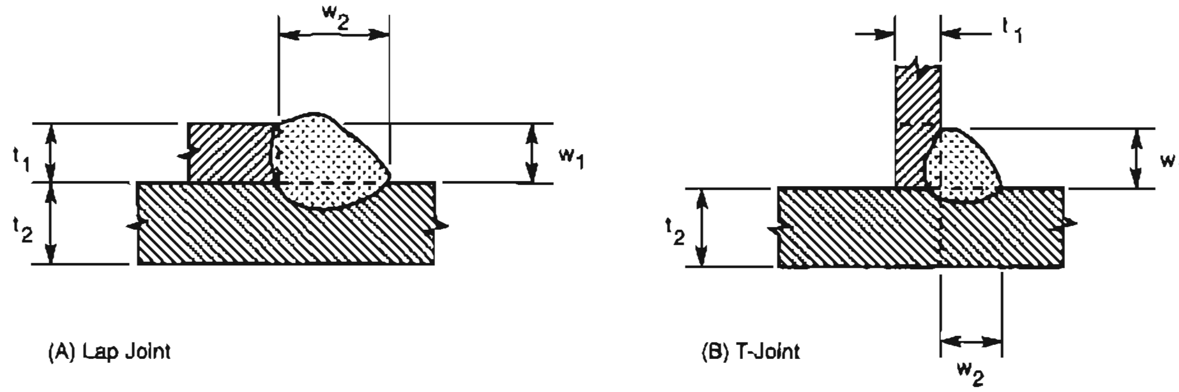


Figure E2.4 Fillet Welds

E2.5 Flare Groove Welds

Flare groove welds covered by this 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 design shear strength, ϕP_n, of a flare groove weld shall be determined as follows:

(a) For flare–bevel groove welds, transverse loading [see Figure E2.5(A)]:

$\phi = 0.55$

$P_n = 0.833tLF_u$ (*Eq.* E2.5–1)

(b) For flare groove welds, longitudinal loading [see Figures E2.5(B), E2.5(C), and E2.5(D)]:

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

$\phi = 0.55$

$P_n = 0.75tLF_u$ (*Eq.* E2.5–2)

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

$\phi = 0.55$

$P_n = 1.50tLF_u$ (*Eq.* E2.5–3)

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

$\phi = 0.60$

$P_n = 0.75t_wLF_{xx}$ (*Eq.* E2.5–4)

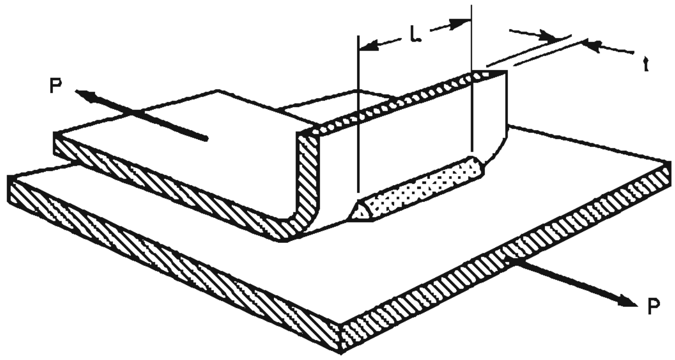
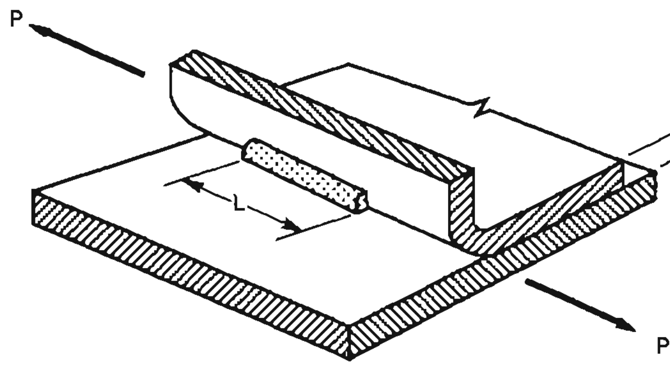
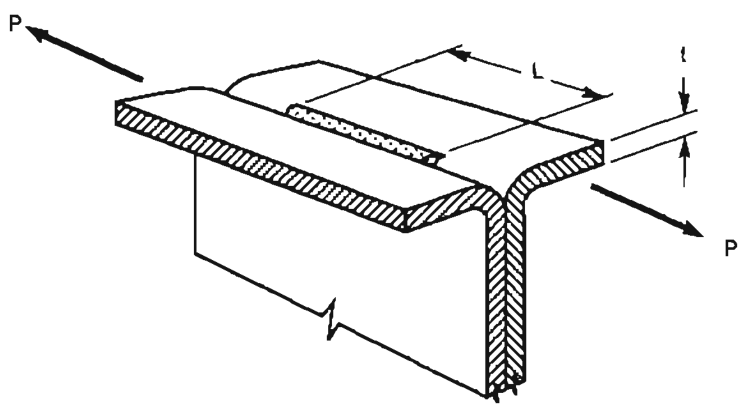


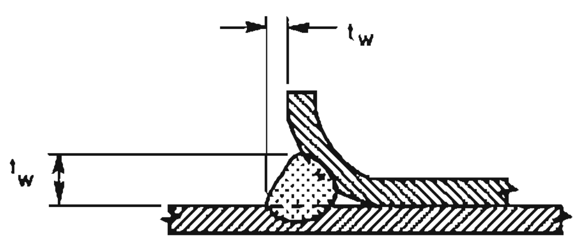
Figure E2.5A Flare–Bevel Groove Weld



(B) Flare Bevel Groove



(C) Flare V-Groove



(D) Throat

Figure E2.5 B, C, D Shear in Flare Groove Welds

E2.6 Resistance Welds

The design shear strength, ϕP_n, of spot welding shall be determined as follows:

ϕ = 0.65

P_n = Tabulated value given in Table E2.6

TABLE E2.6
Nominal Shear Strength Of Spot Welding

Thickness of Thinnest Outside Sheet, in.	Nominal Shear Strength per Spot, kips	Thickness of Thinnest Outside Sheet, in.	Nominal Shear Stength per Spot, kips
0.010	0.13	0.080	3.33
0.020	0.48	0.090	4.00
0.030	1.00	0.100	4.99
0.040	1.42	0.110	6.07
0.050	1.65	0.125	7.29
0.060	2.28	0.190	10.16
0.070	2.83	0.250	15.00

E3 Bolted Connections

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's "Load and Resistance Factor Design Specification for Structural Steel Buildings", September 1, 1986.

Bolts, nuts, and washers shall generally conform to one of the following specifications:

ASTM A194 Carbon and Alloy Steel Nuts for Bolts for High–Pressure and High–Temperature Service

ASTM A307(Type A), Carbon Steel Externally and Internally Threaded Standard Fasteners

ASTM A325 High Strength Bolts for Structural Steel Joints

ASTM A354 (Grade BD), Quenched and Tempered Alloy Steel Bolts, Studs,and Other Externally Threaded Fasteners (for diameter of bolt smaller than $^1/_2$ inch)

ASTM A449 Quenched and Tempered Steel Bolts and Studs (for diameter of bolt smaller than $^1/_2$ inch)

ASTM A490 Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints

ASTM A563 Carbon and Alloy Steel Nuts

ASTM F436 Hardened Steel Washers

ASTM F844 Washers, Steel, Plain (Flat), Unhardened for General Use

ASTM F959 Compressible Washer–Type Direct Tension Indicators for Use with Structural Fasteners

When other than the above are used, drawings shall indicate clearly the type and size of fasteners to be employed and the nominal strength assumed in design.

Bolts shall be installed and tightened to achieve satisfactory performance of the connections involved under usual service conditions.

The holes for bolts shall not exceed the sizes specified in Table E3, except that larger holes may be used in column base details or structural systems connected to concrete walls.

TABLE E3
Maximum Size of Bolt Holes, Inches

Nominal Bolt Diameter, d in.	Standard Hole Diameter, d_h in.	Oversized Hole Diameter, d_h in.	Short–Slotted Hole Dimensions in.	Long–Slotted Hole Dimensions in.
$< 1/2$	$d + 1/32$	$d + 1/16$	$(d + 1/32)$ by $(d + 1/4)$	$(d + 1/32)$ by $(2\,1/2\,d)$
$\geq 1/2$	$d + 1/16$	$d + 1/8$	$(d + 1/16)$ by $(d + 1/4)$	$(d + 1/16)$ by $(2\,1/2\,d)$

Standard holes shall be used in bolted connections, except that oversized and slotted holes may be used as approved by the designer. The length of slotted holes shall be normal to the direction of the shear load. Washers or backup plates shall be installed over oversized or short–slotted holes in an outer ply unless suitable performance is demonstrated by load tests in accordance with Section F.

E3.1 Spacing and Edge Distance

The design shear strength, ϕP_n, of the connected part along two parallel lines in the direction of applied force shall be determined as follows:

$$P_n = teF_u \qquad (Eq.\ E3.1\text{–}1)$$

(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 as specified in Section A3.1 or A3.2 or as reduced for low–ductility steel
F_{sy} = Yield point of the connected part as specified in Section A3.1 or A3.2 or as reduced for low–ductility steel

In addition, the minimum distance between centers of bolt holes shall provide sufficient clearance for bolt heads, nuts, washers and the wrench but shall not be less than 3 times the nominal bolt diameter, d. Also, the distance from the center of any standard hole to the end or other boundary of the connecting member shall not be less than $1\,1/2\,d$.

For oversized and slotted holes, the distance between edges of two adjacent holes and the distance measured from the edge of the hole to the end or other boundary of the connecting member in the line of stress shall not be less than the value of $e-(d_h/2)$, in which e is the required distance computed from the applicable equation given above, and d_h is the diameter of a standard hole defined in Table E3. In no case shall the clear distance between edges of two adjacent holes be less than 2d and the distance between the edge of the hole and the end of the member be less than d.

E3.2 Tension in Connected Part

The design 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

$P_n = (1.0 - 0.9r + 3rd/s)\, F_u A_n \leq F_u A_n$ (*Eq.* E3.2–1)

$\phi = 0.65$ 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

$\phi = 0.65$

$P_n = (1.0 - r + 2.5rd/s)\, F_u A_n \leq F_u A_n$ (*Eq.* E3.2–2)

In addition, the design tensile strength shall not exceed the following values:

$\phi = 0.95$

$P_n = F_y A_n$ (*Eq.* E3.2–3)

where

A_n = Net area of the connected part

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

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

F_u = Tensile strength of the connected part as specified in Section A3.1 or A3.2 or as reduced for low–ductility steel

F_y = Yield point of the connected part

d and t are defined in Section E3.1

E3.3 Bearing

The design bearing strength, ϕP_n, shall be determined by the values of ϕ and P_n given in Tables E3.3–1 and E3.3–2 for the applicable thickness and F_u/F_{sy} ratio of the connected part and the type of joint used in the connection.

In Tables E3.3–1 and E3.3–2, the symbols ϕ, P_n, d, F_u and t were previously defined. For conditions not shown, the design bearing strength of bolted connections shall be determined by tests.

TABLE E3.3–1
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	3.33 F_udt
		< 1.15	0.65	3.00 F_udt
	Single shear and outside sheets of double shear connection	No limit	0.60	3.00 F_udt
≥ $^3/_{16}$	See AISC LRFD Specification			

TABLE E3.3–2
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
≥ 0.036 but < $^3/_{16}$	Inside sheet of double shear connection	≥ 1.15	0.65	3.00 F_udt
	Single shear and outside sheets of double shear connection	≥ 1.15	0.70	2.22 F_udt
≥ $^3/_{16}$	See AISC LRFD Specification			

E3.4 Shear and Tension in Bolts

The required bolt strength in shear or tension shall not exceed the design strength, ϕP_n, determined as follows:

ϕ = Resistance factor given in Table E3.4–1

$P_n = A_b F_n$ (*Eq.* E3.4–1)

where

A_b = Gross cross–sectional area of bolt

F_n is given by F_{nv} or F_{nt} in Table E3.4–1.

TABLE E3.4–1
Nominal Tensile and Shear Strength 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 BD Bolts ($^1/_4$ in. $\leq$ d < $^1/_2$ in.), when threads are not exluded from shear planes		101.0		59.0
A354 Grade BD Bolts ($^1/_4$ in. $\leq$ d < $^1/_2$ in.) when threads are excluded from shear planes		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

* Applies to bolts in holes as limited by Table E3. Washers or back–up plates shall be installed over long–slotted holes and the capacity of connections using long–slotted holes shall be determined by load tests in accordance with Section F.

The pullover strength of the connected sheet at the bolt head, nut or washer should be considered where bolt tension is involved, see Section E5.2.

When bolts are subject to a combination of shear and tension produced by factored loads, the required tension strength shall not exceed the design strength, ϕP_n, based on $\phi = 0.75$ and $P_n = A_b F'_{nt}$, where F'_{nt} is given in Table E3.4–2, in which f_v is the shear stress produced by the same factored loads. The required shear strength shall not exceed the design shear strength, $\phi A_b F_{nv}$, determined in accordance with Table E3.4–1.

TABLE E3.4–2
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	$113 - 2.4f_v \le 90$	$113 - 1.9f_v \le 90$
A354 Grade BD Bolts	$127 - 2.4f_v \le 101$	$127 - 1.9f_v \le 101$
A449 Bolts	$101 - 2.4f_v \le 81$	$101 - 1.9f_v \le 81$
A490 Bolts	$141 - 2.4f_v \le 112.5$	$141 - 1.9f_v \le 112.5$
A307 Bolts, Grade A		
When $1/4$ in. $\le d < 1/2$ in.	$47 - 2.4f_v \le 40.5$	
When $d \ge 1/2$ in.	$52 - 2.4f_v \le 45$	

E4 Shear Rupture

At beam–end connections, where one or more flanges are coped and failure might occur along a plane through the fasteners, the required shear strength shall not exceed the design shear strength, ϕV_n .

where

ϕ = 0.75

$V_n = 0.6 F_u A_{wn}$ (*Eq.* E4–1)

$A_{wn} = (d_{wc} - n d_h)t$ (*Eq.* E4–2)

d_{wc} = Coped web depth

n = Number of holes in the critical plane

d_h = Hole diameter

F_u = Tensile strength as specified in Section A3.1 or A3.2 or as reduced for low–ductility steel

t = Thickness of coped web

E5 Connections to Other Materials

E5.1 Bearing

Proper provisions shall be made to transfer bearing forces resulting from axial loads and moments from steel components covered by the Specification to adjacent structural components made of other materials. The required bearing strength in the contact area shall not exceed the design strength, $\phi_c P_p$.

In the absence of code regulations, the design bearing strength on concrete may be taken as $\phi_c P_p$:

On the full area of a concrete support . . . . $P_p = 0.85f'_c A_1$
On less than the full area of a concrete support . . . . $P_p = 0.85f'_c A_1 \sqrt{A_2 / A_1}$
where
ϕ_c =0.60
f'_c =Specified compression strength of concrete
A_1 =Bearing area
A_2 =Full cross–sectional area of concrete support
The value of $\sqrt{A_2 / A_1}$ shall not exceed 2.

E5.2 Tension

The pull–over shear/tension forces in the steel sheet around the head of the fastener should be considered as well as the pull–out force resulting from factored axial loads and bending moments transmitted onto the fastener from various adjacent structural components in the assembly.

The nominal tensile strength of the fastener and the nominal imbedment strength of the adjacent structural component shall be determined by applicable product code approvals, or product specifications and/or product literature.

E5.3 Shear

Proper provisions shall be made to transfer shearing forces from steel components covered by this Specification to adjacent structural components made of other materials. The required shear and/or bearing strength on the steel components shall not exceed that allowed by this Specification. The design shear strength on the fasteners and other material shall not be exceeded. Imbedment requirements are to be met. Proper provision shall also be made for shearing forces in combination with other forces.

F. TESTS FOR SPECIAL CASES

(a) Tests shall be made by an independent testing laboratory or by a testing laboratory of a manufacturer.

(b) The provisions of Chapter F do not apply to cold–formed steel diaphragms.

F1 Tests for Determining Structural Performance

Where the composition or configuration of elements, assemblies, connections, or details of cold–formed steel structural members are such that calculation of their load–carrying capacity or deflection cannot be made in accordance with the provisions of this Specification, their structural performance shall be established from tests and evaluated in accordance with the following procedure.

(a) Where practicable, evaluation of the test results shall be made on the basis of the average value of test data resulting from tests of not fewer than four identical specimens, provided the deviation of any individual test result from the average value obtained from all tests does not exceed ±10 percent. If such deviation from the average value exceeds 10 percent, at least three more tests of the same kind shall be made. The average value of all tests made shall then be regarded as the predicted capacity, R_p, for the series of the tests. The mean value and the coefficient of variation of the tested–to–predicted load ratios for all tests, P_m and V_p, shall be determined for statistical analysis.

(b) The load–carrying capacity of the tested elements, assemblies, connections, or members shall satisfy Eq. F1–1.

$$\phi R_P \geq \Sigma \gamma_i Q_i \qquad (Eq.\ F1\text{–}1)$$

where

$\Sigma\gamma_i Q_i$ = Required resistance based on the most critical load combination determined in accordance with Section A5.1.4. γ_i and Q_i are load factors and load effects, respectively.

R_P = Average value of all test results

ϕ = Resistance factor

$$= 1.5(M_m F_m P_m)\exp\left(-\beta_o\sqrt{V_M^2 + V_F^2 + C_P V_P^2 + V_Q^2}\right)^* \qquad (Eq.\ F1\text{–}2)$$

M_m = Mean value of the material factor listed in Table F1 for the type of component involved

F_m = Mean value of the fabrication factor listed in Table F1 for the type of component involved

P_m = Mean value of the tested–to–predicted load ratios determined in Section F1(a)

β_o = Target reliability index

= 2.5 for structural members and 3.5 for connections

V_M = Coefficient of variation of the material factor listed in Table F1 for the type of component involved

V_F = Coefficient of variation of the fabrication factor listed in Table F1 for the type of component involved

C_P = Correction factor

$$= (n-1)/(n-3) \qquad (Eq.\ F1\text{–}3)$$

V_P = Coefficient of variation of the tested–to–predicted load ratios determined in Section F1(a)

n = Number of tests

V_Q = Coefficient of variation of the load effect

= 0.21

---

* For beams having tension flange through–fastened to deck or sheathing and with compression flange laterally unbraced, ϕ shall be determined with a coefficient of 1.6 in lieu of 1.5, β_o = 1.5, and V_Q = 0.43.

TABLE F1
Statistical Data for the Determination of Resistance Factor

Type of Component	M_m	V_M	F_m	V_F
Transverse Stiffeners	1.10	0.10	1.00	0.05
Shear Stiffeners	1.00	0.06	1.00	0.05
Tension Members	1.10	0.10	1.00	0.05
Flexural Members				
Bending Strength	1.10	0.10	1.00	0.05
Lateral Buckling Strength	1.00	0.06	1.00	0.05
One Flange Through–Fastened to Deck or Sheathing	1.10	0.10	1.00	0.05
Shear Strength	1.10	0.10	1.00	0.05
Combined Bending and Shear	1.10	0.10	1.00	0.05
Web Crippling Strength	1.10	0.10	1.00	0.05
Combined Bending and Web Crippling	1.10	0.10	1.00	0.05
Concentrically Loaded Compression Members	1.10	0.10	1.00	0.05
Combined Axial Load and Bending	1.05	0.10	1.00	0.05
Cylindrical Tubular Members				
Bending Strength	1.10	0.10	1.00	0.05
Axial Compression	1.10	0.10	1.00	0.05
Wall Studs and Wall Stud Assemblies				
Wall Studs in Compression	1.10	0.10	1.00	0.05
Wall Studs in Bending	1.10	0.10	1.00	0.05
Wall Studs with Combined Axial Load and Bending	1.05	0.10	1.00	0.05

TABLE F1 (Continued)

Statistical Data for the Determination of Resistance Factor

Type of Component	M_m	V_M	F_m	V_F
Welded Connections				
Arc Spot Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Failure	1.10	0.08	1.00	0.15
Arc Seam Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Tearing	1.10	0.10	1.00	0.10
Fillet Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Failure	1.10	0.08	1.00	0.15
Flare Groove Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Failure	1.10	0.10	1.00	0.10
Resistance Welds	1.10	0.10	1.00	0.10
Bolted Connections				
Minimum Spacing and Edge Distance	1.10	0.08	1.00	0.05
Tension Strength on Net Section	1.10	0.08	1.00	0.05
Bearing Strength	1.10	0.08	1.00	0.05

The listing in Table F1 does not exclude the use of other documented statistical data if they are established from sufficient results on material properties and fabrication.*

For steels not listed in Section A3.1, the values of M_m and V_M shall be determined by the statistical analysis for the materials used.

When distortions interfere with the proper functioning of the specimen in actual use, the load effects based on the critical load combination at the occurrence of the acceptable distortion shall also satisfy Eq. F1–1, except that the resistance factor ϕ is taken as unity and that the load factor for dead load may be taken as 1.0.

(c) If the yield point of the steel from which the tested sections are formed is larger than the specified value, the test results shall be adjusted down to the specified minimum yield point of the steel which the manufacturer intends to use. The test results shall not be adjusted upward if the yield point of the test specimen is less than the minimum specified yield point. Similar adjustments shall be made on the basis of tensile strength instead of yield point where tensile strength is the critical factor.

Consideration must also be given to any variation or differences which may exist between the design thickness and the thickness of the specimens used in the tests.

F2 Tests for Confirming Structural Performance

For structural members, connections, and assemblies whose capacities can be computed according to this Specification or its specific references, confirmatory tests may be made to demonstrate the load–carrying capacity not less than the nominal resistance, R_n, specified in this Specification or its specific references for the type of behavior involved.

F3 Tests for Determining Mechanical Properties

F3.1 Full Section

Tests for determination of mechanical properties of full sections to be used in Section A5.2.2 shall be made as specified below:

(a) Tensile testing procedures shall agree with Standard Methods and Definitions for Mechanical Testing of Steel Products, ASTM A370. Compressive yield point determinations shall be made by means of compression tests of short specimens of the section.

(b) The compressive yield stress shall be taken as the smaller value of either the maximum compressive strength of the sections divided by the cross section area or the stress defined by one of the following methods:

  (1) For sharp yielding steel, the yield point shall be determined by the autographic diagram method or by the total strain under load method.

  (2) For gradual yielding steel, the yield point shall be determined by the strain under load method or by the 0.2 percent offset method.

  When the total strain under load method is used, there shall be evidence that the yield point so determined agrees within 5 percent with the yield point which would be determined by the 0.2 percent offset method

(c) Where the principal effect of the loading to which the member will be subjected in service will be to produce bending stresses, the yield point shall be determined for the flanges only. In determining such yield points, each specimen shall consist of one complete flange plus a portion of the web of such flat width ratio that the value of ρ for the specimen is unity.

* See Reference 36 of the Commentary

(d) For acceptance and control purposes, two full section tests shall be made from each lot of not more than 50 tons nor less than 30 tons of each section, or one test from each lot of less than 30 tons of each section. For this purpose a lot may be defined as that tonnage of one section that is formed in a single production run of material from one heat.

(e) At the option of the manufacturer, either tension or compression tests may be used for routine acceptance and control purposes, provided the manufacturer demonstrates that such tests reliably indicate the yield point of the section when subjected to the kind of stress under which the member is to be used.

F3.2 Flat Elements of Formed Sections

Tests for determining mechanical properties of flat elements of formed sections and representative mechanical properties of virgin steel to be used in Section A5.2.2 shall be made in accordance with the following provisions:

The yield point of flats, F_{yf}, shall be established by means of a weighted average of the yield points of standard tensile coupons taken longitudinally from the flat portions of a representative cold–formed member. The weighted average shall be the sum of the products of the average yield point for each flat portion times its cross sectional area, divided by the total area of flats in the cross section. The exact number of such coupons will depend on the shape of the member, i.e., on the number of flats in the cross section. At least one tensile coupon shall be taken from the middle of each flat. If the actual virgin yield point exceeds the specified minimum yield point, the yield point of the flats, F_{yf}, shall be adjusted by multiplying the test values by the ratio of the specified minimum yield point to the actual virgin yield point.

F3.3 Virgin Steel

The following provisions apply to steel produced to other than the ASTM Specifications listed in Section A3.1 when used in sections for which the increased yield point of the steel after cold forming shall be computed from the virgin steel properties according to Section A5.2.2. For acceptance and control purposes, at least four tensile specimens shall be taken from each lot as defined in Section F3.1(d) for the establishment of the representative values of the virgin tensile yield point and ultimate strength. Specimens shall be taken longitudinally from the quarter points of the width near the outer end of the coil.

COMMENTARY ON THE LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD–FORMED STEEL STRUCTURAL MEMBERS

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Prepared by University of Missouri-Rolla
Rolla, Missouri

Preface

This document provides a commentary on the background for the *Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members.*

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COMMENTARY ON THE LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

INTRODUCTION

In the design of steel buildings, the "Allowable Stress Design Criteria" have long been used for the design of cold-formed steel structural members in the United States and other countries. Even though the theoretical concept of reliability analysis has been available for some time and the significance of such a concept in structural safety and design is well recognized, the probabilistic method has not yet been explicitly adopted as a basis for the American design standard for cold-formed steel structures.

Recently, the load and resistance factor design (LRFD) criteria have been developed for steel buildings using hot-rolled shapes and built-up members fabricated from steel plates. It became evident that the development of a new specification for load and resistance factor design of cold-formed steel is highly desirable because the design criteria for heavy hot-rolled steel construction cannot possibly cover the design features of thin-walled, cold-formed steel construction completely.

Since 1976, a joint project has been conducted at University of Missouri-Rolla and Washington University to develop the new design criteria for cold-formed steel structural members and connections based on the probabilistic approach.

The Load and Resistance Factor Design criteria developed on the basis of the 1986 Edition of the AISI Specification with 1989 Addendum for allowable stress design are included in Sections A through F of this Specification.

This commentary contains a brief presentation of the methodology used for the development of the load and resistance factor design criteria. In addition, it provides a record of the reasoning behind, and the justification for, various provisions of the Specification. For detailed background information, reference is made to the research reports given in the bibliography.

A. GENERAL PROVISIONS

A1 Limits of Applicability and Terms

Section A1 of the LRFD Specification is essentially the same as Section A1 of the AISI Specification for allowable stress design. The definitions and various terms used for the LRFD criteria are the same as that used for the allowable stress design.

A2 Non-Conforming Shapes and Constructions

Section A2 of the LRFD Specification is essentially the same as Section A2 of the AISI Specification for allowable stress design.

A3 Material

This Section is essentially the same as Section A3 of the AISI Specification for allowable stress design.

In lieu of the tensile-to-yield strength limit of 1.08, the Specification permits the use of elongation requirements using the measurement technique as given in Ref. 1, and Part VII of the Manual. Because of limited experimental verification of the structural performance of members using material having a tensile-to-yield strength ratio less than 1.08 (Ref. 2), the Specification limits the use of this material to purlins and girts meeting the elastic design requirements of Sections C3.1.1(a), C3.1.2, and C3.1.3. Thus, the use of such steel in other applications (compression members, tension members, other flexural members including those whose strength is based on inelastic reserve capacity, etc.) is prohibited. However, in purlins and girts, concurrent axial loads of relatively small magnitude are acceptable providing the requirements of Section C5 are met and $P_u/\phi_c P_n$ does not exceed 0.15.

A4 Loads

This Section is the same as Section A4 of the AISI Specification for allowable stress design.

With regard to ponding, design guidance can be found from Section K2 of the AISC Load and Resistance Factor Design Specification for Structural Steel Buildings (Ref. 3).

A5 Structural Analysis and Design

A5.1 Design Basis

The current method of designing cold-formed steel structural members, as presented in the 1986 AISI Specification (Ref. 4), is based on the Allowable Stress Design method. In this approach, the forces (bending moments, axial forces, shear forces) in structural members are computed by accepted methods of structural analysis for the specified working loads. These member forces or moments should not exceed the allowable values permitted by the AISI Specification. The AISI allowable load or moment is determined by dividing the nominal load or moment at a limit state by a factor of safety. Usual factors of safety inherent in the AISI Specification for the Design of Cold-Formed Steel Structural Members are 5/3 for beams and 23/12 for columns.

A limit state is the condition at which the structural usefulness of a load-carrying element or member is impaired to such an extent that it becomes unsafe for the occupants of the structure, or the element no longer performs its intended function. Typical limit states for cold-formed steel members are excessive deflection, yielding, buckling and attainment of maximum strength after local buckling (i.e., post-buckling strength). These limit states have been established through experience in practice or in the laboratory, and they have been thoroughly investigated through analytical and experimental research. The background for the establishment of the limit states is extensively documented in the Commentary on the AISI Specification (Refs. 5 and 6)(see also Refs. 7 and 8), and a continuing research effort provides further improvement in understanding them.

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

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

$$\phi R_n \geq \Sigma \gamma_i Q_i \qquad \text{(CA5.1-1)}$$

where

R_n = the nominal resistance

ϕ = resistance factor

γ_i = load factors

Q_i = load effects

The nominal resistance is the strength of the element or member for a given limit state, computed for nominal section properties and for minimum specified material properties according to the appropriate analytical model which defines the strength. The resistance factor ϕ accounts for the uncertainties and variabilities inherent in the R_n, and it is usually less than unity. The load effects Q_i are the forces on the cross section (bending moment, axial force, shear force) determined from the specified minimum loads by structural analysis, and γ_i are the corresponding load factors which account for the uncertainties and variabilities of the loads. The load factors are greater than unity.

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

Probabilistic Concepts

Factors of safety or load factors are provided against the uncertainties and variabilities which are inherent in the design process. Structural design consists of comparing nominal load effects Q to nominal resistances R, but both Q and R are random parameters (see Fig. CA5.1-1). A limit state is violated if $R < Q$. While the possibility of this event ever occurring is never zero, a successful design should, nevertheless, have only an acceptably small probability of exceeding the limit state. If the exact probability distributions of Q and R were known, then the probability of $(R - Q) < 0$ could be exactly determined for any design. In general the distributions of Q and R are not known, and only the means, Q_m and R_m, and the standard deviations, σ_Q and σ_R are available. Nevertheless it is possible to determine relative reliabilities of several designs by the scheme illustrated in Fig. CA5.1-2. The distribution curve shown is for ln(R/Q), and a limit state is exceeded when $\ln(R/Q) \leq 0$. The area under $\ln(R/Q) \leq 0$ is the probability of violating the limit state. The size of this area is dependent on the distance between the origin and the mean of ln(R/Q). For given statistical data R_m, Q_m, σ_R and σ_Q, the area under $\ln(R/Q) \leq 0$ can be varied by changing the value of β (Fig. CA5.1-2), since $\beta\sigma_{\ln(R/Q)} = \ln(R/Q)_m$, from which approximately

$$\beta = \frac{\ln(R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}} \qquad \text{(CA5.1 -2)}$$

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

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

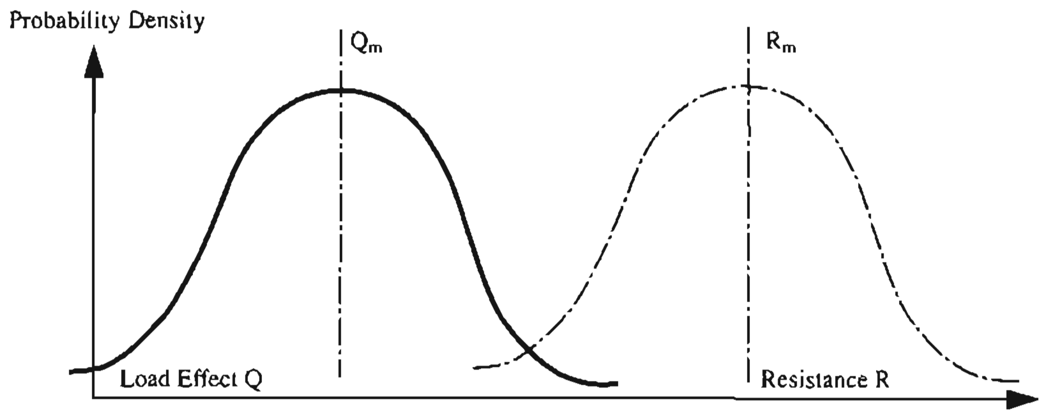

Fig. CA5.1-1 Definition of the Randomness of Q and R

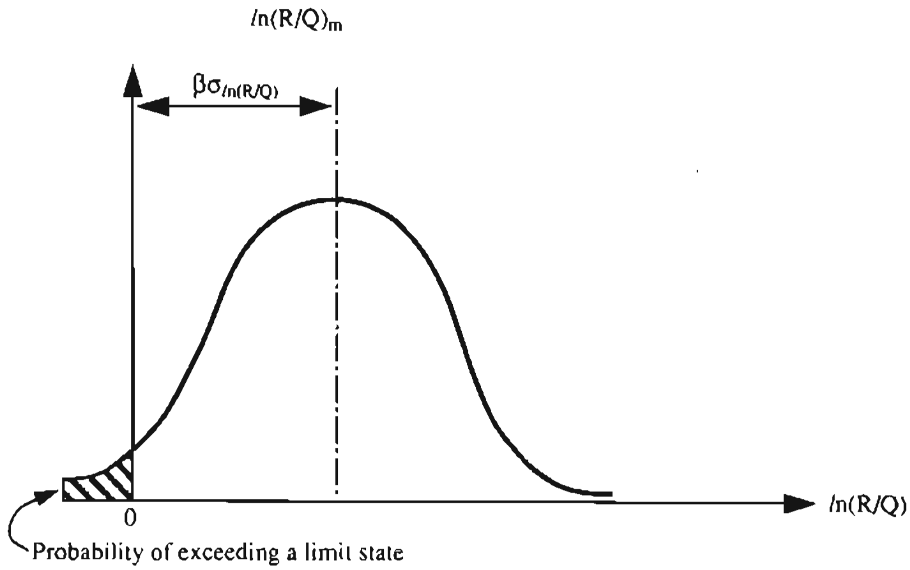

Fig. CA5.1-2 Definition of the Reliability Index β

The design requirement of the 1986 AISI Specification for such a beam is

$$S_eF_y/FS = (L_s^2\,s/8)(D+L) \qquad \text{(CA5.1-3)}$$

where

S_e = elastic section modulus based on the effective section

FS = 5/3 = the factor of safety for bending

F_y = specified yield point

L_s = span length, and s = beam spacing

D and L are, respectively, the code specified dead and live load intensities.

The mean resistance is defined as (Ref. 9)

$$R_m = R_n(P_mM_mF_m) \qquad \text{(CA5.1-4)}$$

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

$$R_n = S_eF_y \qquad \text{(CA5.1-5)}$$

that is, the nominal moment predicted on the basis of the post-buckling strength of the compression flange. The mean values P_m, M_m, and F_m, and the corresponding coefficients of variation V_P, V_M and V_F, are the statistical parameters which define the variability of the resistance:

P_m = the mean ratio of the experimentally determined moment to the predicted moment for the actual material and cross-sectional properties of the test specimens

M_m = mean ratio of the yield point to the minimum specified value

F_m = mean ratio of the actual section modulus to the specified (nominal) value

The coefficient of variation of R equals

$$V_R = \sqrt{V_P^2 + V_M^2 + V_F^2} \qquad \text{(CA5.1 - 6)}$$

The values of these data were obtained from examining the available tests on beams having different compression flanges with partially and fully effective flanges and webs, and from analyzing data on yield point values from tests and cross-sectional dimensions from many measurements. This information was developed in Ref. 10 and is given below:

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

The mean load effect is equal to

$$Q_m = (L_s^2 s/8)(D_m + L_m) \qquad \text{(CA5.1 - 7)}$$

and

$$V_Q = \frac{\sqrt{(D_mV_D)^2 + (L_mV_L)^2}}{D_m + L_m} \qquad \text{(CA5.1 - 8)}$$

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

Load statistics have been analyzed in Ref. 11, where it was shown that $D_m = 1.05D$, $V_D = 0.1$; $L_m = L$, $V_L = 0.25$.

The mean live load intensity equals the code live load intensity if the tributary area is small enough so that no live load reduction is included. Substitution of the load statistics into Eqs. CA5.1-7 and CA5.1-8 gives

$$Q_m = \frac{L_s^2 s}{8}\left(\frac{1.05D}{L} + 1\right)L \qquad \text{(CA5.1 - 9)}$$

$$V_Q = \frac{\sqrt{(1.05D/L)^2 V_D^2 + V_L^2}}{(1.05D/L + 1)} \qquad \text{(CA5.1 -10)}$$

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

From Eq. CA5.1-3 we obtain the nominal design capacity for D/L = 1/5 and FS = 5/3. Thus

$$\frac{R_m}{Q_m} = \frac{1.22x2.0xL(L_s^2 s/8)}{1.21L(L_s^2 s/8)} = 2.02$$

and, from Eq. CA5.1-2

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

Of itself β = 2.79 for beams having different compression flanges with partially and fully effective flanges and webs designed by the 1986 AISI Specification means nothing. However, when this is compared to β for other types of cold-formed members, and to β for designs of various types from hot-rolled steel shapes or even for other materials, then it is possible to say that this particular cold-formed steel beam has about an average reliability (Ref. 12).

Basis for LRFD of Cold-Formed Steel Structures

A great deal of work has been performed for determining the values of the reliability index β inherent in traditional design as exemplified by the current structural design specifications such as the AISC Specification for hot-rolled steel, the AISI Specification for cold-formed steel, the ACI Code for reinforced concrete members, etc. The studies for hot-rolled steel are summarized in Ref 9, where also many further papers are referenced which contain additional data. The determination of β for cold-formed steel elements or members is presented in Refs. 10 and 13 through 17., where both the basic research data as well as the β's inherent in the AISI Specification are presented in great detail. The β's computed in the above referenced publications were developed with slightly different load statistics than those of this Commentary, but the essential conclusions remain the same.

The entire set of data for hot-rolled steel and cold-formed steel designs, as well as data for reinforced concrete, aluminum, laminated timber, and masonry walls was re-analyzed in Refs. 11, 12 and 18 by using (a) updated load statistics and (b) a more advanced level of probability analysis which was able to incorporate probability distributions which describe the true distributions more realistically. The details of this extensive reanalysis are presented in Refs. 11, 12 and 18 and also only the final conclusions from the analysis are summarized here:

(1) The values of the reliability index β vary considerably for the different kinds of loading, the different types of construction, and the different types of members within a

given material design specification. In order to achieve more consistent reliability, it was suggested in Ref. 18 that the following values of β would provide this improved consistency while at the same time give, on the average, essentially the same design by the new LRFD method as is obtained by current design for all materials of construction. These target reliabilities β_o for use in LRFD are:

Basic case: Gravity loading, $\beta_o = 3.0$

For connections: $\beta_o = 4.5$

For wind loading: $\beta_o = 2.5$

These target reliability indices are the ones inherent in the load factors recommended in the ANSI/ASCE 7-88 Load Code (Ref. 19).

For simply supported, braced cold-formed steel beams with stiffened flanges, which were designed according to the 1986 AISI allowable stress design specification or to any previous version of this specification, it was shown above that for the representative dead-to-live load ratio of 1/5 the reliability index $\beta = 2.8$. Considering the fact that for other such load ratios, or for other types of members, the reliability index inherent in current cold-formed steel construction could be more or less than this value of 2.8, a somewhat lower target reliability index of $\beta_o = 2.5$ is recommended as a lower limit for the new LRFD Specification. The resistance factors ϕ were selected such that $\beta_o = 2.5$ is essentially the lower bound of the actual β's for members. In order to assure that failure of a structure is not initiated in the connections, a higher target reliability of $\beta_o = 3.5$ is recommended for joints and fasteners. These two targets of 2.5 and 3.5 for members and connections, respectively, are somewhat lower than those recommended by ANSI/ASCE 7-88 (i.e., 3.0 and 4.5, respectively), but they are essentially the same targets as are the basis for the 1986 AISC LRFD Specification (Ref. 3).

(2) The following load factors and load combinations were developed in Refs. 11 and 18 to give essentially the same β's as the target β_o's, and are recommended for use with the ANSI/ASCE 7-88 Load Code (Ref. 19) for all materials, including cold-formed steel:

1. $1.4D$
2. $1.2D+1.6L+0.5(L_r \text{ or } S \text{ or } R_r)$
3. $1.2D+1.6(L_r \text{ or } S \text{ or } R_r)+(0.5L \text{ or } 0.8W)$
4. $1.2D+1.3W+0.5L+0.5(L_r \text{ or } S \text{ or } R_r)$
5. $1.2D+1.5E+(0.5L \text{ or } 0.2S)$
6. $0.9D-(1.3W \text{ or } 1.5E)$

where

D = nominal dead load

E = nominal earthquake load

L = nominal live load due to occupancy;
weight of wet concrete for composite construction

L_r = nominal roof live load

R_r = nominal roof rain load

S = nominal snow load

W = nominal wind load

In view of the fact that the dead load of cold-formed steel structures is usually smaller than that of heavy construction, the first case of load combinations included in Section A5.1.4 of the Specification is (1.4D+L) instead of the ANSI/ASCE value of 1.4D. This AISI requirement is identical with the ANSI/ASCE Code when L = 0.

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

(a) For roof and floor composite construction

$$1.2D_s + 1.6\,C_w + 1.4C$$

where

D_s = weight of steel deck

C_w = nominal weight of wet concrete during construction

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

This suggestion provides safe construction practices for cold-formed steel decks and panels which otherwise may be damaged during construction. The load factor used for the weight of wet concrete is 1.6 because of delivering methods and an individual sheet can be subjected to this load. The use of a load factor of 1.4 for the construction load reflects a general practice of 33% strength increase for concentrated loads.

It should be noted that for the third case of load combinations, the load factor used for the nominal roof live load, L_r, in Section A5.1.4 of the AISI Specification is 1.4 instead of the ANSI/ASCE value of 1.6. The use of a relatively small load factor is because the roof live load is due to the presence of workmen and materials during repair operations and, therefore, can be considered as a type of construction load.

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

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

The load factors and load combinations given above are recommended for use with the LRFD criteria for cold-formed steel. The following portions of this Commentary present the background for the resistance factors ϕ which are recommended in Section A5.1.5 for the various members and connections in Sections B, C, D and E. These ϕ factors are determined in conformance with the load factors given above to approximately provide a target β_0 of 2.5 for members and 3.5 for connections, respectively, for the load combination 1.2D+1.6L. For practical reasons it is desirable to have relatively few different resistance factors, and so the actual values of β will differ from the derived targets. This means that

$$\phi R_n = c(1.2D+1.6L) = (1.2D/L+1.6)cL \qquad \text{(CA5.1-11)}$$

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

By assuming D/L = 1/5, Eqs. CA5.1-11 and CA5.1-9 can be rewritten as follows:

$$R_n = 1.84(cL/\phi) \quad \text{(CA5.1-12)}$$

$$Q_m = (1.05D/L+1)cL = 1.21cL \quad \text{(CA5.1-13)}$$

Therefore,

$$R_m/Q_m = (1.521/\phi)(R_m/R_n) \quad \text{(CA5.1-14)}$$

The ϕ factors can be computed from Eq. CA5.1-14 and the following equation by using $V_Q = 0.21$:

$$\text{Target } \beta_o = \frac{\ln(R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}} \quad \text{(CA5.1 - 15)}$$

A5.2 Yield Point and Strength Increase from Cold Work of Forming

This section is the same as Section A5.2 of the 1986 AISI Specification.

The following statistical data (mean values and coefficients of variation) on material and cross-sectional properties were develpoed in Refs. 13 and 14 for use in the derivation of the resistance factors ϕ:

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

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

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

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

The subscript m refers to mean values. The symbol V stands for coefficient of variation. The symbols M and F are, respectively, the ratio of the mean-to-the nominal material property or cross-sectional property; and F_y, F_{ya}, and F_u are, respectively, the specified minimum yield point, the average yield point including the effect of cold forming, and the specified minimum tensile strength.

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

A6 Reference Documents

The specifications and standards to which this Specification makes reference in various provisions are listed in Section A6 to provide the effective dates of these standards at the time of approval of this Specification.

Additional references which the designer may use for related information are :

1. American Institute of Steel Construction, "Specification for Structural Steel Buildings - Allowable Stress Design and Plastic Design," American Institute of Steel Construction (AISC), One East Wacker Drive, Suite 3100, Chicago, Illinois 60601-2001, June 1, 1989
2. Research Council on Structural Connections, "Allowable Stress Design Specification for Structural Joints Using ASTM A325 or A490 Bolts," Research Council on Structural Connections (RCSC), American Institute of Steel Construction (AISC), One East Wacker Drive, Suite 3100, Chicago, Illinois 60601-2001, November 13, 1985
3. Metal Building Manufacturers Association, "Low Rise Building Systems Manual," Metal Building Manufacturers Association (MBMA), 1230 Keith Building, Cleveland, Ohio 44115

4. Steel Deck Institute, "Design Manual for Composite Decks, Formed Decks, and Roof Decks," Steel Deck Institute, Inc. (SDI), P. O. Box 9506, Canton, Ohio 44711, 1984
5. Steel Joist Institute, "Standard Specifications Load Tables and Weight Tables for Steel Joists and Joist Girders," Steel Joist Institute (SJI), Suite A, 1205 48th Avenue North, Myrtle Beach, South Carolina 29577, 1986
6. Rack Manufacturers Institute, "Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks," Rack Manufacturers Institute (RMI), 8720 Red Oak Boulevard, Suite 201, Charlotte, North Carolina 28210, 1985
7. American Iron and Steel Institute, "Stainless Steel Cold-Formed Structural Design Manual," American Iron and Steel Institute (AISI), 1133 15th Street, N. W., Washington, D. C. 20005, 1974 Edition
8. American Society of Civil Engineers, "ASCE Standard, Specification for the Design and Construction of Composite Slabs," American Society of Civil Engineers (ASCE), 345 East 47th Street, New York, New York 10017, October, 1984
9. American Iron and Steel Institute, "Tentative Criteria for Structural Applications of Steel Tubing and Pipe," American Iron and Steel Institute (AISI), 1133 15th Street, N. W., Washington, D. C. 20005, August, 1976

B. ELEMENTS

B1 Dimensional Limits and Considerations

This section is the same as Section B1 of the AISI Specification for allowable stress design.

B2 Effective Widths of Stiffened Elements

This section is the same as Section B2 of the AISI Specification for allowable stress design.

B3 Effective Widths of Unstiffened Elements

This section is the same as Section B3 of the AISI Specification for allowable stress design.

B4 Effective Widths of Elements with an Edge Stiffener or One Intermediate Stiffener

This section is the same as Section B4 of the AISI Specification for allowable stress design.

Test data to verify the accuracy of the simple lip stiffener design was collected from a number of sources, both university and industry. These tests showed good correlation with the equations in Section B4.2. However, proprietary testing conducted in 1989 revealed that lip lengths with a d/t ratio of greater than 14 gave unconservative results.

A review of the original research data showed a lack of data for simple stiffening lips with d/t ratios greater than 14. Therefore, an upper limit of 14 is recommended pending further research.

B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener

This section is the same as Section B5 of the AISI Specification for allowable stress design.

B6 Stiffeners

B6.1 Transverse Stiffeners

The available experimental data on cold-formed steel transverse stiffeners were evaluated in Ref 10. The test results were compared to the predictions based on the same mathematical models on which the AISI Specification was based . The design provisions in these LRFD criteria are also based on the same mathematical models.

Load capacity in these LRFD criteria is based on the same prediction models as were employed in the formulation of the AISI Specification. A total of 61 tests were examined. The resistance factor $\phi_c = 0.85$ was selected on the basis of the statistical data given in Ref. 10. The corresponding safety indices vary from 3.32 to 3.41. A summary of the information is given in Table CB6.1.

Table CB6.1
Computed Safety Index β for Transverse Stiffeners
($\phi_c = 0.85$)

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

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

B6.2 Shear Stiffeners

The available experimental data on shear strength of the beam webs with shear stiffeners were calibrated in Ref. 10. The ϕ_v factors were taken as the same as those for shear strength of beams (Section C3.2). The statistical data used for determining the ϕ_v factor are given in Ref. 10 as follows:

$$P_m = 1.60; \qquad V_P = 0.09$$
$$M_m = 1.00; \qquad V_M = 0.06$$
$$F_m = 1.00; \qquad V_F = 0.05$$

Based on all these data, the value of β was found to be 4.10 for $\phi_v = 0.90$.

It should be noted that the equations for determining I_{smin} and A_{st} of attached shear stiffeners are based on the studies summarized in Ref. 43.

B6.3 Non-Conforming Stiffeners

This Section is the same as Section B6.3 of the AISI Specification for allowable stress design.

C. MEMBERS

C1 Properties of Sections

This section is the same as Section C1 of the AISI Specification for allowable stress design.

C2 Tension Members

Section C2 of the LRFD criteria was developed on the basis of Section C2 of the AISI Specification for allowable stress design, in which the design of tension members is based only on the yield point of steel.

The resistance factor of $\phi_t = 0.95$ used for tension member design was derived from the procedure described in Section A5.1 of this Commentary and a selected β_o value of 2.5. In the determination of the resistance factor, the following formulas were used for R_m and R_n:

$$R_m = A_n (F_y)_m \tag{CC2-1}$$

$$R_n = A_n F_y \tag{CC2-2}$$

$$\text{i.e. } R_m/R_n = (F_y)_m/F_y \tag{CC2-3}$$

in which A_n is the net area of the cross section, $(F_y)_m$ is equal to $1.10F_y$ as discussed in Section A5.2 of the Commentary. By using $V_M = 0.10$, $V_F = 0.05$ and $V_P = 0$, the coefficient of variation V_R is:

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

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

C3 Flexural Members

C3.1 Strength for Bending Only

Bending strengths of flexural members are differentiated according to whether or not the member is laterally braced. If such members are laterally supported, then they are proportioned according to the nominal section strength (Sec. C3.1.1). If they are laterally unbraced, then the limit state is lateral-torsional buckling (Sec. C3.1.2). For C- or Z-section with the tension flange attached to deck or sheathing and with compression flange laterally unbraced, the bending capacity is less than that of a fully braced member but greater than that of an unbraced member (Sec. C3.1.3).

C3.1.1 Nominal Section Strength

The bending strength of beams with a compression flange having stiffened, partially stiffened, or unstiffened elements is based on the post-buckling strength of the member, and use is made in LRFD of the effective width concept in the same way as in the 1986 AISI Specification. References 5, 6, 7, and 8 provide an extensive treatment of the background research.

The experimental bases for the post-buckling strengths of cold-formed beams were examined in Refs. 8 and 10, where different cases were studied according to the types of compression flanges and the effectiveness of webs.

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

Table CC3.1.1
Computed Safety Index β for Section Bending Strength of Beams
Based on Initiation of Yielding

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
Stiffened or Partially Stiffened Compression Flanges (ϕ_b = 0.95)								
FF. FW.	8	1.10	0.10	1.0	0.05	1.10543	0.03928	2.76
PF. FW.	30	1.10	0.10	1.0	0.05	1.11400	0.08889	2.65
PF. PW.	5	1.10	0.10	1.0	0.05	1.08162	0.09157	2.53
Unstiffened Compression Flanges (ϕ_b = 0.90)								
FF. FW.	3	1.10	0.10	1.0	0.05	1.43330	0.04337	4.05
PF. FW.	40	1.10	0.10	1.0	0.05	1.12384	0.13923	2.67
PF. PW.	10	1.10	0.10	1.0	0.05	1.03162	0.05538	2.66

Note: FF. = Fully effective flanges
PF. = Partially effective flanges
FW. = Fully effective webs
PW. = Partially effective webs
For details, see Ref. 10.

The computed values of β for the selected values of ϕ_b = 0.95 for sections with stiffened or partially stiffened compression flanges and 0.90 for sections with unstiffened compression flanges, and for a dead-to-live load ratio of 1/5 for different cases are listed in Table CC3.1.1. It can be seen that the β values vary from 2.53 to 4.05. In Table CC3.1.1, the values of M_m, V_M, F_m and V_F are the values presented in Sec. A5.2 of this Commentary for the material strength.

C3.1.2 Lateral Buckling Strength

There are not many test data on laterally unsupported cold-formed beams. The available test results are summarized in Ref. 10, and they are compared with predictions from AISI design formulas, theoretical formulas and SSRC formulas.

The statistical data used in Ref. 10 are listed in Table CC3.1.2. The symbol P is the ratio of the tested capacity to the predicted value, M is the ratio of the actual to the specified value of the modulus of elasticity, and F is the ratio of the actual to the nominal sectional properties.

Using the recommended resistance factor ϕ_b = 0.90, the values of β vary from 2.35 to 3.8. See Table CC3.1.2. It should be noted that the recommended design criteria use some simplified and conservative formulas, which are the same as the allowable stress design rules included in the 1986 AISI Specification.

Table CC3.1.2
Computed Safety Index β for Lateral Buckling Strength of Bending
($\phi_b = 0.90$)

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
1	47	1.0	0.06	1.0	0.05	2.5213	0.30955	3.79
2	47	1.0	0.06	1.0	0.05	1.2359	0.19494	2.48
3	47	1.0	0.06	1.0	0.05	1.1800	0.19000	2.35
4	47	1.0	0.06	1.0	0.05	1.7951	0.21994	3.53
5	47	1.0	0.06	1.0	0.05	1.8782	0.20534	3.80

Note: Case 1 = AISI approach
Case 2 = Theoretical approach with $J = 0.0026$ in.^{4}
Case 3 = SSRC approach with $J = 0.0026$ in.^{4}
Case 4 = Theoretical approach with $J = 0.0008213$ in.^{4}
Case 5 = SSRC approach with $J = 0.0008213$ in.^{4}

C3.1.3 Beams Having One Flange Through-Fastened to Deck or Sheathing

For beams having the tension flange attached to deck or sheathing and the compression flange unbraced, e.g., a roof purlin or wall girt subjected to wind suction, the bending capacity is less than a fully braced member, but greater than an unbraced member. This partial restraint is a function of the rotational stiffness provided by the panel-to-purlin connection. The Specification contains factors that represent the reduction in capacity from a fully braced condition. These factors are based on experimental results obtained for both simple and continuous span purlins (Refs. 20 to 24).

As indicated in Ref. 25, the rotational stiffness of the panel-to-purlin connection is primarily a function of the member thickness, sheet thickness, fastener type and fastener location. For compressed glass fiber blanket insulation of initial thicknesses of zero to six inches, the rotational stiffness was not measurably affected (Ref. 25). To ensure adequate rotational stiffness of the roof and wall systems designed using the Specification provision, Section C3.1.3 explicitly states the acceptable panel and fastener types.

Continuous beam tests were made on three equal spans and the R values were calculated from the failure loads, using as a maximum positive moment, $M = 0.08wL_s^2$.

The provisions of Section C3.1.3 apply to beams on which the tension flange is attached to deck or sheathing and the compression flange is completely unbraced. Beams with discrete point braces on the compression flange may have a bending capacity greater than those completely unbraced. Available data from simple span tests (Refs. 20, 23, 37, 38, and 39) indicate that for members having a lip edge stiffener at an angle of 75 degrees or greater with the plane of the compression flange and braces to the compression flange located at third points or more frequently, member capacities may be increased over those without discrete braces.

In this section, the ϕ_b factor is determined for the load combination of 1.17W-0.9D to approximately provide a target β_o of 1.5 for counteracting loads with a reduction factor of 0.9 applied to the load factor for the nominal wind load. The reasons for using a low target β_o are discussed in Section A5.1 of this Commentary. Based on this type of load combination, the following equations can be established:

$$\phi R_n = c(1.17W - 0.9D) = (1.17 - 0.9D/W)cW \qquad \text{(CC3.1.3-1)}$$

$$Q_m = c(W_m - D_m) \qquad \text{(CC3.1.3-2)}$$

$$V_Q = \frac{\sqrt{(W_m V_W)^2 + (D_m V_D)^2}}{W_m - D_m} \qquad \text{(CC3.1.3-3)}$$

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

Load statistics have been analyzed in Ref. 11, where it was shown that

$$D_m = 1.05D,\ V_D = 0.1;\ W_m = 0.78W,\ V_W = 0.37$$

The substitution of the load statistics into Eqs. CC3.1.3-2 and CC3.1.3-3 gives

$$Q_m = c(0.78W - 1.05D) = (0.78 - 1.05D/W)cW \qquad \text{(CC3.1.3-4)}$$

$$V_Q = \frac{\sqrt{(0.78 \text{x} 0.37)^2 + (1.05D/W \text{x} 0.1)^2}}{0.78 - 1.05D/W} \qquad \text{(CC3.1.3-5)}$$

By assuming D/W = 0.1, Eqs. CC3.1.3-1, CC3.1.3-4, and CC3.1.3-5 can be rewritten as follows:

$$\phi R_n = 1.08cW$$

$$Q_m = 0.675cW = 0.675(\phi R_n/1.08) = 0.625\phi R_n \qquad \text{(CC3.1.3-6)}$$

$$V_Q = 0.43$$

The application of Eqs. CA5.1-2, CA5.1-4, CC3.1.3-6, and CA5.1-6 gives

$$\beta = \frac{\ln(1.6 M_m F_m P_m / \phi)}{\sqrt{V_M^2 + V_F^2 + V_P^2 + V_Q^2}} \qquad \text{(CC3.1.3-7)}$$

or

$$\phi = 1.6(M_m F_m P_m)\exp(-\beta\sqrt{V_M^2 + V_F^2 + V_P^2 + V_Q^2}) \qquad \text{(CC3.1.3-8)}$$

The computed values of β for the selected value of $\phi_b = 0.90$ for different cases are listed in Table CC3.1.3. It can be seen that the β values vary from 1.50 to 1.60 which are satisfactory for the target value of 1.5. In Table CC3.1.3, the values of M_m, V_M, F_m, and V_F are the values presented in Section A5.2 of this Commentary for the material strength and fabrication.

Table CC3.1.3
Computed Safety Index β for Beams Having One Flange Through-Fastened to Deck or Sheathing
($\phi_b = 0.90$)

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
1	5	1.10	0.10	1.0	0.05	1.1995	0.2991	1.60
2	15	1.10	0.10	1.0	0.05	1.0128	0.1112	1.50
3	5	1.10	0.10	1.0	0.05	1.0466	0.1010	1.58
4	14	1.10	0.10	1.0	0.05	1.0034	0.0689	1.51

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

C3.2 Strength for Shear Only

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

$$V_n = A_w \tau_y = A_w F_y / \sqrt{3} = 0.577 F_y ht \qquad \text{(CC3.2-1)}$$

in which A_w is the area of the beam web computed by (ht), and τ_y is the yield point of steel in shear, which can be computed by $F_y/\sqrt{3}$.

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

$$V_n = A_w \tau_{cr} = \frac{k_v \pi^2 E A_w}{12(1-\mu^2)(h/t)^2} \qquad \text{(CC3.2-2)}$$

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

$$V_n = 0.905 E k_v t^3/h \qquad \text{(CC3.2-3)}$$

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

$$V_n = 0.64 t^2 \sqrt{k_v F_y E} \qquad \text{(CC3.2-4)}$$

In view of the fact that the appropriate test data on shear are not available, the ϕ_v factors used in Section C3.2 were derived from the condition that the nominal resistance for the LRFD method is the same as the nominal resistance for the allowable stress design method. Thus,

$$(R_n)_{LRFD} = (R_n)_{ASD} \qquad \text{(CC3.2-5)}$$

Since

$$(R_n)_{LRFD} \geq c(1.2D + 1.6L) / \phi_v \tag{CC3.2-6}$$

$$(R_n)_{ASD} \geq c(F.S.)(D + L) \tag{CC3.2-7}$$

the resistance factors can be computed from the following formula:

$$\phi_v = \frac{1.2D + 1.6L}{(F.S.)(D + L)}$$

$$= \frac{1.2(D/L) + 1.6}{(F.S.)(D/L + 1)} \tag{CC3.2-8}$$

By using a dead-to-live load ratio of $D/L = 1/5$, the ϕ_v factors computed from the above equation are listed in Table CC3.2 for three different ranges of h/t ratios. The factors of safety are adopted from the AISI Specification for allowable stress design. It should be noted that the use of a small safety factor of 1.44 for yielding in shear is justified by long standing use and by the minor consequences of incipient yielding in shear compared with those associated with yielding in tension and compression.

Table CC3.2

Range of h / t Ratio	F.S. for Allowable Load Design	ϕ_v Factor computed by Eq. CC3.2-8	Recommended ϕ_v Factor
$h/t \leq \sqrt{Ek_v/F_y}$	1.44	1.06	1.00
$\sqrt{Ek_v/F_y} \leq h/t \leq 1.415\sqrt{Ek_v/F_y}$	1.67	0.92	0.90
$h/t > 1.415\sqrt{Ek_v/F_y}$	1.71	0.90	0.90

C3.3 Strength for Combined Bending and Shear

This section is based on the interaction formulas included in Section C3.3 of the AISI Specification for allowable stress design.

C3.4 Web Crippling Strength

The nominal concentrated load or reaction, P_n, is determined by the allowable load given in Section C3.4 of the AISI Specification times the appropriate factor of safety. In this regard, a factor of safety of 1.85 is used for Eqs. C3.4-1, C3.4-2, C3.4-4, C3.4-6 and C3.4-8, and a factor of safety of 2.0 is used for Eqs. C3.4-3, C3.4-5, C3.4-7 and C3.4-9.

On the basis of the statistical analysis of the available test data on web crippling, the values of P_m, M_m, F_m, V_P, V_M and V_F were computed and selected. These values are presented in Table CC3.4 (see Table 76 of Ref. 10). By using $\beta_o = 2.5$, the resistance factors $\phi_w = 0.75$ and 0.80 were selected for single unreinforced webs and I-sections, respectively, and is used in Sections A5.1.5 and C3.4. The values of β corresponding to these values of ϕ_w are also given in Table CC3.4.

Table CC3.4
Computed Safety Index β for Web Crippling Strength of Beams

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

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

C3.5 Combined Bending and Web Crippling Strength

This section is based on the interaction formulas included in Section C3.5 of the AISI Specification for allowable stress design.

A total of 551 tests were calibrated for combined bending and web crippling strength. Six different cases were studied. Based on $\phi_w = 0.75$ for single unreinforced webs and $\phi_w = 0.80$ for I-sections, the values of safety indices vary from 2.45 to 3.27 as given in Table CC3.5.

Table CC3.5
Computed Safety Index β for Combined Bending and Web Crippling

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

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

C4 Concentrically Loaded Compression Members

The available experimental data on cold-formed steel concentrically loaded compression members were evaluated in Ref 10. The test results were compared to the predictions based on the same mathematical models on which the AISI Specification was based . The design provisions in these LRFD criteria are also based on the same mathematical models.

Column capacity in these LRFD criteria is based on the same prediction models as were employed in the formulation of the AISI Specification. A total of 264 tests were examined; 14 different cases were studied according to the types of the column, the types of the compression flanges and the failure modes. The resistance factor $\phi_c = 0.85$ was selected on the basis of the statistical data given in Ref. 10. The corresponding safety indices vary from 2.39 to 3.34. A summary of the information is given in Table CC4.

The safety indices were determined from Eq. CA5.1-2 for a D/L ratio of 1/5. Different ϕ_c factors could have been used for different cases.

Table CC4
Computed Safety Index β for Concentrically Loaded Compression Member
(ϕ_c= 0.85)

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
1	5	1.10	0.10	1.0	0.05	1.14610	0.10452	3.13
2	24	1.10	0.10	1.0	0.05	1.05053	0.07971	2.89
3	15	1.10	0.10	1.0	0.05	1.05523	0.07488	2.93
4	3	1.10	0.10	1.0	0.05	1.10550	0.07601	3.11
5	28	1.10	0.10	1.0	0.05	1.04750	0.11072	2.76
6	25	1.10	0.10	1.0	0.05	1.22391	0.21814	2.72
7	9	1.00	0.06	1.0	0.05	0.96330	0.04424	2.39
8	41	1.10	0.10	1.0	0.05	1.19620	0.09608	3.34
9	18	1.10	0.10	1.0	0.05	1.02900	0.08131	2.81
10	12	1.10	0.11	1.0	0.05	1.06180	0.11062	2.77
11	8	1.00	0.06	1.0	0.05	1.15290	0.10544	2.92
12	30	1.10	0.10	1.0	0.05	1.07960	0.15061	2.68
13	14	1.10	0.10	1.0	0.05	1.07930	0.08042	3.00
14	32	1.10	0.10	1.0	0.05	1.08050	0.10772	2.89

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

C5 Combined Axial Load and Bending

The LRFD Specification provide the similar interaction equations as the 1986 Edition of the AISI Specification with 1989 Addendum.

A total of 144 tests were calibrated for combined axial load and bending. Nine different cases were studied according to the types of sections, the stable conditions and the loading conditions. Based on $\phi_c = 0.85$, $\phi_b = 0.95$ or 0.90 for nominal section strength (see Section C3.1.1), and $\phi_b = 0.90$ for lateral buckling strength, the values of safety indices vary from 2.7 to 3.34 as given in Table CC5.

Table CC5
Computed Safety Index β for Combined Axial Load and Bending
(based on $\phi_c = 0.85$)

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
1	18	1.05	0.10	1.0	0.05	1.0367	0.06619	2.70
2	13	1.05	0.10	1.0	0.05	1.0509	0.07792	2.72
3	33	1.05	0.10	1.0	0.05	1.1028	0.09182	2.86
4	18	1.05	0.10	1.0	0.05	1.1489	0.10478	2.96
5	6	1.05	0.10	1.0	0.05	1.1600	0.13000	2.87
6	17	1.05	0.10	1.0	0.05	1.1200	0.09000	2.92
7	10	1.05	0.10	1.0	0.05	1.2300	0.08000	3.34
8	17	1.05	0.10	1.0	0.05	1.0910	0.07950	2.86
9	12	1.05	0.10	1.0	0.05	1.1110	0.11450	2.79

Note:
Case 1 = Locally stable beam-columns, hat sections of Pekoz and Winter (1967)[26]
Case 2 = Locally unstable beam-columns, lipped channel sections of Thomasson (1978)[27]
Case 3 = Locally unstable beam-columns, lipped channel sections of Loughlan (1979)[28]
Case 4 = Locally unstable beam-columns, lipped channel sections of Mulligan and Pekoz (1983)[29]
Case 5 = Locally stable beam-columns, lipped channel sections of Loh and Pekoz (1985)[30] with $e_x \neq 0$ and $e_y = 0$
Case 6 = Locally stable beam-columns, lipped channel sections of Loh and Pekoz (1985) with $e_x = 0$ and $e_y \neq 0$
Case 7 = Locally stable beam-columns, lipped channel sections of Loh and Pekoz (1985) with $e_x \neq 0$ and $e_y \neq 0$
Case 8 = Locally unstable beam-columns, lipped channel sections of Loh and Pekoz (1985) with $e_x = 0$ and $e_y \neq 0$
Case 9 = Locally unstable beam-columns, lipped channel sections of Loh and Pekoz (1985) with $e_x \neq 0$ and $e_y \neq 0$

C6 Cylindrical Tubular Members

Section C6 of the LRFD criteria is based on Section C6 of the AISI Specification for allowable stress design.

The ϕ_b factor of 0.95 used in Section C6.1 for bending is the same as that used in Section C3.1.1, while the ϕ_c factor of 0.85 used in Section C6.2 for compression is the same as that used in Section C4 for concentrically loaded compression members.

D. STRUCTURAL ASSEMBLIES

D1 Built-Up Sections

This section is the same as Section D1 of the AISI Specification for allowable stress design.

D2 Mixed Systems

This section is the same as Section D2 of the AISI Specification for allowable stress design.

D3 Lateral Bracing

This section is the same as Section D3 of the AISI Specification for allowable stress design.

With regard to the footnote for Section D3.2, it should be noted that in conventional metal building roof systems, the roof panels are attached to the top flange of each purlin throughout its length using self-drilling or self-tapping, through-the-sheet, fasteners spaced at approximately 1 foot on center. This panel usually provides sufficient stiffness to prevent the relative movement of the purlins with respect to each other, however, unless external restraint is provided, the system as a whole will tend to move laterally. This restraint or anchorage may consist of members attached to the purlin at discrete locations along the span and designed to carry forces necessary to restrain the system against lateral movement. The design rules for Z-purlin supported roof systems are based on a first order, elastic stiffness model (Reference 31).

D4 Wall Studs and Wall Stud Assemblies

D4.1 Wall Studs in Compression

The AISI design provisions on the compression strength of wall studs were calibrated in Ref. 10. The statistical data used for determining the ϕ_c factor are given in Ref. 10 as follows:

$$P_m = 1.14; \qquad V_P = 0.10$$

$$M_m = 1.10; \qquad V_M = 0.10$$

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

Based on all these data and $\phi_c = 0.85$, the β value was found to be 3.14.

The provisions in this Specification section are given to prevent three possible modes of failure. Provision (a) is for column buckling between fasteners, even if one fastener is missing or otherwise ineffective. Provision (b) contains formulas for nominal axial strengths for overall column buckling. Essential to these provisions is the magnitude of the shear rigidity of the sheathing material.

A table of shearing parameters and an equation for determining the shear rigidity are provided in this Specification. These values are based on the small scale tests described in References 32 and 33. For other types of materials, the sheathing parameters can be determined using the procedures described in these references.

Provision (c) is to insure that the sheathing has sufficient distortion capacity. The procedure involves assuming a value of the stress and checking whether the shear strain at the load corresponding to the stress exceeds the permissible design shear strain of the sheathing material. In principle, the procedure is one of successive approximations.

However, if the smaller of F_e (provision a) or σ_{cr} (provision b) is tried and shown to be satisfactory, then the need for iteration is eliminated.

D4.2 Wall Studs In Bending

The ϕ_b factors for bending strength of wall studs were taken as the same as those for section bending strength of beams (Section C3.1.1). The available test data on wall stud sections with stiffened or partially stiffened compression flanges were calibrated in Ref. 10. The statistical data used for determining the β value are given in Ref. 10 as follows:

$$P_m = 1.27; \qquad V_P = 0.01$$

$$M_m = 1.10; \qquad V_M = 0.10$$

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

Based on all these data and $\phi_b = 0.95$, the β value was found to be 3.37.

D4.3 Wall Studs with Combined Axial Load and Bending

The LRFD criteria provide the same interaction equations as the AISI Specification for allowable stress design.

The available test data on wall studs with combined axial load and bending were calibrated in Ref. 10. The statistical data used for determining the β value are given in Ref. 10 as follows:

$$P_m = 1.19; \qquad V_P = 0.13$$

$$M_m = 1.05; \qquad V_M = 0.10$$

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

Based on $\phi_c = 0.85$ and $\phi_b = 0.95$ for sections with stiffened or partially stiffened compression flanges or $\phi_b = 0.90$ for sections with unstiffened compression flanges, the β value was found to be 2.94.

E. CONNECTIONS AND JOINTS

Section E of the LRFD criteria is based on Section E of the AISI Specification for allowable stress design. This section contains the design provisions for welded connections, bolted connections, shear rupture and connections to other materials.

The resistance factors used for welded and bolted connections were derived for a target reliability index $\beta_o = 3.5$ and the statistical data are summarized in the subsequent sections.

E1 General Provisions

This section is based on Section E1 of the AISI Specification for allowable stress design.

E2 Welded Connections

Section E2 contains the design provisions for arc-welds (groove welds in butt joints, arc spot welds, arc seam welds, fillet welds, and flare groove welds) and resistance welds. The design equations for the nominal strength and the ϕ factors for groove welds in butt joints are the same as that used in the AISC LRFD criteria. (Ref. 3)

For arc spot welds, the ϕ factor of 0.60 used for determining the design shear strength of welds is based on the test data reported in Ref. 34. It gives a β value of 3.55. The statistical data used for deriving the ϕ factor are given in Ref 10 as follows:

$$P_m = 1.17; \quad V_P = 0.22$$

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

$$F_m = 1.00; \quad V_F = 0.10$$

Table CE2-1

Computed Safety Index β for Plate Failure in Weld Connections

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

Note: Case 1 = For $d_a/t \leq 0.815 \sqrt{(E/F_u)}$
Case 2 = For $d_a/t > 1.397 \sqrt{(E/F_u)}$
Case 3 = Longitudinal Loading, $L/t < 25$
Case 4 = Longitudinal Loading, $L/t \geq 25$
Case 5 = Transverse Loading

Table CE2-2
Computed Safety Index β for Tensile Strength of Arc Spot Weld
($\phi = 0.65$)

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

Note: Case 1 is for 1.2D+1.6L ($\beta_o = 3.5$)
Case 2 is for 1.17W-0.9D ($\beta_o = 2.5$)

With regard to the types of the plate failure governed by Eqs. E2.2-2 through Eq. E2.2-4 in the design criteria, ϕ factors were derived from the statistical data presented in Table CE2-1 (Ref. 35). The ϕ factors used for minimum edge distance were taken as the same as those used for bolted connections.

For tension loads on arc spot welds, tests were performed which included variables of steel strengths from 50 to 68.5 ksi, steel sheet thickness ranging from 0.031 to 0.072 inches, galvanized steel sheet, prime painted and galvanized steel plate, and visible diameter of welds ranging from 0.47 to 0.94 inches. Thus, the additional limitations only applicable to arc spot welds in tension were included in the Specification. For use in calculating the tension load on arc spot welds, the tensile strength of the connected sheet, F_u, is limited to a maximum of 60 ksi, although sheet with greater tensile strengths may be used. The development of the equation is contained in Reference 40. The tests were reported in Reference 41.

The statistical data for deriving the ϕ factor are presented in Table CE2-2 (Ref. 42). Two cases were considered in the determination of ϕ factor: (1) 1.2D+1.6L with $\beta_o = 3.5$, and (2) 1.17W-0.9D with $\beta_o = 2.5$ (counteracting loads with a reduction factor of 0.9 applied to the load factor for the nominal wind load). $\phi = 0.65$ was selected for both cases, and the values of β corresponding to the selected ϕ factor are given in Table CE2-2. It can be seen that for both cases, the β values compare satisfactorily to the target reliability indices.

For arc seam welds, the design shear strength of welds is determined from the same ϕ factor used for arc spot welds. The derivation of the ϕ factor for plate tearing is based on the following statistical data (Ref. 10):

$$P_m = 1.00; \qquad V_P = 0.10$$
$$M_m = 1.10; \qquad V_M = 0.10$$
$$F_m = 1.00; \qquad V_F = 0.10$$

For the selected value of $\phi = 0.60$, the value of $\beta = 3.81$.

For fillet welds, the ϕ factors used for longitudinal loading (Eqs. E2.4-1 and E2.4-2) and transverse loading (Eq. E2.4-3) are based on the statistical data presented in Table CE2-1 (Ref. 35).

Similar to the arc spot welds, a ϕ factor of 0.60 is used for the design shear strength of welds.

For flare groove welds, the following statistical data were used to determined the ϕ factors:

(a) Transverse Flare Bevel Welds ($\phi = 0.55$, $\beta = 3.81$)

$P_m = 1.04$; $V_P = 0.17$

$M_m = 1.10$; $V_M = 0.10$

$F_m = 1.00$; $V_F = 0.10$

(b) Longitudinal Flare Bevel Welds ($\phi = 0.55$, $\beta = 3.56$)

$P_m = 0.97$; $V_P = 0.17$

$M_m = 1.10$; $V_M = 0.10$

$F_m = 1.00$; $V_F = 0.10$

For detailed information, see Ref. 10.

For resistance welds, the nominal shear strength is based on the following equation:

$R_n = (2.5)$ x (allowable shear per spot specified in Section E2.6 of the AISI Specification for allowable stress design)

In the above equation, the safety factor is 2.5.

The ϕ factor of 0.65 used in Section E2.6 for the design of resistance welds was determined on the basis of the following statistical data reported in Ref. 10. It gives a β value of 3.71.

$P_m = 1.00$; $V_P = 0.03$

$M_m = 1.10$; $V_M = 0.10$

$F_m = 1.00$; $V_F = 0.10$

E3 Bolted Connections

Section E3 of the LRFD criteria is based on Section E3 of the AISI Specification for allowable stress design. It deals only with the design of bolted connections used for connected parts thinner than 3/16 inch in thickness. For the design of bolted connections using materials equal to or greater than 3/16 inch in thickness, the AISC Specification should be used.

All ϕ factors were computed from the statistical data given in Ref. 10 and $\beta_o = 3.5$. The statistical data used in the study are presented in Table CE3.

The ϕ factors used for the high strength bolts for design shear and tensile strengths are adopted from Ref. 3.

Table CE3
Computed Safety Index β for Bolted Connections

Case	M_m	V_M	F_m	V_F	P_m	V_P	ϕ	β
Minimum Spacing and Edge Distance								
1	1.10	0.08	1.00	0.05	1.13	0.12	0.70	3.75
2	1.10	0.08	1.00	0.05	1.18	0.14	0.70	3.84
3	1.10	0.08	1.00	0.05	0.84	0.05	0.60	3.61
4	1.10	0.08	1.00	0.05	0.94	0.09	0.60	3.90
5	1.10	0.08	1.00	0.05	1.06	0.11	0.70	3.62
6	1.10	0.08	1.00	0.05	1.14	0.19	0.60	3.87
Tension Stress on Net Section								
7	1.10	0.08	1.00	0.05	1.14	0.20	0.65	3.53
8	1.10	0.08	1.00	0.05	0.95	0.21	0.55	3.41
9	1.10	0.08	1.00	0.05	1.04	0.14	0.65	3.63
Bearing Stress on Bolted Connections								
10	1.10	0.08	1.00	0.05	1.08	0.23	0.55	3.65
11	1.10	0.08	1.00	0.05	0.97	0.07	0.65	3.80
12	1.10	0.08	1.00	0.05	1.02	0.20	0.60	3.43
13	1.10	0.08	1.00	0.05	1.05	0.13	0.60	4.06
14	1.10	0.08	1.00	0.05	1.01	0.04	0.70	3.71
15	1.10	0.08	1.00	0.05	0.93	0.05	0.65	3.70
Shear Strength on A307 Bolts								
16	1.28	0.08	1.00	0.05	0.68	0.11	0.65	4.73
17	1.13	0.08	1.00	0.05	0.60	0.10	0.65	3.85
18	1.28	0.08	1.00	0.05	0.75	0.10	0.65	5.23
19	1.36	0.08	1.00	0.05	0.63	0.06	0.65	4.49
20	1.13	0.08	1.00	0.05	0.76	0.06	0.65	5.09

Note: Case 1 = Single shear, with washers, $F_u/F_y \geq 1.15$
Case 2 = Double shear, with washers, $F_u/F_y \geq 1.15$

Table CE3 (Continued)
Computed Safety Index β for Bolted Connections

Case 3 = Single shear, with washers, $F_u/F_y < 1.15$
Case 4 = Double shear, with washers, $F_u/F_y < 1.15$
Case 5 = Single shear, without washers, $F_u/F_y \geq 1.15$
Case 6 = Single shear, without washers, $F_u/F_y < 1.15$
Case 7 = $t < 3/16$ in., double shear, with washers
Case 8 = $t < 3/16$ in., single shear, with washers
Case 9 = $t < 3/16$ in., single shear, without washers
Case 10 = $0.024 \leq t < 3/16$ in., double shear, with washers, $F_u/F_y \geq 1.15$
Case 11 = $0.024 \leq t < 3/16$ in., double shear, with washers, $F_u/F_y < 1.15$
Case 12 = $0.024 \leq t < 3/16$ in., single shear, with washers, $F_u/F_y \geq 1.15$
Case 13 = $0.024 \leq t < 3/16$ in., single shear, with washers, $F_u/F_y < 1.15$
Case 14 = $0.036 \leq t < 3/16$ in., single shear, without washers, $F_u/F_y \geq 1.15$
Case 15 = $0.036 \leq t < 3/16$ in., double shear, without washers, $F_u/F_y \geq 1.15$
Case 16 = Double shear, with washers, 3/8 in. diameter
Case 17 = Double shear, with washers, 3/4 in. diameter
Case 18 = Single shear, with washers, 3/8 in. diameter
Case 19 = Single shear, with washers, 1/2 in. diameter
Case 20 = Single shear, with washers, 3/4 in. diameter

E4 Shear Rupture

Section E4 of the LRFD criteria is based on Section E4 of the AISI Specification for allowable stress design. The ϕ factor used in this section is adopted from Ref. 3.

E5 Connections to Other Materials

Section E5 of the LRFD criteria is based on Section E5 of the AISI Specification for allowable stress design. The ϕ factor used for bearing is adopted from Ref. 3.

F. TESTS FOR SPECIAL CASES

F1 Tests for Determining Structural Performance

The determination of load-carrying capacity of the tested elements, assemblies, connections, or members is based on the same basis for the LRFD design criteria. The correction factor C_P is used in the determination of ϕ factor to account for the influence due to the small number of tests (Ref. 36). It should be noted that when the number of tests is large enough, the effect of correction factor is negligible.

For beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced (subject to wind uplift), the calibration is based on a load combination of 1.17W-0.9D with D/W = 0.1 (see Section C3.1.3 of this Commentary for detailed discussion).

The statistical data needed for the determination of resistance factor are listed in Table F1.

F2 Tests for Confirming Structural Performance

This section is basically the same as Section F2 of the AISI Specification for allowable stress design.

F3 Tests for Determining Mechanical Properties

This section is the same as Section F3 of the AISI Specification for allowable stress design.

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COMMENTARY

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Preface

This document provides a commentary on the background for the *Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members*.

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COMMENTARY ON THE LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

INTRODUCTION

In the design of steel buildings, the "Allowable Stress Design Criteria" have long been used for the design of cold-formed steel structural members in the United States and other countries. Even though the theoretical concept of reliability analysis has been available for some time and the significance of such a concept in structural safety and design is well recognized, the probabilistic method has not yet been explicitly adopted as a basis for the American design standard for cold-formed steel structures.

Recently, the load and resistance factor design (LRFD) criteria have been developed for steel buildings using hot-rolled shapes and built-up members fabricated from steel plates. It became evident that the development of a new specification for load and resistance factor design of cold-formed steel is highly desirable because the design criteria for heavy hot-rolled steel construction cannot possibly cover the design features of thin-walled, cold-formed steel construction completely.

Since 1976, a joint project has been conducted at University of Missouri-Rolla and Washington University to develop the new design criteria for cold-formed steel structural members and connections based on the probabilistic approach.

The Load and Resistance Factor Design criteria developed on the basis of the 1986 Edition of the AISI Specification with 1989 Addendum for allowable stress design are included in Sections A through F of this Specification.

This commentary contains a brief presentation of the methodology used for the development of the load and resistance factor design criteria. In addition, it provides a record of the reasoning behind, and the justification for, various provisions of the Specification. For detailed background information, reference is made to the research reports given in the bibliography.

A. GENERAL PROVISIONS

A1 Limits of Applicability and Terms

Section A1 of the LRFD Specification is essentially the same as Section A1 of the AISI Specification for allowable stress design. The definitions and various terms used for the LRFD criteria are the same as that used for the allowable stress design.

A2 Non-Conforming Shapes and Constructions

Section A2 of the LRFD Specification is essentially the same as Section A2 of the AISI Specification for allowable stress design.

A3 Material

This Section is essentially the same as Section A3 of the AISI Specification for allowable stress design.

In lieu of the tensile-to-yield strength limit of 1.08, the Specification permits the use of elongation requirements using the measurement technique as given in Ref. 1, and Part VII of the Manual. Because of limited experimental verification of the structural performance of members using material having a tensile-to-yield strength ratio less than 1.08 (Ref. 2), the Specification limits the use of this material to purlins and girts meeting the elastic design requirements of Sections C3.1.1(a), C3.1.2, and C3.1.3. Thus, the use of such steel in other applications (compression members, tension members, other flexural members including those whose strength is based on inelastic reserve capacity, etc.) is prohibited. However, in purlins and girts, concurrent axial loads of relatively small magnitude are acceptable providing the requirements of Section C5 are met and $P_u/\phi_c P_n$ does not exceed 0.15.

A4 Loads

This Section is the same as Section A4 of the AISI Specification for allowable stress design.

With regard to ponding, design guidance can be found from Section K2 of the AISC Load and Resistance Factor Design Specification for Structural Steel Buildings (Ref. 3).

A5 Structural Analysis and Design

A5.1 Design Basis

The current method of designing cold-formed steel structural members, as presented in the 1986 AISI Specification (Ref. 4), is based on the Allowable Stress Design method. In this approach, the forces (bending moments, axial forces, shear forces) in structural members are computed by accepted methods of structural analysis for the specified working loads. These member forces or moments should not exceed the allowable values permitted by the AISI Specification. The AISI allowable load or moment is determined by dividing the nominal load or moment at a limit state by a factor of safety. Usual factors of safety inherent in the AISI Specification for the Design of Cold-Formed Steel Structural Members are 5/3 for beams and 23/12 for columns.

A limit state is the condition at which the structural usefulness of a load-carrying element or member is impaired to such an extent that it becomes unsafe for the occupants of the structure, or the element no longer performs its intended function. Typical limit states for cold-formed steel members are excessive deflection, yielding, buckling and attainment of maximum strength after local buckling (i.e., post-buckling strength). These limit states have been established through experience in practice or in the laboratory, and they have been thoroughly investigated through analytical and experimental research. The background for the establishment of the limit states is extensively documented in the Commentary on the AISI Specification (Refs. 5 and 6)(see also Refs. 7 and 8), and a continuing research effort provides further improvement in understanding them.

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

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

$$\phi R_n \geq \Sigma \gamma_i Q_i \qquad \text{(CA5.1-1)}$$

where

R_n = the nominal resistance

ϕ = resistance factor

γ_i = load factors

Q_i = load effects

The nominal resistance is the strength of the element or member for a given limit state, computed for nominal section properties and for minimum specified material properties according to the appropriate analytical model which defines the strength. The resistance factor ϕ accounts for the uncertainties and variabilities inherent in the R_n, and it is usually less than unity. The load effects Q_i are the forces on the cross section (bending moment, axial force, shear force) determined from the specified minimum loads by structural analysis, and γ_i are the corresponding load factors which account for the uncertainties and variabilities of the loads. The load factors are greater than unity.

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

Probabilistic Concepts

Factors of safety or load factors are provided against the uncertainties and variabilities which are inherent in the design process. Structural design consists of comparing nominal load effects Q to nominal resistances R, but both Q and R are random parameters (see Fig. CA5.1-1). A limit state is violated if $R < Q$. While the possibility of this event ever occurring is never zero, a successful design should, nevertheless, have only an acceptably small probability of exceeding the limit state. If the exact probability distributions of Q and R were known, then the probability of $(R - Q) < 0$ could be exactly determined for any design. In general the distributions of Q and R are not known, and only the means, Q_m and R_m, and the standard deviations, σ_Q and σ_R are available. Nevertheless it is possible to determine relative reliabilities of several designs by the scheme illustrated in Fig. CA5.1-2. The distribution curve shown is for ln(R/Q), and a limit state is exceeded when $\ln(R/Q) \leq 0$. The area under $\ln(R/Q) \leq 0$ is the probability of violating the limit state. The size of this area is dependent on the distance between the origin and the mean of ln(R/Q). For given statistical data R_m, Q_m, σ_R and σ_Q, the area under $\ln(R/Q) \leq 0$ can be varied by changing the value of β (Fig. CA5.1-2), since $\beta\sigma_{\ln(R/Q)} = \ln(R/Q)_m$, from which approximately

$$\beta = \frac{\ln(R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}} \qquad \text{(CA5.1 -2)}$$

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

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

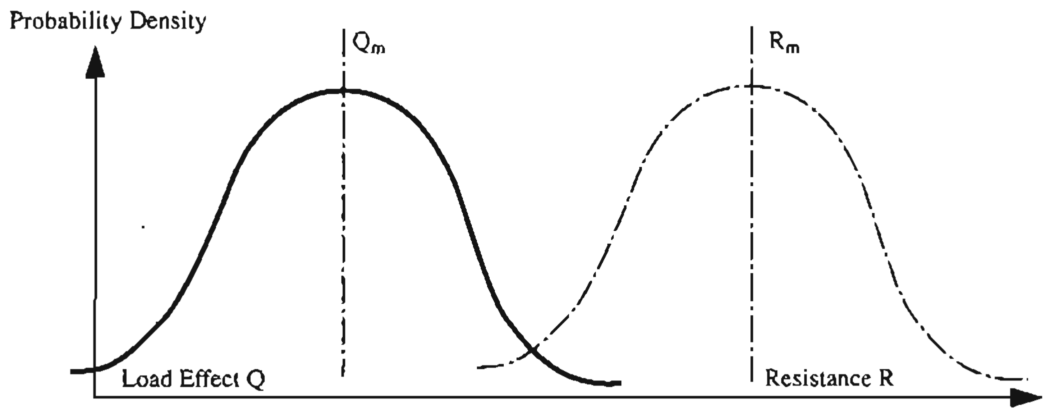

Fig. CA5.1-1 Definition of the Randomness of Q and R

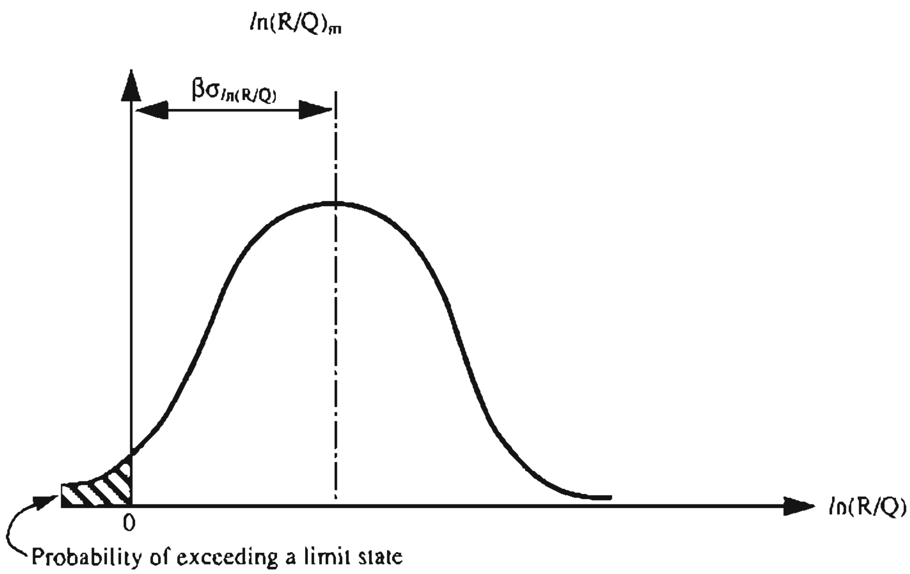

Fig. CA5.1-2 Definition of the Reliability Index β

The design requirement of the 1986 AISI Specification for such a beam is

$$S_eF_y/FS = (L_s^2\, s/8)(D+L) \qquad \text{(CA5.1-3)}$$

where

S_e = elastic section modulus based on the effective section

$FS = 5/3$ = the factor of safety for bending

F_y = specified yield point

L_s = span length, and s = beam spacing

D and L are, respectively, the code specified dead and live load intensities.

The mean resistance is defined as (Ref. 9)

$$R_m = R_n(P_mM_mF_m) \qquad \text{(CA5.1-4)}$$

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

$$R_n = S_eF_y \qquad \text{(CA5.1-5)}$$

that is, the nominal moment predicted on the basis of the post-buckling strength of the compression flange. The mean values P_m, M_m, and F_m, and the corresponding coefficients of variation V_P, V_M and V_F, are the statistical parameters which define the variability of the resistance:

P_m = the mean ratio of the experimentally determined moment to the predicted moment for the actual material and cross-sectional properties of the test specimens

M_m =mean ratio of the yield point to the minimum specified value

F_m = mean ratio of the actual section modulus to the specified (nominal) value

The coefficient of variation of R equals

$$V_R = \sqrt{V_P^2 + V_M^2 + V_F^2} \qquad \text{(CA5.1 - 6)}$$

The values of these data were obtained from examining the available tests on beams having different compression flanges with partially and fully effective flanges and webs, and from analyzing data on yield point values from tests and cross-sectional dimensions from many measurements. This information was developed in Ref. 10 and is given below:

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

The mean load effect is equal to

$$Q_m = (L_s^2 s/8)(D_m + L_m) \qquad \text{(CA5.1 - 7)}$$

and

$$V_Q = \frac{\sqrt{(D_mV_D)^2 + (L_mV_L)^2}}{D_m + L_m} \qquad \text{(CA5.1 - 8)}$$

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

Load statistics have been analyzed in Ref. 11, where it was shown that $D_m = 1.05D$, $V_D = 0.1$; $L_m = L$, $V_L = 0.25$.

The mean live load intensity equals the code live load intensity if the tributary area is small enough so that no live load reduction is included. Substitution of the load statistics into Eqs. CA5.1-7 and CA5.1-8 gives

$$Q_m = \frac{L_s^2 s}{8}\left(\frac{1.05D}{L} + 1\right)L \qquad \text{(CA5.1 - 9)}$$

$$V_Q = \frac{\sqrt{(1.05D/L)^2 V_D^2 + V_L^2}}{(1.05D/L + 1)} \qquad \text{(CA5.1 -10)}$$

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

From Eq. CA5.1-3 we obtain the nominal design capacity for D/L = 1/5 and FS = 5/3. Thus

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

and, from Eq. CA5.1-2

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

Of itself $\beta = 2.79$ for beams having different compression flanges with partially and fully effective flanges and webs designed by the 1986 AISI Specification means nothing. However, when this is compared to β for other types of cold-formed members, and to β for designs of various types from hot-rolled steel shapes or even for other materials, then it is possible to say that this particular cold-formed steel beam has about an average reliability (Ref. 12).

Basis for LRFD of Cold-Formed Steel Structures

A great deal of work has been performed for determining the values of the reliability index β inherent in traditional design as exemplified by the current structural design specifications such as the AISC Specification for hot-rolled steel, the AISI Specification for cold-formed steel, the ACI Code for reinforced concrete members, etc. The studies for hot-rolled steel are summarized in Ref 9, where also many further papers are referenced which contain additional data. The determination of β for cold-formed steel elements or members is presented in Refs. 10 and 13 through 17., where both the basic research data as well as the β's inherent in the AISI Specification are presented in great detail. The β's computed in the above referenced publications were developed with slightly different load statistics than those of this Commentary, but the essential conclusions remain the same.

The entire set of data for hot-rolled steel and cold-formed steel designs, as well as data for reinforced concrete, aluminum, laminated timber, and masonry walls was re-analyzed in Refs. 11, 12 and 18 by using (a) updated load statistics and (b) a more advanced level of probability analysis which was able to incorporate probability distributions which describe the true distributions more realistically. The details of this extensive reanalysis are presented in Refs. 11, 12 and 18 and also only the final conclusions from the analysis are summarized here:

(1) The values of the reliability index β vary considerably for the different kinds of loading, the different types of construction, and the different types of members within a

given material design specification. In order to achieve more consistent reliability, it was suggested in Ref. 18 that the following values of β would provide this improved consistency while at the same time give, on the average, essentially the same design by the new LRFD method as is obtained by current design for all materials of construction. These target reliabilities β_o for use in LRFD are:

Basic case: Gravity loading, $\beta_o = 3.0$

For connections: $\beta_o = 4.5$

For wind loading: $\beta_o = 2.5$

These target reliability indices are the ones inherent in the load factors recommended in the ANSI/ASCE 7-88 Load Code (Ref. 19).

For simply supported, braced cold-formed steel beams with stiffened flanges, which were designed according to the 1986 AISI allowable stress design specification or to any previous version of this specification, it was shown above that for the representative dead-to-live load ratio of 1/5 the reliability index $\beta = 2.8$. Considering the fact that for other such load ratios, or for other types of members, the reliability index inherent in current cold-formed steel construction could be more or less than this value of 2.8, a somewhat lower target reliability index of $\beta_o = 2.5$ is recommended as a lower limit for the new LRFD Specification. The resistance factors ϕ were selected such that $\beta_o = 2.5$ is essentially the lower bound of the actual β's for members. In order to assure that failure of a structure is not initiated in the connections, a higher target reliability of $\beta_o = 3.5$ is recommended for joints and fasteners. These two targets of 2.5 and 3.5 for members and connections, respectively, are somewhat lower than those recommended by ANSI/ASCE 7-88 (i.e., 3.0 and 4.5, respectively), but they are essentially the same targets as are the basis for the 1986 AISC LRFD Specification (Ref. 3).

(2) The following load factors and load combinations were developed in Refs. 11 and 18 to give essentially the same β's as the target β_o's, and are recommended for use with the ANSI/ASCE 7-88 Load Code (Ref. 19) for all materials, including cold-formed steel:

1. $1.4D$
2. $1.2D+1.6L+0.5(L_r \text{ or } S \text{ or } R_r)$
3. $1.2D+1.6(L_r \text{ or } S \text{ or } R_r)+(0.5L \text{ or } 0.8W)$
4. $1.2D+1.3W+0.5L+0.5(L_r \text{ or } S \text{ or } R_r)$
5. $1.2D+1.5E+(0.5L \text{ or } 0.2S)$
6. $0.9D-(1.3W \text{ or } 1.5E)$

where

D = nominal dead load

E = nominal earthquake load

L = nominal live load due to occupancy;
weight of wet concrete for composite construction

L_r = nominal roof live load

R_r = nominal roof rain load

S = nominal snow load

W = nominal wind load

In view of the fact that the dead load of cold-formed steel structures is usually smaller than that of heavy construction, the first case of load combinations included in Section A5.1.4 of the Specification is (1.4D+L) instead of the ANSI/ASCE value of 1.4D. This AISI requirement is identical with the ANSI/ASCE Code when L = 0.

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

(a) For roof and floor composite construction

$$1.2D_s + 1.6\,C_w + 1.4C$$

where

D_s = weight of steel deck

C_w = nominal weight of wet concrete during construction

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

This suggestion provides safe construction practices for cold-formed steel decks and panels which otherwise may be damaged during construction. The load factor used for the weight of wet concrete is 1.6 because of delivering methods and an individual sheet can be subjected to this load. The use of a load factor of 1.4 for the construction load reflects a general practice of 33% strength increase for concentrated loads.

It should be noted that for the third case of load combinations, the load factor used for the nominal roof live load, L_r, in Section A5.1.4 of the AISI Specification is 1.4 instead of the ANSI/ASCE value of 1.6. The use of a relatively small load factor is because the roof live load is due to the presence of workmen and materials during repair operations and, therefore, can be considered as a type of construction load.

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

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

The load factors and load combinations given above are recommended for use with the LRFD criteria for cold-formed steel. The following portions of this Commentary present the background for the resistance factors ϕ which are recommended in Section A5.1.5 for the various members and connections in Sections B, C, D and E. These ϕ factors are determined in conformance with the load factors given above to approximately provide a target β_o of 2.5 for members and 3.5 for connections, respectively, for the load combination 1.2D+1.6L. For practical reasons it is desirable to have relatively few different resistance factors, and so the actual values of β will differ from the derived targets. This means that

$$\phi R_n = c(1.2D+1.6L) = (1.2D/L+1.6)cL \qquad \text{(CA5.1-11)}$$

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

By assuming D/L = 1/5, Eqs. CA5.1-11 and CA5.1-9 can be rewritten as follows:

$$R_n = 1.84(cL/\phi) \qquad \text{(CA5.1-12)}$$

$$Q_m = (1.05D/L+1)cL = 1.21cL \qquad \text{(CA5.1-13)}$$

Therefore,

$$R_m/Q_m = (1.521/\phi)(R_m/R_n) \qquad \text{(CA5.1-14)}$$

The ϕ factors can be computed from Eq. CA5.1-14 and the following equation by using $V_Q = 0.21$:

$$\text{Target } \beta_o = \frac{\ln(R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}} \qquad \text{(CA5.1 - 15)}$$

A5.2 Yield Point and Strength Increase from Cold Work of Forming

This section is the same as Section A5.2 of the 1986 AISI Specification.

The following statistical data (mean values and coefficients of variation) on material and cross-sectional properties were develpoed in Refs. 13 and 14 for use in the derivation of the resistance factors ϕ:

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

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

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

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

The subscript m refers to mean values. The symbol V stands for coefficient of variation. The symbols M and F are, respectively, the ratio of the mean-to-the nominal material property or cross-sectional property; and F_y, F_{ya}, and F_u are, respectively, the specified minimum yield point, the average yield point including the effect of cold forming, and the specified minimum tensile strength.

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

A6 Reference Documents

The specifications and standards to which this Specification makes reference in various provisions are listed in Section A6 to provide the effective dates of these standards at the time of approval of this Specification.

Additional references which the designer may use for related information are :

1. American Institute of Steel Construction, "Specification for Structural Steel Buildings - Allowable Stress Design and Plastic Design," American Institute of Steel Construction (AISC), One East Wacker Drive, Suite 3100, Chicago, Illinois 60601-2001, June 1, 1989
2. Research Council on Structural Connections, "Allowable Stress Design Specification for Structural Joints Using ASTM A325 or A490 Bolts," Research Council on Structural Connections (RCSC), American Institute of Steel Construction (AISC), One East Wacker Drive, Suite 3100, Chicago, Illinois 60601-2001, November 13, 1985
3. Metal Building Manufacturers Association, "Low Rise Building Systems Manual," Metal Building Manufacturers Association (MBMA), 1230 Keith Building, Cleveland, Ohio 44115

4. Steel Deck Institute, "Design Manual for Composite Decks, Formed Decks, and Roof Decks," Steel Deck Institute, Inc. (SDI), P. O. Box 9506, Canton, Ohio 44711, 1984
5. Steel Joist Institute, "Standard Specifications Load Tables and Weight Tables for Steel Joists and Joist Girders," Steel Joist Institute (SJI), Suite A, 1205 48th Avenue North, Myrtle Beach, South Carolina 29577, 1986
6. Rack Manufacturers Institute, "Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks," Rack Manufacturers Institute (RMI), 8720 Red Oak Boulevard, Suite 201, Charlotte, North Carolina 28210, 1985
7. American Iron and Steel Institute, "Stainless Steel Cold-Formed Structural Design Manual," American Iron and Steel Institute (AISI), 1133 15th Street, N. W., Washington, D. C. 20005, 1974 Edition
8. American Society of Civil Engineers, "ASCE Standard, Specification for the Design and Construction of Composite Slabs," American Society of Civil Engineers (ASCE), 345 East 47th Street, New York, New York 10017, October, 1984
9. American Iron and Steel Institute, "Tentative Criteria for Structural Applications of Steel Tubing and Pipe," American Iron and Steel Institute (AISI), 1133 15th Street, N. W., Washington, D. C. 20005, August, 1976

B. ELEMENTS

B1 Dimensional Limits and Considerations

This section is the same as Section B1 of the AISI Specification for allowable stress design.

B2 Effective Widths of Stiffened Elements

This section is the same as Section B2 of the AISI Specification for allowable stress design.

B3 Effective Widths of Unstiffened Elements

This section is the same as Section B3 of the AISI Specification for allowable stress design.

B4 Effective Widths of Elements with an Edge Stiffener or One Intermediate Stiffener

This section is the same as Section B4 of the AISI Specification for allowable stress design.

Test data to verify the accuracy of the simple lip stiffener design was collected from a number of sources, both university and industry. These tests showed good correlation with the equations in Section B4.2. However, proprietary testing conducted in 1989 revealed that lip lengths with a d/t ratio of greater than 14 gave unconservative results.

A review of the original research data showed a lack of data for simple stiffening lips with d/t ratios greater than 14. Therefore, an upper limit of 14 is recommended pending further research.

B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener

This section is the same as Section B5 of the AISI Specification for allowable stress design.

B6 Stiffeners

B6.1 Transverse Stiffeners

The available experimental data on cold-formed steel transverse stiffeners were evaluated in Ref 10. The test results were compared to the predictions based on the same mathematical models on which the AISI Specification was based . The design provisions in these LRFD criteria are also based on the same mathematical models.

Load capacity in these LRFD criteria is based on the same prediction models as were employed in the formulation of the AISI Specification. A total of 61 tests were examined. The resistance factor $\phi_c = 0.85$ was selected on the basis of the statistical data given in Ref. 10. The corresponding safety indices vary from 3.32 to 3.41. A summary of the information is given in Table CB6.1.

Table CB6.1
Computed Safety Index β for Transverse Stiffeners
($\phi_c = 0.85$)

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

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

B6.2 Shear Stiffeners

The available experimental data on shear strength of the beam webs with shear stiffeners were calibrated in Ref. 10. The ϕ_v factors were taken as the same as those for shear strength of beams (Section C3.2). The statistical data used for determining the ϕ_v factor are given in Ref. 10 as follows:

$$P_m = 1.60; \quad V_P = 0.09$$
$$M_m = 1.00; \quad V_M = 0.06$$
$$F_m = 1.00; \quad V_F = 0.05$$

Based on all these data, the value of β was found to be 4.10 for $\phi_v = 0.90$.

It should be noted that the equations for determining I_{smin} and A_{st} of attached shear stiffeners are based on the studies summarized in Ref. 43.

B6.3 Non-Conforming Stiffeners

This Section is the same as Section B6.3 of the AISI Specification for allowable stress design.

C. MEMBERS

C1 Properties of Sections

This section is the same as Section C1 of the AISI Specification for allowable stress design.

C2 Tension Members

Section C2 of the LRFD criteria was developed on the basis of Section C2 of the AISI Specification for allowable stress design, in which the design of tension members is based only on the yield point of steel.

The resistance factor of $\phi_t = 0.95$ used for tension member design was derived from the procedure described in Section A5.1 of this Commentary and a selected β_o value of 2.5. In the determination of the resistance factor, the following formulas were used for R_m and R_n:

$$R_m = A_n (F_y)_m \tag{CC2-1}$$

$$R_n = A_n F_y \tag{CC2-2}$$

$$\text{i.e. } R_m/R_n = (F_y)_m/F_y \tag{CC2-3}$$

in which A_n is the net area of the cross section, $(F_y)_m$ is equal to $1.10F_y$ as discussed in Section A5.2 of the Commentary. By using $V_M = 0.10$, $V_F = 0.05$ and $V_P = 0$, the coefficient of variation V_R is:

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

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

C3 Flexural Members

C3.1 Strength for Bending Only

Bending strengths of flexural members are differentiated according to whether or not the member is laterally braced. If such members are laterally supported, then they are proportioned according to the nominal section strength (Sec. C3.1.1). If they are laterally unbraced, then the limit state is lateral-torsional buckling (Sec. C3.1.2). For C- or Z-section with the tension flange attached to deck or sheathing and with compression flange laterally unbraced, the bending capacity is less than that of a fully braced member but greater than that of an unbraced member (Sec. C3.1.3).

C3.1.1 Nominal Section Strength

The bending strength of beams with a compression flange having stiffened, partially stiffened, or unstiffened elements is based on the post-buckling strength of the member, and use is made in LRFD of the effective width concept in the same way as in the 1986 AISI Specification. References 5, 6, 7, and 8 provide an extensive treatment of the background research.

The experimental bases for the post-buckling strengths of cold-formed beams were examined in Refs. 8 and 10, where different cases were studied according to the types of compression flanges and the effectiveness of webs.

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

Table CC3.1.1
Computed Safety Index β for Section Bending Strength of Beams
Based on Initiation of Yielding

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
Stiffened or Partially Stiffened Compression Flanges ($\phi_b = 0.95$)								
FF. FW.	8	1.10	0.10	1.0	0.05	1.10543	0.03928	2.76
PF. FW.	30	1.10	0.10	1.0	0.05	1.11400	0.08889	2.65
PF. PW.	5	1.10	0.10	1.0	0.05	1.08162	0.09157	2.53
Unstiffened Compression Flanges ($\phi_b = 0.90$)								
FF. FW.	3	1.10	0.10	1.0	0.05	1.43330	0.04337	4.05
PF. FW.	40	1.10	0.10	1.0	0.05	1.12384	0.13923	2.67
PF. PW.	10	1.10	0.10	1.0	0.05	1.03162	0.05538	2.66

Note: FF. = Fully effective flanges
PF. = Partially effective flanges
FW. = Fully effective webs
PW. = Partially effective webs
For details, see Ref. 10.

The computed values of β for the selected values of $\phi_b = 0.95$ for sections with stiffened or partially stiffened compression flanges and 0.90 for sections with unstiffened compression flanges, and for a dead-to-live load ratio of 1/5 for different cases are listed in Table CC3.1.1. It can be seen that the β values vary from 2.53 to 4.05. In Table CC3.1.1, the values of M_m, V_M, F_m and V_F are the values presented in Sec. A5.2 of this Commentary for the material strength.

C3.1.2 Lateral Buckling Strength

There are not many test data on laterally unsupported cold-formed beams. The available test results are summarized in Ref. 10, and they are compared with predictions from AISI design formulas, theoretical formulas and SSRC formulas.

The statistical data used in Ref. 10 are listed in Table CC3.1.2. The symbol P is the ratio of the tested capacity to the predicted value, M is the ratio of the actual to the specified value of the modulus of elasticity, and F is the ratio of the actual to the nominal sectional properties.

Using the recommended resistance factor $\phi_b = 0.90$, the values of β vary from 2.35 to 3.8. See Table CC3.1.2. It should be noted that the recommended design criteria use some simplified and conservative formulas, which are the same as the allowable stress design rules included in the 1986 AISI Specification.

Table CC3.1.2
Computed Safety Index β for Lateral Buckling Strength of Bending
($\phi_b = 0.90$)

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
1	47	1.0	0.06	1.0	0.05	2.5213	0.30955	3.79
2	47	1.0	0.06	1.0	0.05	1.2359	0.19494	2.48
3	47	1.0	0.06	1.0	0.05	1.1800	0.19000	2.35
4	47	1.0	0.06	1.0	0.05	1.7951	0.21994	3.53
5	47	1.0	0.06	1.0	0.05	1.8782	0.20534	3.80

Note: Case 1 = AISI approach
Case 2 = Theoretical approach with $J = 0.0026$ in.^{4}
Case 3 = SSRC approach with $J = 0.0026$ in.^{4}
Case 4 = Theoretical approach with $J = 0.0008213$ in.^{4}
Case 5 = SSRC approach with $J = 0.0008213$ in.^{4}

C3.1.3 Beams Having One Flange Through-Fastened to Deck or Sheathing

For beams having the tension flange attached to deck or sheathing and the compression flange unbraced, e.g., a roof purlin or wall girt subjected to wind suction, the bending capacity is less than a fully braced member, but greater than an unbraced member. This partial restraint is a function of the rotational stiffness provided by the panel-to-purlin connection. The Specification contains factors that represent the reduction in capacity from a fully braced condition. These factors are based on experimental results obtained for both simple and continuous span purlins (Refs. 20 to 24).

As indicated in Ref. 25, the rotational stiffness of the panel-to-purlin connection is primarily a function of the member thickness, sheet thickness, fastener type and fastener location. For compressed glass fiber blanket insulation of initial thicknesses of zero to six inches, the rotational stiffness was not measurably affected (Ref. 25). To ensure adequate rotational stiffness of the roof and wall systems designed using the Specification provision, Section C3.1.3 explicitly states the acceptable panel and fastener types.

Continuous beam tests were made on three equal spans and the R values were calculated from the failure loads, using as a maximum positive moment, $M = 0.08wL_s^2$.

The provisions of Section C3.1.3 apply to beams on which the tension flange is attached to deck or sheathing and the compression flange is completely unbraced. Beams with discrete point braces on the compression flange may have a bending capacity greater than those completely unbraced. Available data from simple span tests (Refs. 20, 23, 37, 38, and 39) indicate that for members having a lip edge stiffener at an angle of 75 degrees or greater with the plane of the compression flange and braces to the compression flange located at third points or more frequently, member capacities may be increased over those without discrete braces.

In this section, the ϕ_b factor is determined for the load combination of 1.17W-0.9D to approximately provide a target β_o of 1.5 for counteracting loads with a reduction factor of 0.9 applied to the load factor for the nominal wind load. The reasons for using a low target β_o are discussed in Section A5.1 of this Commentary. Based on this type of load combination, the following equations can be established:

$$\phi R_n = c(1.17W - 0.9D) = (1.17 - 0.9D/W)cW \qquad \text{(CC3.1.3-1)}$$

$$Q_m = c(W_m - D_m) \qquad \text{(CC3.1.3-2)}$$

$$V_Q = \frac{\sqrt{(W_m V_W)^2 + (D_m V_D)^2}}{W_m - D_m} \qquad \text{(CC3.1.3-3)}$$

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

Load statistics have been analyzed in Ref. 11, where it was shown that

$$D_m = 1.05D,\ V_D = 0.1;\ W_m = 0.78W,\ V_W = 0.37$$

The substitution of the load statistics into Eqs. CC3.1.3-2 and CC3.1.3-3 gives

$$Q_m = c(0.78W - 1.05D) = (0.78 - 1.05D/W)cW \qquad \text{(CC3.1.3-4)}$$

$$V_Q = \frac{\sqrt{(0.78\text{x}0.37)^2 + (1.05D/W\text{x}0.1)^2}}{0.78 - 1.05D/W} \qquad \text{(CC3.1.3-5)}$$

By assuming D/W = 0.1, Eqs. CC3.1.3-1, CC3.1.3-4, and CC3.1.3-5 can be rewritten as follows:

$$\phi R_n = 1.08cW$$

$$Q_m = 0.675cW = 0.675(\phi R_n/1.08) = 0.625\phi R_n \qquad \text{(CC3.1.3-6)}$$

$$V_Q = 0.43$$

The application of Eqs. CA5.1-2, CA5.1-4, CC3.1.3-6, and CA5.1-6 gives

$$\beta = \frac{\ln(1.6 M_m F_m P_m / \phi)}{\sqrt{V_M^2 + V_F^2 + V_P^2 + V_Q^2}} \qquad \text{(CC3.1.3-7)}$$

or

$$\phi = 1.6(M_m F_m P_m)\exp(-\beta\sqrt{V_M^2 + V_F^2 + V_P^2 + V_Q^2}) \qquad \text{(CC3.1.3-8)}$$

The computed values of β for the selected value of $\phi_b = 0.90$ for different cases are listed in Table CC3.1.3. It can be seen that the β values vary from 1.50 to 1.60 which are satisfactory for the target value of 1.5. In Table CC3.1.3, the values of M_m, V_M, F_m, and V_F are the values presented in Section A5.2 of this Commentary for the material strength and fabrication.

Table CC3.1.3
Computed Safety Index β for Beams Having One Flange
Through-Fastened to Deck or Sheathing
($\phi_b = 0.90$)

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
1	5	1.10	0.10	1.0	0.05	1.1995	0.2991	1.60
2	15	1.10	0.10	1.0	0.05	1.0128	0.1112	1.50
3	5	1.10	0.10	1.0	0.05	1.0466	0.1010	1.58
4	14	1.10	0.10	1.0	0.05	1.0034	0.0689	1.51

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

C3.2 Strength for Shear Only

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

$$V_n = A_w \tau_y = A_w F_y/\sqrt{3} = 0.577 F_y ht \tag{CC3.2-1}$$

in which A_w is the area of the beam web computed by (ht), and τ_y is the yield point of steel in shear, which can be computed by $F_y/\sqrt{3}$.

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

$$V_n = A_w \tau_{cr} = \frac{k_v \pi^2 E A_w}{12(1-\mu^2)(h/t)^2} \tag{CC3.2-2}$$

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

$$V_n = 0.905 E k_v t^3/h \tag{CC3.2-3}$$

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

$$V_n = 0.64 t^2 \sqrt{k_v F_y E} \tag{CC3.2-4}$$

In view of the fact that the appropriate test data on shear are not available, the ϕ_v factors used in Section C3.2 were derived from the condition that the nominal resistance for the LRFD method is the same as the nominal resistance for the allowable stress design method. Thus,

$$(R_n)_{LRFD} = (R_n)_{ASD} \tag{CC3.2-5}$$

Since

$$(R_n)_{LRFD} \geq c(1.2D + 1.6L) / \phi_v \quad \text{(CC3.2-6)}$$

$$(R_n)_{ASD} \geq c(F.S.)(D + L) \quad \text{(CC3.2-7)}$$

the resistance factors can be computed from the following formula:

$$\phi_v = \frac{1.2D + 1.6L}{(F.S.)(D + L)}$$

$$= \frac{1.2(D/L) + 1.6}{(F.S.)(D/L + 1)} \quad \text{(CC3.2-8)}$$

By using a dead-to-live load ratio of D/L = 1/5, the ϕ_v factors computed from the above equation are listed in Table CC3.2 for three different ranges of h/t ratios. The factors of safety are adopted from the AISI Specification for allowable stress design. It should be noted that the use of a small safety factor of 1.44 for yielding in shear is justified by long standing use and by the minor consequences of incipient yielding in shear compared with those associated with yielding in tension and compression.

Table CC3.2

Range of h / t Ratio	F.S. for Allowable Load Design	ϕ_v Factor computed by Eq. CC3.2-8	Recommended ϕ_v Factor
$h/t \leq \sqrt{Ek_v/F_y}$	1.44	1.06	1.00
$\sqrt{Ek_v/F_y} \leq h/t \leq 1.415\sqrt{Ek_v/F_y}$	1.67	0.92	0.90
$h/t > 1.415\sqrt{Ek_v/F_y}$	1.71	0.90	0.90

C3.3 Strength for Combined Bending and Shear

This section is based on the interaction formulas included in Section C3.3 of the AISI Specification for allowable stress design.

C3.4 Web Crippling Strength

The nominal concentrated load or reaction, P_n, is determined by the allowable load given in Section C3.4 of the AISI Specification times the appropriate factor of safety. In this regard, a factor of safety of 1.85 is used for Eqs. C3.4-1, C3.4-2, C3.4-4, C3.4-6 and C3.4-8, and a factor of safety of 2.0 is used for Eqs. C3.4-3, C3.4-5, C3.4-7 and C3.4-9.

On the basis of the statistical analysis of the available test data on web crippling, the values of P_m, M_m, F_m, V_P, V_M and V_F were computed and selected. These values are presented in Table CC3.4 (see Table 76 of Ref. 10). By using $\beta_o = 2.5$, the resistance factors ϕ_w= 0.75 and 0.80 were selected for single unreinforced webs and I-sections, respectively, and is used in Sections A5.1.5 and C3.4. The values of β corresponding to these values of ϕ_w are also given in Table CC3.4.

Table CC3.4
Computed Safety Index β for Web Crippling Strength of Beams

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

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

C3.5 Combined Bending and Web Crippling Strength

This section is based on the interaction formulas included in Section C3.5 of the AISI Specification for allowable stress design.

A total of 551 tests were calibrated for combined bending and web crippling strength. Six different cases were studied. Based on $\phi_w = 0.75$ for single unreinforced webs and $\phi_w = 0.80$ for I-sections, the values of safety indices vary from 2.45 to 3.27 as given in Table CC3.5.

Table CC3.5
Computed Safety Index β for Combined Bending and Web Crippling

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

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

C4 Concentrically Loaded Compression Members

The available experimental data on cold-formed steel concentrically loaded compression members were evaluated in Ref 10. The test results were compared to the predictions based on the same mathematical models on which the AISI Specification was based . The design provisions in these LRFD criteria are also based on the same mathematical models.

Column capacity in these LRFD criteria is based on the same prediction models as were employed in the formulation of the AISI Specification. A total of 264 tests were examined; 14 different cases were studied according to the types of the column, the types of the compression flanges and the failure modes. The resistance factor $\phi_c = 0.85$ was selected on the basis of the statistical data given in Ref. 10. The corresponding safety indices vary from 2.39 to 3.34. A summary of the information is given in Table CC4.

The safety indices were determined from Eq. CA5.1-2 for a D/L ratio of 1/5. Different ϕ_c factors could have been used for different cases.

Table CC4
Computed Safety Index β for Concentrically Loaded Compression Member
(ϕ_c= 0.85)

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
1	5	1.10	0.10	1.0	0.05	1.14610	0.10452	3.13
2	24	1.10	0.10	1.0	0.05	1.05053	0.07971	2.89
3	15	1.10	0.10	1.0	0.05	1.05523	0.07488	2.93
4	3	1.10	0.10	1.0	0.05	1.10550	0.07601	3.11
5	28	1.10	0.10	1.0	0.05	1.04750	0.11072	2.76
6	25	1.10	0.10	1.0	0.05	1.22391	0.21814	2.72
7	9	1.00	0.06	1.0	0.05	0.96330	0.04424	2.39
8	41	1.10	0.10	1.0	0.05	1.19620	0.09608	3.34
9	18	1.10	0.10	1.0	0.05	1.02900	0.08131	2.81
10	12	1.10	0.11	1.0	0.05	1.06180	0.11062	2.77
11	8	1.00	0.06	1.0	0.05	1.15290	0.10544	2.92
12	30	1.10	0.10	1.0	0.05	1.07960	0.15061	2.68
13	14	1.10	0.10	1.0	0.05	1.07930	0.08042	3.00
14	32	1.10	0.10	1.0	0.05	1.08050	0.10772	2.89

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

C5 Combined Axial Load and Bending

The LRFD Specification provide the similar interaction equations as the 1986 Edition of the AISI Specification with 1989 Addendum.

A total of 144 tests were calibrated for combined axial load and bending. Nine different cases were studied according to the types of sections, the stable conditions and the loading conditions. Based on $\phi_c = 0.85$, $\phi_b = 0.95$ or 0.90 for nominal section strength (see Section C3.1.1), and $\phi_b = 0.90$ for lateral buckling strength, the values of safety indices vary from 2.7 to 3.34 as given in Table CC5.

Table CC5

Computed Safety Index β for Combined Axial Load and Bending

(based on $\phi_c = 0.85$)

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
1	18	1.05	0.10	1.0	0.05	1.0367	0.06619	2.70
2	13	1.05	0.10	1.0	0.05	1.0509	0.07792	2.72
3	33	1.05	0.10	1.0	0.05	1.1028	0.09182	2.86
4	18	1.05	0.10	1.0	0.05	1.1489	0.10478	2.96
5	6	1.05	0.10	1.0	0.05	1.1600	0.13000	2.87
6	17	1.05	0.10	1.0	0.05	1.1200	0.09000	2.92
7	10	1.05	0.10	1.0	0.05	1.2300	0.08000	3.34
8	17	1.05	0.10	1.0	0.05	1.0910	0.07950	2.86
9	12	1.05	0.10	1.0	0.05	1.1110	0.11450	2.79

Note:
Case 1 = Locally stable beam-columns, hat sections of Pekoz and Winter (1967)[26]
Case 2 = Locally unstable beam-columns, lipped channel sections of Thomasson (1978)[27]
Case 3 = Locally unstable beam-columns, lipped channel sections of Loughlan (1979)[28]
Case 4 = Locally unstable beam-columns, lipped channel sections of Mulligan and Pekoz (1983)[29]
Case 5 = Locally stable beam-columns, lipped channel sections of Loh and Pekoz (1985)[30] with $e_x \neq 0$ and $e_y = 0$
Case 6 = Locally stable beam-columns, lipped channel sections of Loh and Pekoz (1985) with $e_x = 0$ and $e_y \neq 0$
Case 7 = Locally stable beam-columns, lipped channel sections of Loh and Pekoz (1985) with $e_x \neq 0$ and $e_y \neq 0$
Case 8 = Locally unstable beam-columns, lipped channel sections of Loh and Pekoz (1985) with $e_x = 0$ and $e_y \neq 0$
Case 9 = Locally unstable beam-columns, lipped channel sections of Loh and Pekoz (1985) with $e_x \neq 0$ and $e_y \neq 0$

C6 Cylindrical Tubular Members

Section C6 of the LRFD criteria is based on Section C6 of the AISI Specification for allowable stress design.

The ϕ_b factor of 0.95 used in Section C6.1 for bending is the same as that used in Section C3.1.1, while the ϕ_c factor of 0.85 used in Section C6.2 for compression is the same as that used in Section C4 for concentrically loaded compression members.

D. STRUCTURAL ASSEMBLIES

D1 Built-Up Sections

This section is the same as Section D1 of the AISI Specification for allowable stress design.

D2 Mixed Systems

This section is the same as Section D2 of the AISI Specification for allowable stress design.

D3 Lateral Bracing

This section is the same as Section D3 of the AISI Specification for allowable stress design.

With regard to the footnote for Section D3.2, it should be noted that in conventional metal building roof systems, the roof panels are attached to the top flange of each purlin throughout its length using self-drilling or self-tapping, through-the-sheet, fasteners spaced at approximately 1 foot on center. This panel usually provides sufficient stiffness to prevent the relative movement of the purlins with respect to each other, however, unless external restraint is provided, the system as a whole will tend to move laterally. This restraint or anchorage may consist of members attached to the purlin at discrete locations along the span and designed to carry forces necessary to restrain the system against lateral movement. The design rules for Z-purlin supported roof systems are based on a first order, elastic stiffness model (Reference 31).

D4 Wall Studs and Wall Stud Assemblies

D4.1 Wall Studs in Compression

The AISI design provisions on the compression strength of wall studs were calibrated in Ref. 10. The statistical data used for determining the ϕ_c factor are given in Ref. 10 as follows:

$$P_m = 1.14; \qquad V_P = 0.10$$

$$M_m = 1.10; \qquad V_M = 0.10$$

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

Based on all these data and $\phi_c = 0.85$, the β value was found to be 3.14.

The provisions in this Specification section are given to prevent three possible modes of failure. Provision (a) is for column buckling between fasteners, even if one fastener is missing or otherwise ineffective. Provision (b) contains formulas for nominal axial strengths for overall column buckling. Essential to these provisions is the magnitude of the shear rigidity of the sheathing material.

A table of shearing parameters and an equation for determining the shear rigidity are provided in this Specification. These values are based on the small scale tests described in References 32 and 33. For other types of materials, the sheathing parameters can be determined using the procedures described in these references.

Provision (c) is to insure that the sheathing has sufficient distortion capacity. The procedure involves assuming a value of the stress and checking whether the shear strain at the load corresponding to the stress exceeds the permissible design shear strain of the sheathing material. In principle, the procedure is one of successive approximations.

However, if the smaller of F_e (provision a) or σ_{cr} (provision b) is tried and shown to be satisfactory, then the need for iteration is eliminated.

D4.2 Wall Studs In Bending

The ϕ_b factors for bending strength of wall studs were taken as the same as those for section bending strength of beams (Section C3.1.1). The available test data on wall stud sections with stiffened or partially stiffened compression flanges were calibrated in Ref. 10. The statistical data used for determining the β value are given in Ref. 10 as follows:

$$P_m = 1.27; \qquad V_P = 0.01$$

$$M_m = 1.10; \qquad V_M = 0.10$$

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

Based on all these data and $\phi_b = 0.95$, the β value was found to be 3.37.

D4.3 Wall Studs with Combined Axial Load and Bending

The LRFD criteria provide the same interaction equations as the AISI Specification for allowable stress design.

The available test data on wall studs with combined axial load and bending were calibrated in Ref. 10. The statistical data used for determining the β value are given in Ref. 10 as follows:

$$P_m = 1.19; \qquad V_P = 0.13$$

$$M_m = 1.05; \qquad V_M = 0.10$$

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

Based on $\phi_c = 0.85$ and $\phi_b = 0.95$ for sections with stiffened or partially stiffened compression flanges or $\phi_b = 0.90$ for sections with unstiffened compression flanges, the β value was found to be 2.94.

E. CONNECTIONS AND JOINTS

Section E of the LRFD criteria is based on Section E of the AISI Specification for allowable stress design. This section contains the design provisions for welded connections, bolted connections, shear rupture and connections to other materials.

The resistance factors used for welded and bolted connections were derived for a target reliability index $\beta_o = 3.5$ and the statistical data are summarized in the subsequent sections.

E1 General Provisions

This section is based on Section E1 of the AISI Specification for allowable stress design.

E2 Welded Connections

Section E2 contains the design provisions for arc-welds (groove welds in butt joints, arc spot welds, arc seam welds, fillet welds, and flare groove welds) and resistance welds. The design equations for the nominal strength and the ϕ factors for groove welds in butt joints are the same as that used in the AISC LRFD criteria. (Ref. 3)

For arc spot welds, the ϕ factor of 0.60 used for determining the design shear strength of welds is based on the test data reported in Ref. 34. It gives a β value of 3.55. The statistical data used for deriving the ϕ factor are given in Ref 10 as follows:

$$P_m = 1.17; \quad V_P = 0.22$$

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

$$F_m = 1.00; \quad V_F = 0.10$$

Table CE2-1
Computed Safety Index β for Plate Failure in Weld Connections

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

Note: Case 1 = For $d_a/t \leq 0.815 \sqrt{(E/F_u)}$
Case 2 = For $d_a/t > 1.397 \sqrt{(E/F_u)}$
Case 3 = Longitudinal Loading, $L/t < 25$
Case 4 = Longitudinal Loading, $L/t \geq 25$
Case 5 = Transverse Loading

Table CE2-2
Computed Safety Index β for Tensile Strength of Arc Spot Weld
($\phi = 0.65$)

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

Note: Case 1 is for 1.2D+1.6L ($\beta_o = 3.5$)
Case 2 is for 1.17W-0.9D ($\beta_o = 2.5$)

With regard to the types of the plate failure governed by Eqs. E2.2-2 through Eq. E2.2-4 in the design criteria, ϕ factors were derived from the statistical data presented in Table CE2-1 (Ref. 35). The ϕ factors used for minimum edge distance were taken as the same as those used for bolted connections.

For tension loads on arc spot welds, tests were performed which included variables of steel strengths from 50 to 68.5 ksi, steel sheet thickness ranging from 0.031 to 0.072 inches, galvanized steel sheet, prime painted and galvanized steel plate, and visible diameter of welds ranging from 0.47 to 0.94 inches. Thus, the additional limitations only applicable to arc spot welds in tension were included in the Specification. For use in calculating the tension load on arc spot welds, the tensile strength of the connected sheet, F_u, is limited to a maximum of 60 ksi, although sheet with greater tensile strengths may be used. The development of the equation is contained in Reference 40. The tests were reported in Reference 41.

The statistical data for deriving the ϕ factor are presented in Table CE2-2 (Ref. 42). Two cases were considered in the determination of ϕ factor: (1) 1.2D+1.6L with $\beta_o = 3.5$, and (2) 1.17W-0.9D with $\beta_o = 2.5$ (counteracting loads with a reduction factor of 0.9 applied to the load factor for the nominal wind load). $\phi = 0.65$ was selected for both cases, and the values of β corresponding to the selected ϕ factor are given in Table CE2-2. It can be seen that for both cases, the β values compare satisfactorily to the target reliability indices.

For arc seam welds, the design shear strength of welds is determined from the same ϕ factor used for arc spot welds. The derivation of the ϕ factor for plate tearing is based on the following statistical data (Ref. 10):

$$P_m = 1.00; \qquad V_P = 0.10$$
$$M_m = 1.10; \qquad V_M = 0.10$$
$$F_m = 1.00; \qquad V_F = 0.10$$

For the selected value of $\phi = 0.60$, the value of $\beta = 3.81$.

For fillet welds, the ϕ factors used for longitudinal loading (Eqs. E2.4-1 and E2.4-2) and transverse loading (Eq. E2.4-3) are based on the statistical data presented in Table CE2-1 (Ref. 35).

Similar to the arc spot welds, a ϕ factor of 0.60 is used for the design shear strength of welds.

For flare groove welds, the following statistical data were used to determined the ϕ factors:

(a) Transverse Flare Bevel Welds ($\phi = 0.55$, $\beta = 3.81$)

$P_m = 1.04$; $V_P = 0.17$

$M_m = 1.10$; $V_M = 0.10$

$F_m = 1.00$; $V_F = 0.10$

(b) Longitudinal Flare Bevel Welds ($\phi = 0.55$, $\beta = 3.56$)

$P_m = 0.97$; $V_P = 0.17$

$M_m = 1.10$; $V_M = 0.10$

$F_m = 1.00$; $V_F = 0.10$

For detailed information, see Ref. 10.

For resistance welds, the nominal shear strength is based on the following equation:

R_n = (2.5) x (allowable shear per spot specified in Section E2.6
of the AISI Specification for allowable stress design)

In the above equation, the safety factor is 2.5.

The ϕ factor of 0.65 used in Section E2.6 for the design of resistance welds was determined on the basis of the following statistical data reported in Ref. 10. It gives a β value of 3.71.

$P_m = 1.00$; $V_P = 0.03$

$M_m = 1.10$; $V_M = 0.10$

$F_m = 1.00$; $V_F = 0.10$

E3 Bolted Connections

Section E3 of the LRFD criteria is based on Section E3 of the AISI Specification for allowable stress design. It deals only with the design of bolted connections used for connected parts thinner than 3/16 inch in thickness. For the design of bolted connections using materials equal to or greater than 3/16 inch in thickness, the AISC Specification should be used.

All ϕ factors were computed from the statistical data given in Ref. 10 and $\beta_o = 3.5$. The statistical data used in the study are presented in Table CE3.

The ϕ factors used for the high strength bolts for design shear and tensile strengths are adopted from Ref. 3.

Table CE3
Computed Safety Index β for Bolted Connections

Case	M_m	V_M	F_m	V_F	P_m	V_P	ϕ	β
Minimum Spacing and Edge Distance								
1	1.10	0.08	1.00	0.05	1.13	0.12	0.70	3.75
2	1.10	0.08	1.00	0.05	1.18	0.14	0.70	3.84
3	1.10	0.08	1.00	0.05	0.84	0.05	0.60	3.61
4	1.10	0.08	1.00	0.05	0.94	0.09	0.60	3.90
5	1.10	0.08	1.00	0.05	1.06	0.11	0.70	3.62
6	1.10	0.08	1.00	0.05	1.14	0.19	0.60	3.87
Tension Stress on Net Section								
7	1.10	0.08	1.00	0.05	1.14	0.20	0.65	3.53
8	1.10	0.08	1.00	0.05	0.95	0.21	0.55	3.41
9	1.10	0.08	1.00	0.05	1.04	0.14	0.65	3.63
Bearing Stress on Bolted Connections								
10	1.10	0.08	1.00	0.05	1.08	0.23	0.55	3.65
11	1.10	0.08	1.00	0.05	0.97	0.07	0.65	3.80
12	1.10	0.08	1.00	0.05	1.02	0.20	0.60	3.43
13	1.10	0.08	1.00	0.05	1.05	0.13	0.60	4.06
14	1.10	0.08	1.00	0.05	1.01	0.04	0.70	3.71
15	1.10	0.08	1.00	0.05	0.93	0.05	0.65	3.70
Shear Strength on A307 Bolts								
16	1.28	0.08	1.00	0.05	0.68	0.11	0.65	4.73
17	1.13	0.08	1.00	0.05	0.60	0.10	0.65	3.85
18	1.28	0.08	1.00	0.05	0.75	0.10	0.65	5.23
19	1.36	0.08	1.00	0.05	0.63	0.06	0.65	4.49
20	1.13	0.08	1.00	0.05	0.76	0.06	0.65	5.09

Note: Case 1 = Single shear, with washers, $F_u/F_y \geq 1.15$
Case 2 = Double shear, with washers, $F_u/F_y \geq 1.15$

Table CE3 (Continued)
Computed Safety Index β for Bolted Connections

Case 3	=	Single shear, with washers, $F_u/F_y < 1.15$
Case 4	=	Double shear, with washers, $F_u/F_y < 1.15$
Case 5	=	Single shear, without washers, $F_u/F_y \geq 1.15$
Case 6	=	Single shear, without washers, $F_u/F_y < 1.15$
Case 7	=	$t < 3/16$ in., double shear, with washers
Case 8	=	$t < 3/16$ in., single shear, with washers
Case 9	=	$t < 3/16$ in., single shear, without washers
Case 10	=	$0.024 \leq t < 3/16$ in., double shear, with washers, $F_u/F_y \geq 1.15$
Case 11	=	$0.024 \leq t < 3/16$ in., double shear, with washers, $F_u/F_y < 1.15$
Case 12	=	$0.024 \leq t < 3/16$ in., single shear, with washers, $F_u/F_y \geq 1.15$
Case 13	=	$0.024 \leq t < 3/16$ in., single shear, with washers, $F_u/F_y < 1.15$
Case 14	=	$0.036 \leq t < 3/16$ in., single shear, without washers, $F_u/F_y \geq 1.15$
Case 15	=	$0.036 \leq t < 3/16$ in., double shear, without washers, $F_u/F_y \geq 1.15$
Case 16	=	Double shear, with washers, 3/8 in. diameter
Case 17	=	Double shear, with washers, 3/4 in. diameter
Case 18	=	Single shear, with washers, 3/8 in. diameter
Case 19	=	Single shear, with washers, 1/2 in. diameter
Case 20	=	Single shear, with washers, 3/4 in. diameter

E4 Shear Rupture

Section E4 of the LRFD criteria is based on Section E4 of the AISI Specification for allowable stress design. The ϕ factor used in this section is adopted from Ref. 3.

E5 Connections to Other Materials

Section E5 of the LRFD criteria is based on Section E5 of the AISI Specification for allowable stress design. The ϕ factor used for bearing is adopted from Ref. 3.

F. TESTS FOR SPECIAL CASES

F1 Tests for Determining Structural Performance

The determination of load-carrying capacity of the tested elements, assemblies, connections, or members is based on the same basis for the LRFD design criteria. The correction factor C_P is used in the determination of ϕ factor to account for the influence due to the small number of tests (Ref. 36). It should be noted that when the number of tests is large enough, the effect of correction factor is negligible.

For beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced (subject to wind uplift), the calibration is based on a load combination of 1.17W-0.9D with D/W = 0.1 (see Section C3.1.3 of this Commentary for detailed discussion).

The statistical data needed for the determination of resistance factor are listed in Table F1.

F2 Tests for Confirming Structural Performance

This section is basically the same as Section F2 of the AISI Specification for allowable stress design.

F3 Tests for Determining Mechanical Properties

This section is the same as Section F3 of the AISI Specification for allowable stress design.

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SUPPLEMENTARY INFORMATION

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ON THE MARCH 16, 1991 EDITION OF THE

LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

LRFD Cold-Formed Steel Design Manual-Part III

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PREFACE

This document, *Part III of the LRFD Cold-Formed Steel Design Manual* supplements the *Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members.* It contains two different types of information: (a) design procedures of specification nature which are not included in the *Specification* itself, either because they are infrequently used or are regarded as too complex for routine design, and (b) other information intended to assist users of cold-formed steel.

This *Supplementary Information* should be used in conjunction with the other parts of the *Design Manual*, which include *Commentary (Part II), Illustrative Examples (Part IV), Charts and Tables (Part V), Computer Aids (Part VI), and Test Procedures (Part VII),* in addition to the *Specification (Part I).*

American Iron and Steel Institute
December 1991

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PART III
SUPPLEMENTARY INFORMATION ON THE MARCH 16, 1991, EDITION OF THE LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

SECTION 1 — LINEAR METHOD FOR COMPUTING PROPERTIES OF FORMED SECTIONS

Computation of properties of formed sections may be simplified by using a so-called linear method, in which the material of the section is considered concentrated along the centerline of the steel sheet and the area elements replaced by straight or curved "line elements." The thickness dimension, t, is introduced after the linear computations have been completed.

The total area of the section is found from the relation: Area = L × t, where L is the total length of all line elements.

The moment of inertia of the section, I, is found from the relation: $I = I' \times t$, where I' is the moment of inertia of the centerline of the steel sheet. The section modulus is computed as usual by dividing I or $I' \times t$ by the distance from the neutral axis to *the extreme fiber*, not to the centerline of the extreme element.

First power dimensions, such as x, y, and r (radius of gyration) are obtained directly by the linear method and do not involve the thickness dimension.

When the flat width, w, of a stiffened compression element is reduced for design purposes, the effective design width, b, is used directly to compute the total effective length L_{eff} of the line elements.

The elements into which most sections may be divided for application of the linear method consist of straight lines and circular arcs. For convenient reference, the moments of inertia and location of centroid of such elements are identified in the sketches and formulas in Section 1.1.

The formulas for line elements are exact, since the line as such has no thickness dimension; but in computing the properties of an actual section, where the line element represents an actual element with a thickness dimension, the results will be approximate for the following reasons:

(1) The moment of inertia of a straight actual element about its longitudinal axis is considered negligible.

(2) The moment of inertia of a straight actual element inclined to the axes of reference is slightly larger than that of the corresponding line element, but for elements of like length the error involved is even less than the error involved in neglecting the moment of inertia of the element about its longitudinal axis. Obviously, the error disappears when the element is normal to the axis.

(3) Small errors are involved in using the properties of a linear arc to find those of an actual corner, but with the usual small corner radii the error in the location of the centroid of the corner is of little importance, and the moment of inertia generally negligible. When the mean radius of a circular element is over four times its thickness, as for tubular sections and for sheets with circular corrugations, the errors in using linear arc properties practically disappear.

Using the computed values of I_x, I_y, and I_{xy} the moment of inertia about principal axes of the section can be calculated by the following equation:

$$I_{\substack{Max\\Min}} = \frac{I_x + I_y}{2} \pm \sqrt{\left(\frac{I_y - I_x}{2}\right)^2 + I_{xy}^2}$$

where I_x and I_y are the moment of inertia of the section about x- and y-axis, respectively, and I_{xy} is the product of inertia.

The angle between the x-axis, and the minor axis is

$$\theta = \frac{1}{2}\tan^{-1}\left[\frac{2I_{xy}}{I_y - I_x}\right]$$

Examples of Part IV illustrate the application of the linear method.

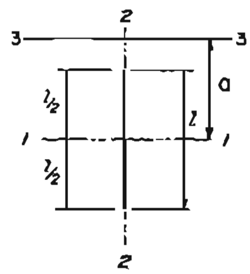
1.1 Properties of Line Elements

1.1.1 Straight Line Elements

Moments of inertia of straight line elements can be calculated using the equations given below:

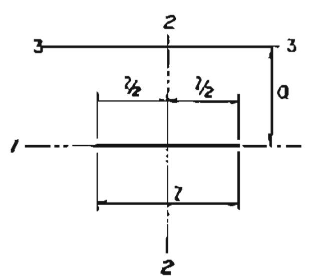
$$I_1 = \frac{l^3}{12}, \qquad I_2 = 0$$

$$I_3 = la^2 + \frac{l^3}{12} = l\left(a^2 + \frac{l^2}{12}\right)$$



$$I_1 = 0, \qquad I_2 = \frac{l^3}{12}$$

$$I_3 = la^2$$

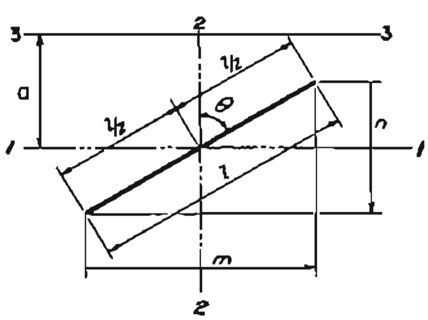


$$I_1 = \left[\frac{\cos^2\theta}{12}\right]l^3 = \frac{ln^2}{12}$$

$$I_2 = \left[\frac{\sin^2\theta}{12}\right]l^3 = \frac{lm^2}{12}$$

$$I_{12} = \left[\frac{\sin\theta\cos\theta}{12}\right]l^3 = \frac{lmn}{12}$$

$$I_3 = la^2 + \frac{ln^2}{12} = l\left(a^2 + \frac{n^2}{12}\right)$$

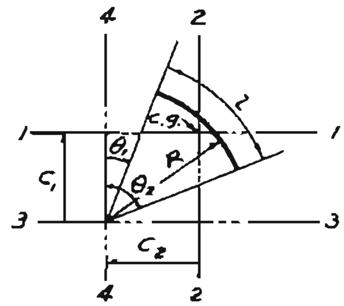


1.1.2 Circular Line Elements

Moments of inertia of circular line elements can be calculated using the equations given below:

θ (expressed in radians) = 0.01745 θ (expressed in degrees and decimals thereof)

$l = (\theta_2 - \theta_1)\, R$



$$c_1 = \frac{\sin\theta_2 - \sin\theta_1}{\theta_2 - \theta_1} R, \qquad c_2 = \frac{\cos\theta_1 - \cos\theta_2}{\theta_2 - \theta_1} R$$

$$I_1 = \left[\frac{\theta_2 - \theta_1 + \sin\theta_2\cos\theta_2 - \sin\theta_1\cos\theta_1}{2} - \frac{(\sin\theta_2 - \sin\theta_1)^2}{\theta_2 - \theta_1}\right] R^3$$

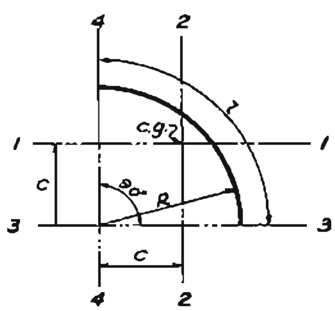
$$I_2 = \left[\frac{\theta_2 - \theta_1 - \sin\theta_2\cos\theta_2 + \sin\theta_1\cos\theta_1}{2} - \frac{(\cos\theta_1 - \cos\theta_2)^2}{\theta_2 - \theta_1}\right] R^3$$

$$I_{12} = \left[\frac{\sin^2\theta_2 - \sin^2\theta_1}{2} + \frac{(\sin\theta_2 - \sin\theta_1)(\cos\theta_2 - \cos\theta_1)}{\theta_2 - \theta_1}\right] R^3$$

$$I_3 = \left[\frac{\theta_2 - \theta_1 + \sin\theta_2\cos\theta_2 - \sin\theta_1\cos\theta_1}{2}\right] R^3,$$

$$I_4 = \left[\frac{\theta_2 - \theta_1 - \sin\theta_2\cos\theta_2 + \sin\theta_1\cos\theta_1}{2}\right] R^3, \quad I_{34} = \left[\frac{\sin^2\theta_2 - \sin^2\theta_1}{2}\right] R^3$$

CASE I: $\theta_1 = 0,\ \theta_2 = 90°$



$l = 1.57\,R,\ c = 0.637\,R$

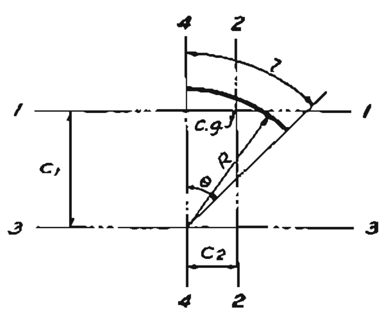
$I_1 = I_2 = 0.149\,R^3$

$I_{12} = -0.137\,R^3$

$I_3 = I_4 = 0.785\,R^3$

$I_{34} = 0.5\,R^3$

CASE II: $\theta_1 = 0,\ \theta_2 = \theta$



$l = \theta R$

$$c_1 = \frac{R\sin\theta}{\theta}$$

$$c_2 = \frac{R(1 - \cos\theta)}{\theta}$$

$$I_1 = \left[\frac{\theta + \sin\theta\cos\theta}{2} - \frac{\sin^2\theta}{\theta}\right] R^3, \quad I_2 = \left[\frac{\theta - \sin\theta\cos\theta}{2} - \frac{(1-\cos\theta)^2}{\theta}\right] R^3$$

$$I_{12} = \left[\frac{\sin^2\theta}{2} + \frac{\sin\theta(\cos\theta - 1)}{\theta}\right] R^3$$

$$I_3 = \left[\frac{\theta + \sin\theta\cos\theta}{2}\right] R^3, \qquad I_4 = \left[\frac{\theta - \sin\theta\cos\theta}{2}\right] R^3$$

$$I_{34} = \left[\frac{\sin^2\theta}{2}\right] R^3$$

1.2 Properties of Sections

Section properties of some sections can be calculated using the equations given below. The following are to be noted:

(1) Three different types of dimensions are employed: capital letters (A) for outside dimensions, lower case barred letters ($\bar{a}$) for centerline dimensions, lower case letters (a) for flat dimensions. The flat dimensions are required to obtain properties such as I where corners are assumed to be round. The centerline dimensions are needed for torsional properties such as C_w where corners are assumed to be square. The outside dimensions are shown because they are the dimensions usually given in tables.

(2) All expressions consider the sections to contain round corners with the exception of those for some torsional properties (m, j and C_w). These expressions are based on a square corner approximation with the exception that round corner values are used for quantities such as moment of inertia which appear in the torsional property expressions. However, allowable stresses calculated by this procedure are sufficiently accurate for routine engineering design of sections with small ratios of corner radius to thickness.

(3) In the moment of inertia calculations, all quantities are accounted for except the moment of inertia of a flat element about its own axis when this is the weak axis. Moments of inertia of corners about their own axis are included to provide for the case of sections with large corner radii.

(4) All expressions are given for the full, unreduced sections.

1.2.1 Equal Angles (Singly-Symmetric) With and Without Lips

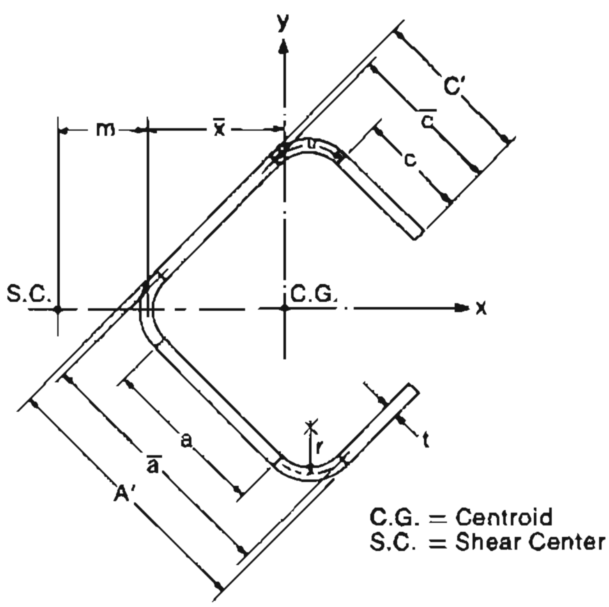


Figure 1.2.1-1
Equal Angle (Singly-Symmetric) With Lips

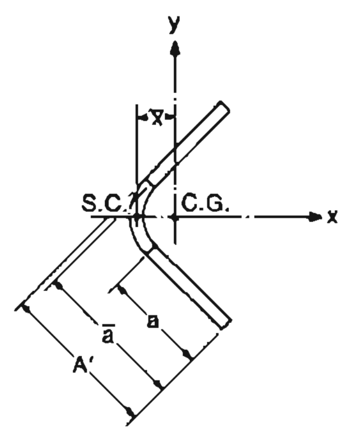


Figure 1.2.1-2
Equal Angle (Singly-Symmetric) Without Lips

NOTE: The x- and y- axes defined in these figures are referred to as the x_2- and y_2- axes in the *Tables of Section Properties, Part V of the Design Manual.*

1. Basic parameters

$$a = A' - [r + t/2 + \alpha(r + t/2)]^*$$
$$\bar{a} = A' - [t/2 + \alpha t/2]$$
$$c = \alpha[C' - (r + t/2)]$$
$$\bar{c} = \alpha[C' - t\ 2]$$
$$u = 1.57r$$

*For sections with lips, $\alpha = 1.0$; for sections without lips, $\alpha = 0$

2. Cross-sectional area

$$A = t[2a + u + \alpha(2c + 2u)]$$

3. Moment of inertia about x-axis

$$I_x = 2t\{a[\tfrac{1}{2}(a/2 + r)^2 + 0.0417a^2] + 0.143r^3 + \alpha[c(\tfrac{1}{2}(r + a - c/2)^2 + 0.0417c^2)] + u(0.707a + 0.898r)^2 + 0.014r^3\}$$

4. Distance between centroid and centerline of corner

$$\overline{x} = \frac{2t}{A}\left\{a(0.353a + 0.293r) + [(u/2)0.102r] + \alpha[c(0.707a + 0.353c + 1.707r) + u(0.707a + r)]\right\}$$

5. Moment of inertia about y-axis

$$I_y = 2t\{a(0.353a + 0.293r)^2 + 0.0417a^3 + 0.015r^3 + \alpha[u(0.707a + r)^2 + c(0.707a + 0.353c + 1.707r)^2 + 0.417c^3 + 0.285r^3]\} - A(\overline{x})^2$$

6. Distance between shear center and centerline of corner

$$m = \frac{t\overline{a}(\overline{c})^2}{3\sqrt{2}I_x}(3\overline{a} - 2\overline{c})$$

7. St. Venant torsion constant

$$J = \frac{t^3}{3}[2a + u + \alpha(2c + 2u)]$$

8. Warping constant $C_w = \dfrac{t^2(\overline{a})^4(\overline{c})^3}{18\,I_x}(4\overline{a} + 3\overline{c})$

9. Distance from centroid to shear center $x_o = -(\overline{x} + m)$*

10. Parameter used to determine elastic critical moment

$$j = \frac{\sqrt{2}t}{48\,I_y}[(\overline{a})^4 + 4(\overline{a})^3(\overline{c}) - 6(\overline{a})^2(\overline{c})^2 + (\overline{c})^4] - x_o$$

1.2.2 Channels (Singly-Symmetric) With and Without Lips and Hat Sections (Singly-Symmetric)

1. Basic parameters

$a = A' - (2r + t)$

$\overline{a} = A' - t$

$b = B' - [r + t/2 + \alpha(r + t/2)]$

$\overline{b} = B' - (t/2 + \alpha t/2)$

$c = \alpha[C' - (r + t/2)]$

$\overline{c} = \alpha(C' - t/2)$

$u = 1.57r$

2. Cross-sectional area $A = t[a + 2b + 2u + \alpha(2c + 2u)]$

3. Moment of inertia about x-axis

Channel: $$I_x = 2t\{0.0417a^3 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.149r^3 + \alpha[0.0833c^3 + \frac{c}{4}(a - c)^2 + u(a/2 + 0.637r)^2 + 0.149r^3]\}$$

Hat Section: $$I_x = 2t\{0.0417a^3 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.149r^3 + \alpha[0.0833c^3 + \frac{c}{4}(a + c + 4r)^2 + u(a/2 + 1.363r)^2 + 0.149r^3]\}$$

*Negative sign indicates x_o is measured in negative x direction.

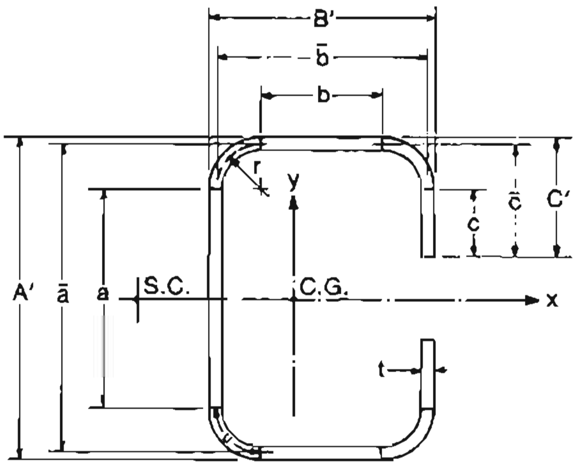


Figure 1.2.2-1
Channel (Singly-Symmetric) With Lips

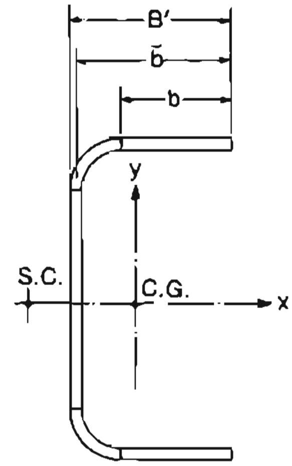
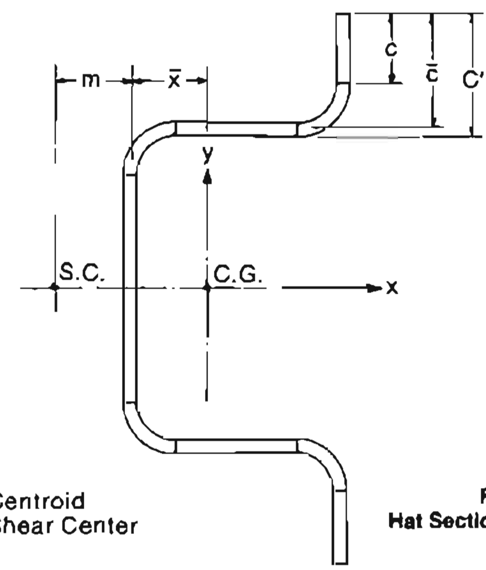


Figure 1.2.2-2
Channel (Singly-Symmetric) Without Lips



C.G. = Centroid
S.C. = Shear Center

Figure 1.2.2-3
Hat Section (Singly-Symmetric)

NOTE: The x- and y- axes defined in these figures are referred to as the x_2- and y_2- axes in the *Tables of Section Properties, Part V of the Design Manual.*

4. Distance between centroid and web centerline

$$\bar{x} = \frac{2t}{A}\{b(b/2 + r) + u(0.363r) + \alpha[u(b + 1.637r) + c(b + 2r)]\}$$

5. Moment of inertia about y-axis

$$I_y = 2t\{b(b/2 + r)^2 + 0.0833b^3 + 0.356r^3 + \alpha[c(b + 2r)^2 + u(b + 1.637r)^2 + 0.149r^3]\} - A(\bar{x})^2$$

6. Distance between shear center and web centerline

$$m = \frac{\bar{b}t}{12I_x}[6\bar{c}(\bar{a})^2 + 3\bar{b}(\bar{a})^2 - 8(\bar{c})^3]$$

7. Distance between centroid and shear center

$x_o = -(\bar{x} + m)$*

8. St. Venant torsion constant

$$J = \frac{t^3}{3}[a + 2b + 2u + \alpha(2c + 2u)]$$

*Negative sign indicates x_o is measured in negative x direction.

9. Warping constant
   a) Channel with lips:

$$C_w = \frac{t^2}{A}\left\{\frac{\bar{x}A(\bar{a})^2}{t}\left[\frac{(\bar{b})^2}{3} + m^2 - m\bar{b}\right]\right.$$

$$+\frac{A}{3t}\left[(m)^2(\bar{a})^3 + (\bar{b})^2(\bar{c})^2(2\bar{c} + 3\bar{a})\right] - \frac{I_x m^2}{t}(2\bar{a} + 4\bar{c})$$

$$+\frac{m(\bar{c})^2}{3}\left[8(\bar{b})^2(\bar{c}) + 2m\left(2\bar{c}(\bar{c} - \bar{a}) + \bar{b}(2\bar{c} - 3\bar{a})\right)\right]$$

$$\left.+\frac{(\bar{b})^2(\bar{a})^2}{6}\left[(3\bar{c} + \bar{b})(4\bar{c} + \bar{a}) - 6(\bar{c})^2\right] - \frac{m^2(\bar{a})^4}{4}\right\}$$

   Channel without lips:

$$C_w = \frac{t\bar{a}^2\bar{b}^3}{12}\,\frac{(3\bar{b} + 2\bar{a})}{6\bar{b} + \bar{a}}$$

   b) Hat section:

$$C_w = \frac{(\bar{a})^2}{4}\left[I_y + (\bar{x})^2A\left(1 - \frac{(\bar{a})^2A}{4I_x}\right)\right]$$

$$+\left[\frac{2(\bar{b})^2t(\bar{c})^3}{3} - \bar{a}(\bar{b})^2(\bar{c})^2t + \frac{(\bar{a})^2\bar{b}t(\bar{c})^3\bar{x}A}{3I_x} - \frac{4(\bar{b})^2t^2(\bar{c})^6}{9I_x}\right]$$

10. Parameter β_w

$$\beta_w = -[0.0833\,(t\bar{x}(\bar{a})^3) + t\,(\bar{x})^3\bar{a}]$$

11. Parameter β_f

$$\beta_f = \frac{t}{2}\left[(\bar{b} - \bar{x})^4 - (\bar{x})^4\right] + \frac{t(\bar{a})^2}{4}\left[(\bar{b} - \bar{x})^2 - (\bar{x})^2\right]$$

12. Parameter β_l

   a) Channel: $\beta_l = 2\bar{c}t(\bar{b} - \bar{x})^3 + \frac{2}{3}t(\bar{b} - \bar{x})\,[(\bar{a}/2)^3 - (\bar{a}/2 - \bar{c})^3]$

   b) Hat section: $\beta_l = 2\bar{c}t\,(\bar{b} - \bar{x})^3 + \frac{2}{3}t(\bar{b} - \bar{x})\,[(\bar{a}/2 + \bar{c})^3 - (\bar{a}/2)^3]$

13. Parameter used in determination of elastic critical moment

$$j = \frac{1}{2I_y}(\beta_w + \beta_f + \beta_l) - x_o$$

1.2.3 I-Sections With Unequal Flanges (Singly-Symmetric) and T-Sections (Singly-Symmetric)

1. Basic parameters
   - $a = A' - [r + t/2 + \alpha(r + t/2)]$*
   - $\bar{a} = A' - (t/2 + \alpha t/2)$
   - $b = B' - (r + t/2)$
   - $\bar{b} = B' - t/2$
   - $c = \alpha[C' - (r + t/2)]$
   - $\bar{c} = \alpha(C' - t/2)$
   - $u = 1.57r$

*For I-sections $\alpha = 1.0$; for T-sections $\alpha = 0$

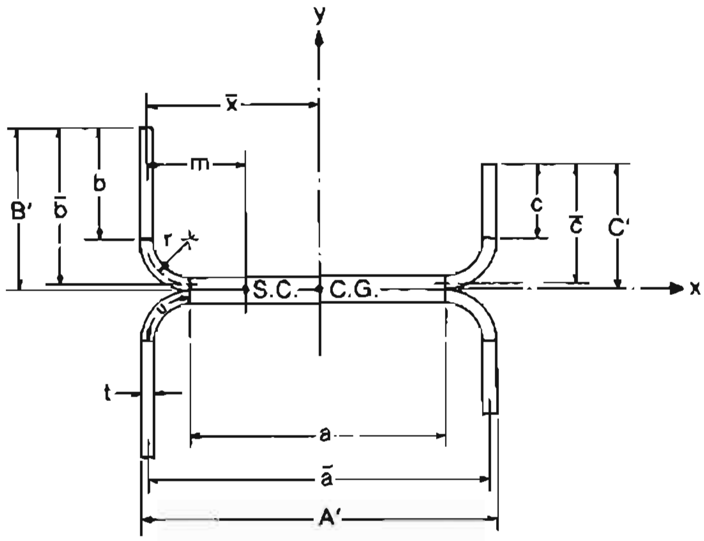


Figure 1.2.3-1
I-Section With Unequal Flanges (Singly-Symmetric)

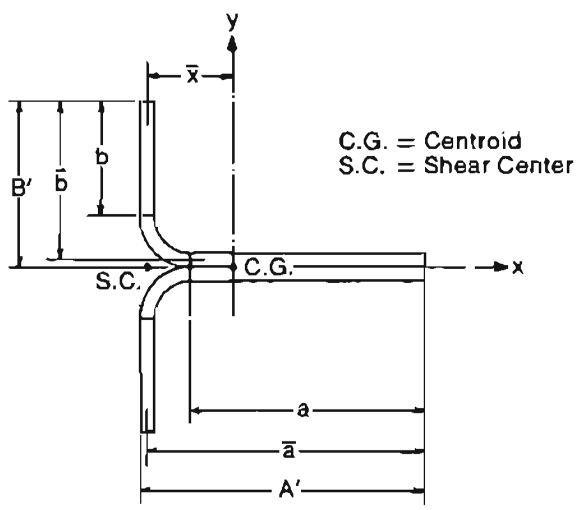


Figure 1.2.3-2
T-Section (Singly-Symmetric)

2. Cross-sectional area

$$A = t[2a + 2b + 2u + \alpha(2c + 2u)]$$

3. Moment of inertia about x-axis

$$I_x = 2t\{b(b/2 + r + t/2)^2 + 0.0833b^3 + u(0.363r + t/2)^2 + 0.149r^3 + \alpha[c(c/2 + r + t/2)^2 + 0.0833b^3 + u(0.363r + t/2)^2 + 0.149r^3]\}$$

4. Distance between centroid and longer flange centerline

$$\bar{x} = \frac{2t}{A}\left\{u(0.363r) + a(a/2 + r) + \alpha[u(a + 1.637r) + c(a + 2r)]\right\}$$

5. Moment of inertia about y-axis

$$I_y = 2t\{0.358r^3 + a(a/2 + r)^2 + 0.0833a^3 + \alpha[u(a + 1.637r)^2 + 0.149r^3 + c(a + 2r)^2]\} - A(\bar{x})^2$$

6. Distance between shear center and longer flange centerline

$$m = \bar{a}\left(1 - \frac{(\bar{b})^3}{(\bar{b})^3 + (\bar{c})^3}\right)$$

7. Distance between shear center and centroid

$$x_o = -(\bar{x} - m)^*$$

8. St. Venant torsion constant

$$J = \frac{2t^3}{3}[a + b + u + \alpha(u + c)]$$

9. Warping constant
For I-Sections the value of C_w is twice the value of each channel if fastened at the middle of the webs; however, if the two channels are continuously welded at both edges of the web to form the I-Section, the warping constants are as follows:

Unlipped I-Sections and T-Sections

$$C_w = \frac{t\bar{a}^2}{12}\left(\frac{8(\bar{b})^3(\bar{c})^3}{(\bar{b})^3 + (\bar{c})^3}\right)$$

For double symmetric, lipped I-Sections

$\bar{c}$ = length of lip, see Figure 1.2.2-1

$$C_w = \frac{t(\bar{b})^2}{3}((\bar{a})^2\bar{b} + 3(\bar{a})^2\bar{c} + 6\bar{a}(\bar{c})^2 + 4(\bar{c})^3)$$

10. Parameter used in determination of elastic critical moment

$$j = \frac{t}{2I_y}\left\{-2\bar{x}\bar{b}((\bar{x})^2 + (\bar{b})^2/3) + 2\bar{c}(\bar{a} - \bar{x})[(\bar{a} - \bar{x})^2 + (\bar{c})^2/3] + \frac{1}{4}[(\bar{a} - \bar{x})^4 - (\bar{x})^4]\right\} - x_o$$

1.2.4 Z-Sections (Point-Symmetric) With and Without Lips

1. Basic parameters

$a = A' - (2r + t)$
$\bar{a} = A' - t$
$b = B' - [r + t/2 + \alpha(r + t/2)]^{**}$
$\bar{b} = B' - (t/2 + \alpha t/2)$
$c = \alpha[C' - (r + t/2)]$
$\bar{c} = \alpha(C' - t/2)$
$u = 1.57r$

2. Cross-sectional area

$$A = t[a + 2b + 2u + \alpha(2c + 2u)]$$

3. Moment of inertia about x-axis

$$I_x = 2t\{0.0417a^3 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.149r^3 + \alpha[0.149r^3 + u(a/2 + 0.637r)^2 + 0.0833c^3 + \frac{c}{4}(a - c)^2]\}$$

4. Moment of inertia about y-axis

$$I_y = 2t\{b(b/2 + r)^2 + 0.0833b^3 + 0.356r^3 + \alpha[c(b + 2r)^2 + u(b + 1.637r)^2 + 0.149r^3]\}$$

* Negative sign indicates x_o is measured in negative x direction.
** For sections with lips $\alpha = 1.0$, for sections without lips, $\alpha = 0$

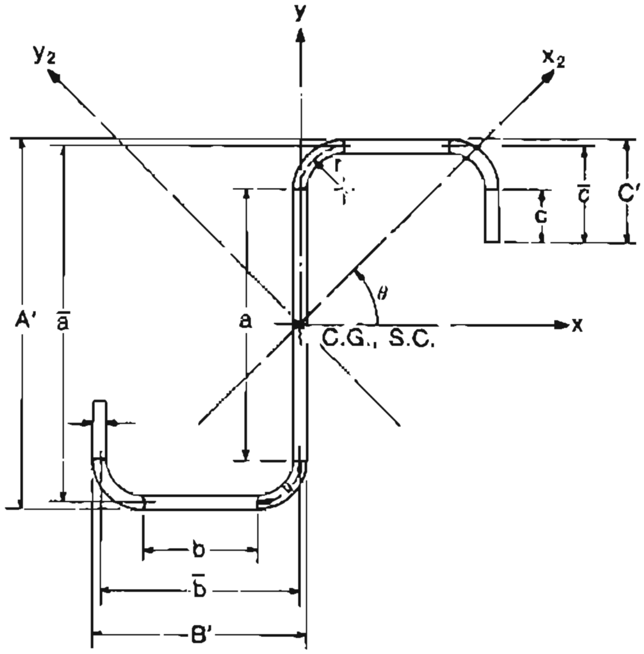


Figure 1.2.4-1
Z-Section (Point-Symmetric) With Lips

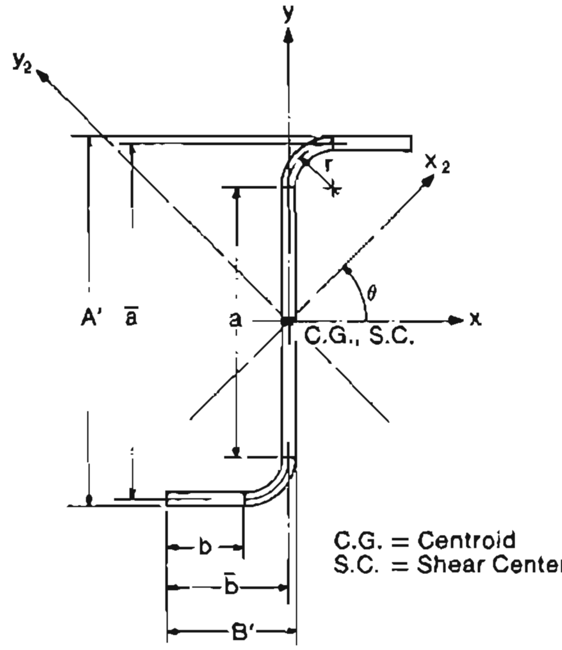


Figure 1.2.4-2
Z-Section (Point Symmetric) Without Lips

5. Product of inertia (See note below)

$$I_{xy} = 2t\{b(a/2 + r)(b/2 + r) + 0.5r^3 + 0.285ar^2 + \alpha[c(2r + b)(a/2 - c/2) - 0.137r^3 + u(b + 1.637r)(0.5a + 0.637r)]\}$$

6. Location of principal axis (See note below)

$$2\theta = \arctan \frac{2I_{xy}}{I_y - I_x}$$

7. Moment of inertia about x_2 axis (See note below)

$$I_{x2} = I_x \cos^2\theta + I_y \sin^2\theta - 2I_{xy} \sin\theta \cos\theta$$

8. Moment of inertia about y_2 axis (See note below)

$$I_{y2} = I_x \sin^2\theta + I_y \cos^2\theta + 2I_{xy} \sin\theta \cos\theta$$

Note: The algebraic signs in Formulas 5, 6, 7 and 8 are correct for the cross-section oriented with respect to the coordinate axes as shown in Figure 1.2.4-1 and Figure 1.2.4-2.

9. Radius of gyration about any axis

$$r = \sqrt{I/A}$$

10. Minimum radius of gyration, about x_2 axis

$$r_{min} = \sqrt{I_{x2}/A}$$

11. St. Venant torsion constant

$$J = \frac{t_3}{3}[a + 2b + 2u + \alpha(2c + 2u)]$$

12. Warping constant

$$C_w = \frac{t(\bar{b})^2}{12}[(\bar{a})^2((\bar{b})^2 + 2\bar{a}\bar{b} + 4\alpha\bar{b}\bar{c} + 6\,\bar{a}\bar{c}) + 4\alpha(\bar{c})^2(3(\bar{a})^2 + 3\bar{a}\bar{b} + 4\bar{b}\bar{c} + 2\bar{a}\bar{c} + (\bar{c})^2)]/(\bar{a} + 2\bar{b} + 2\bar{c})$$

SECTION 2—GRAPHICAL DESIGN AIDS

The use of the charts given in Part V are explained in this section. The charts are given for angle, lipped angle, channel, lipped channel and hat sections.

2.1 Design Aids for Specification Section C3.1.2—Lateral Buckling Strength

Buckling parameters σ_{ex} and σ_t used in *Specification* Sections C3.1.2a, C4.2 and D4.1 can be determined using the Charts given in Part V.

Using the value of Cx given in Charts V1.2, V2.2 and V3.2, parameter σ_{ex} can be determined as follows:

$$\sigma_{ex} = \left(\frac{C_x}{\bar{a}^2}\right)\left(\frac{\bar{a}}{K_xL}\right)^2$$

Using the values of C_T and σ_{t0} given in Charts V1.3, V1.4, V2.3, V2.4, V3.3 and V3.4, parameter σ_t can be determined as follows:

$$\sigma_t = \frac{\sigma_{t0}\bar{a}^2}{t^2}\left(\frac{t}{\bar{a}}\right)^2 + \frac{C_T}{\bar{a}^2}\left(\frac{\bar{a}}{K_tL}\right)^2$$

The lateral buckling moment about the centroidal axis perpendicular to the symmetry axis causing tension on the shear center side of the centroid, M_e can be calculated using the value of W and G_1 determined from Charts V1.6, V1.7, V2.6, V2.7, V3.6 and V3.7 as follows:

$$M_e = A\ \sigma_{ex}\ aWG_1$$

when A is the full cross-sectional area.

2.2 Design Aids for Specification Section C4—Concentrically Loaded Compression Members

2.2.1 Buckling Mode

In *Specifiction* Section C4, F_e is the least of the elastic flexural, torsional or torsional-flexural buckling stress determined according to Sections C4.1 through C4.3. Charts V1.1, V2.1 and V3.1 provide an easy means of determining which buckling mode governs. Based on the cross-sectional dimensions, the governing mode of buckling is determined as explained in these charts. These charts apply when K_y and K_t are equal to 1.0.

2.2.2 Determination of Buckling Parameters

Parameters σ_{ex} and σ_t are determined as described in Section 2.1 above. The torsional-flexural buckling stress F_e needed in *Specification* Section C4.2 can be determined using the value of F given in Charts V1.5, V2.5 and V3.5 as follows:

$$F_e = F\sigma_{ex}$$

2.3 Design Aids for Specification Section D4.1—Wall Studs in Compression

Parameters σ_x and σ_t are determined as described in Section 2.1 above.

SECTION 3—LATERALLY UNBRACED COMPRESSION FLANGES

There are many situations in cold-formed steel structures where a flexural member is so shaped or connected that it will not buckle laterally as a unit, but where the compression flange or flanges themselves are laterally unbraced and can buckle separately by a deflection of the compression flange relative to the tension flange, accompanied by out-of-plane bending of the web and the rest of the section. An example of such a situation is the use of a hat section as a flexural member in such a manner that the "brims" are in compression.

An accurate analysis of such situations is extremely complex and beyond the scope of routine design procedures. The method outlined below is based on considerable simplifications of an exact analysis. Its results have been checked against more than a hundred tests. It has been found that discrepancies rarely exceed 30 percent on the conservative to 20 percent on the unconservative side. Thus, this method allows a reasonable estimate of the design strength to be made which, if desired, can be further improved by test.

The following design procedure was developed based upon tests on individual roof panels or hat-shaped beams. These members were tested as simply supported members with two concentrated loads thus creating a region of uniform moment. Therefore, the design procedure is applicable only to an individual hat-shape type section having its free flange subjected to compression resulting from flexure. It does not apply to the following:

(1) The compression elements of roof panels interconnected by welds, mechanical fasteners or mechanical seams.

(2) A system comprised of flexural members and panels.

For ease of explanation, the design procedure is presented in the following 11 steps:

(1) Determine the location of the neutral axis and define as the "equivalent column" the portion of the beam from the extreme compression fiber to a level which is

$\left(\frac{3c_c - c_t}{12c_c}\right)$ d distance from the extreme compression fiber.

In this expression c_c and c_t are the distances from the neutral axis to the extreme compression and tension fibers respectively; d is the depth of the section.

(2) Determine the distance, y_o, measured parallel to the web, from the centroid of the equivalent column to its shear center. (If the cross section of the equivalent column is of angle or T-shape, its shear center is at the intersection of web and flange; if of channel shape, the location of the shear center is obtained from Section D1.1 of the *Specification*. If the flanges of the channel are of unequal width, for an approximation take w as the mean of the two flange widths, or compute the location of the shear center by rigorous methods.)

(3) To determine the spring constant β, isolate a portion of the member one inch long, apply a force of 0.001 kip perpendicular to the web at the level of the column centroid, and compute the corresponding lateral deflection D of that centroid. Then the spring constant $\beta = 0.001/D$

(4) Calculate $T_o = h/(h + 3.4y_o)$ where h is the distance from the tension flange to the centroid of the equivalent column in inches.

(5) If the flange is laterally braced at two or more points calculate

$P_e = 290{,}000\, I/L^2$, $C = \beta L^2/P_e$, and $L' = 3.7 \sqrt[4]{I(h/t)^3}$

where I = moment of inertia of equivalent column about its gravity parallel to web, in.^{4},
L = unbraced length of equivalent column, in.
If C is smaller than or equal to 30, compute
$P_{cr} = TP_e[1 + \beta L^2/(\pi^2 P_e)]$
If C is larger than 30, compute
$P_{cr} = TP_e(0.60 + 0.635\sqrt{\beta L^2/P_e})$
In both cases, $T = T_o$ if L is equal to or greater than L'
$T = LT_o/L'$ if L is less than L'

(6) If the flange is braced at less than two points, compute

$P_{cr} = T_o\sqrt{4\beta EI}$

(7) Determine the slenderness ratio of the equivalent column

$(KL/r)_{eq} = 490/\sqrt{P_{cr}/A_c}$ where A_c = cross-sectional area of equivalent column.

(8) From paragraph (a) of Section C4 of the Specification, compute the stress, F_n, corresponding to (KL/r)

(9) The design compression bending stress is $F_{b2} = 1.15\, F_n(c_c/y_c)$ with a maximum of F_y

(10) $M_c = F_{b2}S_f$

(11) Go to Equation C3.1.2-1

where

c_c = distance from neutral axis of beam to extreme compression fiber, in.

y_c = distance from neutral axis of beam to centroid of equivalent column, in.

SECTION 4—TORSIONAL-FLEXURAL BUCKLING OF NON-SYMMETRICAL SHAPES

Torsional-flexural buckling of non-symmetrical sections is not covered by the *Specification*. These sections can be designed by taking F_e in Section C4 equal to σ_{TFO}.

The elastic torsional-flexural buckling stress, σ_{TFO}, is less than the smallest of the Euler buckling stresses about the x- and y-axes and the torsional buckling stress. The value of σ_{TFO} can be obtained from the following equation by trial and error:

$$\left(\frac{\sigma_{TFO}^3}{\sigma_{ex}\sigma_{ey}\sigma_t}\right)\rho-\left(\frac{\sigma_{TFO}^2}{\sigma_{ey}\sigma_t}\right)\gamma-\left(\frac{\sigma_{TFO}^2}{\sigma_{ex}\sigma_t}\right)\beta-\left(\frac{\sigma_{TFO}^2}{\sigma_{ex}\sigma_{ey}}\right)+\frac{\sigma_{TFO}}{\sigma_{ex}}+\frac{\sigma_{TFO}}{\sigma_{ey}}+\frac{\sigma_{TFO}}{\sigma_t}=1$$

The following equation may be used for a first approximation:

$$\sigma_{TFO}=\frac{(\sigma_{ex}\sigma_{ey}+\sigma_{ex}\sigma_t+\sigma_{ey}\sigma_t)}{2(\sigma_{ex}\gamma+\sigma_{ey}\beta+\sigma_t)}-\frac{\sqrt{(\sigma_{ex}\sigma_{ey}+\sigma_{ex}\sigma_t+\sigma_{ey}\sigma_t)^2-4(\sigma_{ex}\sigma_{ey}\sigma_t)(\sigma_{ex}\gamma+\sigma_{ey}\beta+\sigma_t)}}{2(\sigma_{ex}\gamma+\sigma_{ey}\beta+\sigma_t)}$$

where

$$\sigma_{ex}=\frac{\pi^2E}{(KL/r_x)^2},\ \text{ksi}$$

$$\sigma_{ey}=\frac{\pi^2E}{(KL/r_y)^2},\ \text{ksi}$$

$$\sigma_t=\frac{1}{I_p}\left[GJ+\frac{\pi^2EC_w}{(KL)^2}\right],\ \text{ksi}$$

ρ = $1-(x_o/r_o)^2-(y_o/r_o)^2$
γ = $1-(y_o/r_o)^2$
β = $1-(x_o/r_o)^2$
E = modulus of elasticity = 29,500 ksi
L = unbraced length of compression member, in.
r_x = radius of gyration of cross section about the x-axis, in.
r_y = radius of gyration of cross section about the y-axis, in.
r_o = polar radius of gyration of cross section about the shear center, in.
I_p = polar moment of inertia about shear center, in.^{4} = $Ar_o^2 = I_x + I_y + Ax_o^2 + Ay_o^2$
G = shear modulus = 11,300 ksi
J = St. Venant torsion constant of the cross section, in.^{4} For open sections composed of n segments of uniform thickness = $(1/3)(l_1t_1^3 + l_2t_2^3 + \ldots + l_nt_n^3)$
C_w = warping constant of torsion of the cross section, in.^{6}
l_i = length of cross section middle line of segment i, in.
t_i = wall thickness of segment i, in.
x_o = distance from shear center to centroid along the principal x-axis, in.
y_o = distance from shear center to centroid along the principal y-axis, in.

For any section, the values of x_o, y_o and C_w can be computed from the following relationships (terms are defined in Figure 4-1):

$$x_o=\frac{1}{I_x}\int_o^l w_o y t\, ds,\ \text{in.}$$

$$y_o = -\frac{1}{I_y}\int_o^l w_c x t \, ds, \text{ in.}$$

$$C_w = \int_o^l (w_o)^2 t ds - \frac{1}{A}\left[\int_o^l w_o t ds\right]^2, \text{ in.}^6$$

where

I_x and I_y = centroidal moments of inertia of the cross section about the principal x- and y-axes, in.^{4}

A = total area of the cross section, in.^{2}

t = wall thickness, in.

$$w_c = \int_o^s R_c \, ds, \text{ in.}^2$$

$$w_o = \int_o^s R_o \, ds, \text{ in.}^2$$

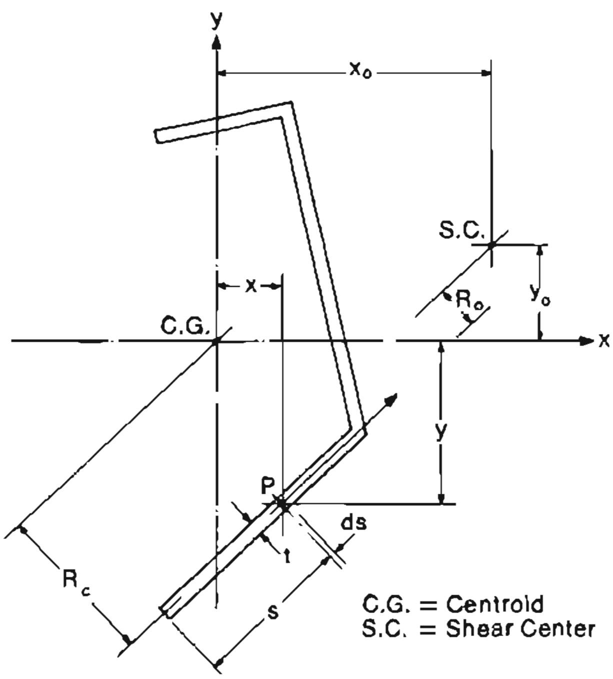


Figure 4-1 Non-Symmetric Cross-Section

x and y = the coordinates measured from the centroid to any point P along the middle line of the cross section, in.

s = distance measured along middle line of cross section from one end to the point P, in.

l = total length of the middle line of the cross section, in.

R_c and R_o = perpendicular distances from the centroid (C.G.) and shear center (S.C.), respectively, to the middle line at P. R_c or R_o is positive if a vector tangent to the middle line at P in the direction of increasing s has a counter-clockwise moment about C.G. or S.C. as shown in Figure 4-1, in.

SECTION 5—SUMMARY OF SCOPE AND PRINCIPLE TENSILE PROPERTIES, ASTM SPECIFICATIONS

ASTM Designation SCOPE (after ASTM)	PRODUCT	GRADE	F_y, ksi (min)	F_u, ksi (min)	Percent elongation in 2 inches (min)	$\frac{F_u}{F_y}$
A36/A36M-84a This specification covers carbon steel shapes, plates, and bars of structural quality for use in riveted, bolted, or welded construction of bridges and buildings, and for general structural purposes. Supplemental requirements are provided where improved notch toughness is important. These shall apply only when specified by the purchaser in the order. When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.	Plates and Bars	—	36	58-80 (range)	23	1.61-2.22
A242/A242M-85 This specification covers high-strength low-alloy structural steel shapes, plates and bars for welded, riveted, or bolted construction intended primarily for use as structural members where savings in weight or added durability are important. These steels have enhanced atmospheric corrosion resistance of approximately two times that of carbon structural steels with copper (Note). This specification is limited to material up to 4 in. [100 mm], inclusive, in thickness. *Note:* Two times carbon structural steel with copper is equivalent to four times carbon structural steel without copper (copper 0.02 max). When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.	Plates and Bars $t \leq 3/4$ in.	—	50	70	18 (in 8 inches)	1.40
A441/A441M-85 This specification covers high-strength low-alloy structural steel shapes, plates, and bars for welded, riveted, or bolted construction but intended primarily for use in welded bridges and buildings where saving in weight or added durability are important. The atmospheric corrosion resistance of this steel is approximately twice that of structural carbon steel. This specification is limited to material up to 8 in. [200 mm] incl. in thickness. When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.	Plates and Bars	—	50	70	18 (in 8 inches)	1.40
A446/A446M-85 This specification covers steel sheet of structural (physical) quality in coils and cut lengths, zinc-coated (galvanized). Material of this quality is intended primarily where mechanical or structural properties of the base metal are specified or required. Such properties or values include those indicated by tension, hardness, or other commonly accepted mechanical tests. Material of this quality can be produced in six grades, A through F, according to the base metal mechanical requirements prescribed in the adjacent table. Structural (physical) quality galvanized sheet is produced with any of the types of coating and coating designations listed in the latest revision of Specification A525 or A525M.	Sheet	A B C D E F	33 37 40 50 80 50	45 52 55 65 82 70	20 18 16 12 — 12	1.36 1.41 1.38 1.30 1.02 1.40
A500-84 This specification covers cold-formed welded and seamless carbon steel round, square, rectangular, or special shape structural tubing for welded, riveted, or bolted construction of bridges and buildings, and for general structural purposes. This tubing is produced in both welded and seamless sizes with a maximum periphery of 64 in. (1626 mm) and a maximum wall of 0.625 in. (15.88 mm). *Note:* Products manufactured to this specification may not be suitable for those applications such as dynamically loaded elements in welded structures, etc., where low-temperature notch-toughness properties may be important.	Round Tubing Shaped Tubing	A B C A B C	33 42 46 39 46 50	45 58 62 45 58 62	25 23 21 25 23 21	1.36 1.38 1.35 1.15 1.26 1.24

ASTM Designation SCOPE (after ASTM)	PRODUCT	GRADE	F_y, ksi (min)	F_u, ksi (min)	Percent elongation in 2 inches (min)	$\frac{F_u}{F_y}$
A529/A529M-85 This specification covers carbon steel plates and bars ½ in. [13 mm] and under in thickness or diameter and Group 1 shapes shown in Table A of Specification A6/A6M of structural quality for use in metal building system frames, trusses, and related riveted, bolted, or welded construction. When used in welded construction, welding procedures shall be suitable for the steel and the intended service. When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.	Plates and Bars	—	42	60-85 (range)	19 (in 8 inches)	1.43-2.02 (range)
A570-85 This specification covers hot-rolled carbon steel sheet and strip of structural quality in cut lengths or coils. This material is intended for structural purposes where mechanical test values are required, and is available in a maximum thickness of 0.229 in. (6.0 mm) except as limited by Specification A568, A568M, A749, or A749M.	Sheet and Strip	30 33 36 40 45 50	30 33 36 40 45 50	49 52 53 55 60 65	21 to 25 18 to 23 17 to 22 15 to 21 13 to 19 11 to 17	1.63 1.58 1.47 1.38 1.33 1.30
A572/A572M-85 This specification covers four grades of high-strength low-alloy structural steel shapes, plates, sheet piling, and bars. Grades 42 [290] and 50 [345] are intended for riveted, bolted, or welded construction of bridges, buildings, and other structures. Grades 60 [415] and 65 [450] are intended for riveted or bolted construction of bridges, or for riveted, bolted, or welded construction in other applications. For welded bridge construction notch toughness is an important requirement. For this or other applications where notch-toughness requirements are indicated, they shall be negotiated between the purchaser and the producer. The use of columbium, vanadium, and nitrogen, or combinations thereof, within the limitations noted in Section 5, shall be at the option of the producer unless otherwise specified. Where designation of one of these elements or combination of elements is desired, reference is made to Supplementary Requirement S1 in which these elements and their common combinations are listed as to type. When such a designation is desired, both the grade and type must be specified. The maximum thicknesses available in the grades and products covered by this specification are shown in Table 1. When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.	Plates and Bars	42 50 60 65	60 65 75 80		24 21 18 17	1.43 1.30 1.25 1.23
A588/A588M-85 This specification covers high-strength low-alloy structural steel shapes, plates, and bars for welded, riveted, or bolted construction but intended primarily for use in welded bridges and buildings where savings in weight or added durability are important. The atmospheric corrosion resistance of this steel is approximately two times that of carbon structural steel with copper. This specification is limited to material up to 8 in. [200 mm] inclusive in thickness. *Note:* Two times carbon structural steel with copper is equivalent to four times carbon structural steel without copper (Cu 0.02 max). When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service will be utilized.	Plates and Bars, $t \leq 4$ inches	—	50	70	21	1.40
A606-85 This specification covers high-strength, low-alloy, hot-and-cold rolled sheet and strip in cut lengths or coils, intended for use in structural and miscellaneous purposes, where savings in weight or added durability are important. These steels have enhanced atmospheric corrosion resistance and are supplied in two types: Type 2 having corrosion resistance at least two times that of plain carbon steel and Type 4 having corrosion resistance at least four times that of plain carbon steel. The degree of corrosion resistance is based on data acceptable to the consumer. *Note:* Type 2 cold-rolled material is intended to replace ASTM Specification A374, for High-Strength Low-Alloy Cold-Rolled Steel Sheets and Strip, and Type 2 hot-rolled material is intended to replace ASTM Specification A375, for High-Strength Low Alloy Hot-Rolled Steel Sheets and Strip, which appear in the 1971 Annual Book of ASTM Standards, Part 3.	Sheet and Strip	Hot Rolled —As Rolled Cut Lengths Hot Rolled —As Rolled Coils Hot Rolled —Annealed or Normalized Cold Rolled	50 45 45 45	70 65 65 65	22 22 22 22	1.40 1.44 1.44 1.44

ASTM Designation SCOPE (after ASTM)	PRODUCT	GRADE	F_y, ksi (min)	F_u, ksi (min)	Percent elongation in 2 inches (min)	$\frac{F_u}{F_y}$
A607-85 This specification covers high-strength, low-alloy columbium, or vanadium hot-rolled sheet and strip, or cold-rolled sheet, or combinations thereof, in either cut lengths or coils, intended for applications where greater strength and savings in weight are important. The material is available as two classes. They are similar in strength level except that Class 2 offers improved weldability and more formability than Class 1. Atmospheric corrosion resistance of these steels is equivalent to plain carbon steels. With copper specified, the atmospheric corrosion resistance is twice that of plain carbon steel. Class 1 material was previously A607 without a class designation.	Sheet and Strip	Class 1				
		45	45	60	Hot-Rolled 23-25 Cold-Rolled 22	1.33
		50	50	65	Hot-Rolled 20-22 Cold-Rolled 20	1.30
		55	55	70	Hot-Rolled 18-20 Cold-Rolled 18	1.27
		60	60	75	Hot-Rolled 16-18 Cold-Rolled 16	1.25
		65	65	80	Hot-Rolled 14-16 Cold-Rolled 15	1.23
		70	70	85	Hot-Rolled 12-14 Cold-Rolled 14	1.21
		Class 2				
		45	45	55	Hot-Rolled 23-25 Cold-Rolled 22	1.22
		50	50	60	Hot-Rolled 20-22 Cold-Rolled 20	1.20
		55	55	65	Hot-Rolled 18-20 Cold-Rolled 18	1.18
		60	60	70	Hot-Rolled 16-18 Cold-Rolled 16	1.17
		65	65	75	Hot-Rolled 14-16 Cold-Rolled 15	1.15
		70	70	80	Hot-Rolled 12-14 Cold-Rolled 14	1.14
A611-85 This specification covers cold-rolled carbon steel sheet, in cut lengths or coils. It includes five strength levels designated as Grade A with yield point 25 ksi (170 MPa) minimum; Grade B with 30 ksi (205 MPa) minimum; Grade C with 33 ksi (230 MPa) minimum; Grade D types 1 and 2 with 40 ksi (275 Mpa) minimum; and Grade E with 80 ksi (550 Mpa) minimum.	Sheet	A B C D, Types 1 and 2 E	25 30 33 40 80	42 45 48 52 82	26 24 22 20 —	1.68 1.50 1.45 1.30 1.02
A715-85 This specification covers high-strength low-alloy, hot-rolled steel sheet and strip having improved formability when compared with steels covered by Specifications A606 and A607. The product is furnished as either cut lengths or coils and is available in four strength levels, Grades 50, 60, 70, and 80 (corresponding to minimum yield point), an in eight types (according to chemical composition). Not all grades are available in all types. The steel is killed, made to a fine grain practice, and includes microalloying elements such as columbium, titanium, vanadium, zirconium, etc. The product is intended for structural and miscellaneous applications where higher strength, savings in weight, improved formability, and weldability are important.	Sheet and Strip	50 60	50 60	60 70	22 to 24 20 to 22	1.20 1.17
A792-85a This specification covers aluminum-zinc alloy-coated steel sheet in coils and cut lengths coated by the hot-dip process. The aluminum-zinc alloy composition by weight is nominally 55% aluminum, 1.6% silicon, and the balance zinc. The product is intended for applications requiring corrosion resistance or heat resistance or both. Aluminum-zinc alloy-coated sheet is available as Commercial Quality, Lock-Forming Quality, and Structural (Physical) Quality. The available grades of Structural Quality are shown in This Table.	Sheet	33 37 40 50 80	33 37 40 50 80	45 52 55 65 82	20 18 16 12 —	1.36 1.41 1.38 1.30 1.03

SECTION 6—SUGGESTED COLD-FORMED STEEL STRUCTURAL FRAMING ENGINEERING, FABRICATION, AND ERECTION PROCEDURES FOR QUALITY CONSTRUCTION

General

Those taking advantage of the economies in building construction afforded by cold-formed steel structural framing are warned to observe the procedures outlined herein to obtain quality construction.

Designing

Building design involving cold-formed steel structural framing for floors, roofs, load-bearing walls, or curtain walls should be performed by, or under the supervision of, registered professional structural engineers.

Detailing

Framing drawings should show size, thickness, type, and spacing of all structural members including bridging and bracing. Large-scale details should be included for all connections either welded or screwed. Details should show the method of anchorage of walls to the foundation.

Fabrication

Manufacturers of cold-formed steel structural members maintain in-house quality control programs. Assembly of components into walls, etc., may be done on the job by the "stick built" method or off the the job by the "panelized" method utilizing assembly jigs. Assemblers must follow details shown on fabrication and/or erection drawings.

Erection

Erection should be performed by experienced mechanics who follow the plans and specifications under the supervision of an experienced foreman or superintendent.

Inspection of Construction

Periodic inspections during the construction phase should be made by a professional structural engineer who either is or represents the design engineer of record.

Sign Off

At the conclusion of the cold-formed steel construction process a final inspection should be made by the engineer of record who should certify that the cold-formed steel framing has been constructed in accordance with the plans and specifications and in accordance with all applicable building codes and regulatory requirements.

ILLUSTRATIVE EXAMPLES

ILLUSTRATIVE EXAMPLES

BASED ON THE
MARCH 16, 1991 EDITION OF THE

LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

LRFD Cold-Formed Steel Design Manual-Part IV

AMERICAN IRON AND STEEL INSTITUTE
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PREFACE

This document, *Part IV of the LRFD Cold-Formed Steel Design Manual*, contains examples intended to illustrate the application of various provisions of the *Specification*.

These *Illustrative Examples* should be used in conjunction with the other parts of the *Design Manual*, which includes *Commentary (Part II)*, *Supplementary Information (Part III)*, and *Design Aids (Part V)*, in addition to the *Specification (Part I)*.

As a general rule, section properties are computed to three significant figures, while dimensions are given to three decimal places. However, in some cases it was impractical to adhere strictly to this guideline.

The weight of the sections is calculated based on steel weighing 40.80 pounds per square foot per inch thickness.

Slight discrepancies should be expected between the calculated section properties given in the examples and the tabulated values given in Part V of the Manual which were calculated by computer.

For the design of compression members, results obtained by utilizing either the graphical or the analytical procedure as outlined in *Part III of the Manual* will differ somewhat. The reason for this is that in the graphical procedure, properties are computed assuming square corners for the section, while the analytical procedure is based on round corners (except for the torsional properties given by C_w j and m which are based on square corners). In general, this will cause only small differences in the results.

The exception occurs when dealing with angle sections. The parameter x which is the distance between the centroid of the section and the centerline of the corner is sensitive to the type of corner utilized. This causes discrepancies in the order of ten percent between the two procedures.

The linear method outlined in *Part III of the Manual* is used for computing the properties of formed sections.

These Illustrative Examples were prepared at Cornell University by graduate students V. Sagan, M. Bou-Shahri, and T. Miller, except for the purlin examples which were prepared by graduate students at the University of Florida.

American Iron and Steel Institute
December 1991

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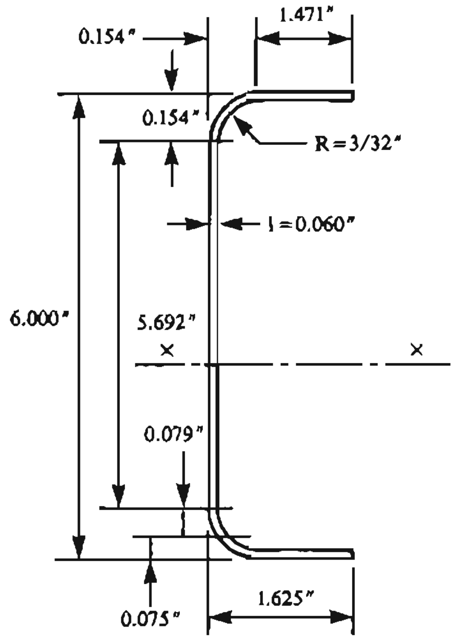
EXAMPLE NO. 1

CHANNEL SECTION



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: 6 x 1.625 x 0.060 channel with unstiffened flanges.
3. Compression flange braced against lateral buckling.
4. Dead load to live load ratio D/L = 1/5 and 1.2D + 1.6L governs the design.

Required:

1. Design flexural strength, $\phi_b M_n$, based on initiation of yielding.
2. Effective moment of inertia based on procedure I for deflection determination at the service moment.

Solution:

1. Calculation of the design flexural strength, $\phi_b M_n$:

   Properties of 90° corners:

   $r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.

Length of arc, $u = 1.57r = 1.57 \times 0.124 = 0.195$ in.

Distance of c.g. from center of radius,

$c = 0.637r = 0.637 \times 0.124 = 0.079$ in.

Computation of I_x:

For the first approximation, assume a compression stress of $f = F_y = 50$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange: $k = 0.43$ (unstiffened compression element) (Section B3.1)

$w/t \quad = 1.471/0.060 = 24.52 < 60$ OK (Section B1.1-(a)-(3))

$\lambda \quad = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{0.43})(24.52)\sqrt{50/29500} = 1.619 > 0.673$

$\rho \quad = [1-(0.22/\lambda)]/\lambda$ (Eq. B2.1-3)

$= [1-(0.22/1.619)]/1.619 = 0.534$

$b \quad = \rho w$ (Eq. B2.1-2)

$= 0.534 \times 1.471$

$= 0.786$ in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Web	5.692	3.000	17.076	51.228	15.368
Upper Corner	0.195	0.075	0.015	0.001	—
Lower Corner	0.195	5.925	1.155	6.846	—
Compression Flange	0.786	0.030	0.024	0.001	—
Tension Flange	1.471	5.970	8.782	52.428	—
Sum	8.339		27.052	110.504	15.368

Distance from top fiber to x-axis is

$y_{cg} \quad = 27.052/8.339 = 3.244$ in.

Since the distance of top compression fiber from neutral axis is greater than one half the beam depth, a compression stress of 50 ksi will govern as assumed (i.e., initial yield is in compression).

To check if web is fully effective (Section B2.3):

$f_1 \quad = [(3.244 - 0.154)/3.244]\times 50 = 47.63$ ksi(compression)

f_2 = -[(2.756 - 0.154)/3.244]x50 = -40.10 ksi(tension)

ψ = f_2/f_1 = -40.10/47.63 = -0.842

k = $4+2(1-\psi)^3+2(1-\psi)$ (Eq. B2.3-4)

= $4+2[1 - (-0.842)]^3+2[1 - (-0.842)]$

= 20.184

h = w = 5.692 in., h/t = w/t = 5.692/0.060 = 94.87

h/t = 94.87 < 200 OK (Section B1.2-(a))

λ = $(1.052/\sqrt{20.184})(94.87)\sqrt{47.63/29500}$ = 0.893>0.673

ρ [1-(0.22/0.893)]/0.893 = 0.844

b_e = 0.844 x 5.692 = 4.804 in.

b_2 = $b_e/2$ (Eq. B2.3-2)

= 4.804/2 = 2.402 in.

b_1 = $b_e/(3-\psi)$ (Eq. B2.3-1)

= 4.804/{3 - (-0.842)}= 1.250 in.

b_1+b_2 = 1.250 + 2.402 = 3.652 in.

Compression portion of the web calculated on the basis of the effective section = y_{cg} - 0.154 = 3.244 - 0.154 = 3.090 in.

Since b_1+b_2 = 3.652 in. > 3.090 in., b_1+b_2 shall be taken as 3.090 in. This verifies the assumption that the web is fully effective.

I'_x = $Ly^2+I'_1=Ly^2_{cg}$

= $110.504 + 15.368 - 8.339(3.244)^2$

= 38.116 $in.^3$

Actual I_x = $I'_x t$

= 38.116x0.060

= 2.287 $in.^4$

S_e = I_x/y_{cg}

= 2.287/3.244

= 0.705 $in.^3$

M_n = S_eF_y (Eq. C3.1.1-1)

= 0.705x50

= 35.25 kip-in.

ϕ_b = 0.90 (Section C3.1.1)

$\phi_b M_n$ = 0.90x35.25 = 31.73 kip-in.(positive bending)

2. Calculation of the effective moment of inertia based on procedure I for deflection determination at the service moment M_s:

$\phi_b M_n := 1.2M_{DL} + 1.6M_{LL}$

$= [1.2(M_{DL}/M_{LL})+1.6]M_{LL}$

$= [1.2(1/5) + 1.6]M_{LL} = 1.84\ M_{LL}$

$M_{LL} = \phi_b M_n/1.84 = 31.73/1.84 = 17.24$ kip-in.

$M_s = M_{DL} + M_{LL}$

$= (1/5+1)M_{LL}$

$= 1.2(17.24) = 20.69$ kip-in.

where

M_{DL} = Moment determined on the basis of nominal dead load

M_{LL} = Moment determined on the basis of nominal live load

The procedure is iterative: one assumes the actual compressive stress f under this service moment M_s. Knowing f, one proceeds as usual to obtain S_e and checks to see if ($f \times S_e$) is equal to M_s as it should. If not, reiterate until one obtains the desired level of accuracy. (Section B2.1-(b)-(1))

a. For the first iteration, assume a compression stress of $f = F_y/2 = 25$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

$\lambda = (1.052/\sqrt{0.43})(24.52)\sqrt{25/29500} = 1.145 > 0.673$ (Eq. B2.1-4)

$\rho = [1 - (0.22/1.145)]/1.145 = -.706$ (Eq. B2.1-3)

$b_d = \rho w$ (Eq. B2.1-6)

$= 0.706 \times 1.471 = 1.039$ in.

Effective section properties about x-axis:

$L = 8.339 - 0.786 + 1.039 = 8.592$ in.

$Ly = 27.052 - 0.024 + 1.039\text{x}0.030 = 27.059$ in.^{2}

$Ly^2 = 110.504 - 0.001 + 1.039(0.030)^2 = 110.504$ in.^{3}

$I'_1 = 15.368$ in.^{3}

$y_{cg} = 27.059/8.592 = 3.149$ in. greater than one half beam depth. Thus top compression fiber controls in determination of S_e.

To check if web is fully effective (Section B2.3-(a),(b)):

$f_1 = [(3.149 - 0.154)/3.149]\text{x}25 = 23.78$ ksi

$f_2 = -[(2.851 - 0.154)/3.149]\text{x}25 = -21.41$ ksi

$\psi = -21.41/23.78 = -0.900$

$k \quad = 4 + 2[1 - (-0.900)]^3 + 2[1 - (-0.900)] = 21.518$

$\lambda \quad = (1.052/\sqrt{21.518})(94.87)\sqrt{23.78/29500} = 0.611 < 0.673$

$b_e \quad = w$ (Eq. B2.1-1)

$\quad = 5.692$ in.

$b_2 \quad = 5.692/2 = 2.846$ in. (Eq. B2.3-2)

$b_1 \quad = 5.692/[3 - (-0.900)] = 1.459$ in. (Eq. B2.3-1)

Compression portion of the web calculated on the basis of the effective section = 3.149 - 0.154 = 2.995 in.

Since b_1+b_2 = 4.305 in. > 2.995 in., b_1+b_2 shall be taken as 2.995 in.. This verifies the assumption that the web is fully effective.

$I'_x \quad = 110.504 + 15.368 - 8.592(3.149)^2$

$\quad = 40.672 \text{ in.}^3$

Actual $I_x \quad = 40.672 \times 0.060$

$\quad = 2.440 \text{ in.}^4$

$S_e \quad = 2.440/3.149 = 0.775 \text{ in.}^3$

$M \quad = f \times S_e = 25 \times 0.775$

$\quad = 19.38 \text{ kip-in.} < M_s = 20.69 \text{ kip-in.}$

Need to do another iteration and also to increase f.

b. After several iterations, assume f = 27.01 ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

$\lambda \quad = (1.052/\sqrt{0.43})(24.52)\sqrt{27.01/29500} = 1.190 > 0.673$

$\rho \quad = [1 - (0.22/1.190)]/1.190 = .685$

$b_d \quad = .685 \times 1.471 = 1.008$ in.

Effective section properties about x-axis:

$L \quad = 8.339 - 0.786 + 1.008 = 8.561$ in.

$Ly \quad = 27.052 - 0.024 + 1.008 \times 0.030 = 27.058 \text{ in.}^2$

$Ly^2 \quad = 110.504 - 0.001 + 1.008(0.030)^2 = 110.504 \text{ in.}^3$

$I'_1 \quad = 15.368 \text{ in.}^3$

$y_{cg} = 27.058/8.561 = 3.161$ in. greater than one half beam depth. Thus top compression fiber controls in determination of S_e.

To check if web is fully effective:

$f_1 \quad = [3.161 - 0.154)/3.161] \times 27.01 = 25.69$ ksi

f_2 = -[(2.839 - 0.154)/3.161]x27.01 = -22.94 ksi

ψ = -22.94/25.69 = 0.893

k = $4 + 2[1 - (-0.893)]^3+2[1 - (-0.893)] = 21.353$

λ = $(1.052/\sqrt{21.353})(94.87)\sqrt{25.69/29500} = 0.637 < 0.673$

b_e = 5.692 in.

b_2 = 5.692/2 = 2.846 in.

b_1 = 5.692/[3 - (-0.893)] = 1.462 in.

Compression portion of the web calculated on the basis of the effective section = 3.161 - 0.154 = 3.007 in..

Since b_1+b_2 = 4.308 in. > 3.007 in., b_1+b_2 shall be taken as 3.004 in.. This verifies the assumption that the web is fully effective.

I'_x = $110.504 + 15.368 - 8.561(3.161)^2$

= 40.331 in.^{3}

Actual I_x = 40.331 x 0.060

= 2.420 in.^{4}

S_e = 2.420/3.161 = 0.766 in.^{3}

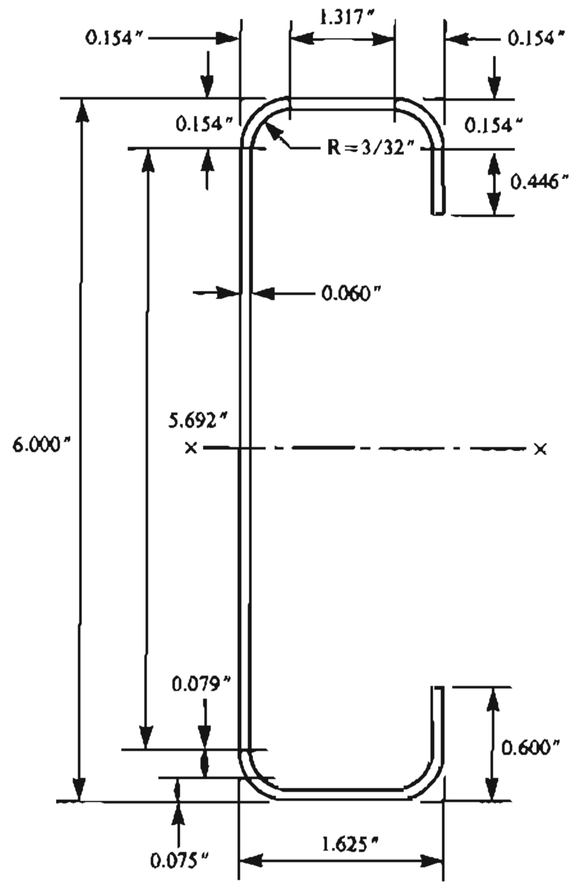
M = f x S_e = 27.01 x 0.766

= 20.68 kip-in. ≈ M_s OK

Thus I_x = 2.420 in.^{4} using procedure I for deflection determination.

EXAMPLE NO. 2

C-SECTION



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: 6 x 1.625 x 0.060 channel with stiffened flanges.
3. Compression flange braced against lateral buckling.
4. Dead load to live load ratio D/L = 1/5 and 1.2D + 1.6L governs the design.

Required:

1. Design flexural strength, $\phi_b M_n$, based on initiation of yielding.
2. Effective moment of inertia based on procedure I for deflection determination at the service moment.

Solution:

1. Calculation of the design flexural strength, $\phi_b M_n$:
   Properties of 90° corners:

$r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.

Length of arc, $u = 1.57r = 1.57 \times 0.124 = 0.195$ in.

Distance of c.g. from center of radius,

$c = 0.637r = 0.637 \times 0.124 = 0.079$ in.

Computation of I_x:

For the first approximation, assume a compression stress of $f = F_y = 50$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

$w = 1.317$ in.

$w/t = 1.317/0.060 = 21.95$

$S = 1.28 \sqrt{E/f}$ (Eq. B4-1)

$= 1.28 \sqrt{29500/50} = 31.09$

$S/3 = 10.36 < (w/t) = 21.95 < S = 31.09$

$I_a = 399t^4 \{[(w/t)/S] - 0.33\}^3$ (Eq. B4.2-6)

$= 399(0.060)^4 [(21.95/31.09) - 0.33]^3$

$= 0.000275$ in.^{4}

$D = 0.450$ in.

$d = 0.296$ in., $d/t = 0.296/0.060 = 4.93 < 14$ OK (Section B4 of the Commentary)

$I_s = d^3t/12$ (Eq. B4-2)

$I_s = (0.296)^3(0.060)/12 = 0.000130$ in.^{4}

$D/w = 0.450/1.317 = 0.342$, $0.25 < D/w = 0.342 < 0.80$ (Eq. B4-2)

$k = [4.82 - 5(D/w)](I_s/I_a)^n + 0.43 \leq 5.25 - 5(D/w)$ (Eq. B4.2-9)

$n = 1/2$

$[4.82 - 5(0.342)](0.000130/0.000275)^{1/2} + 0.43 = 2.568$

$5.25 - 5(0.342) = 3.540 > 2.568$

$k = 2.568$

Since $I_s < I_a$, the stiffener is considered a simple lip.

$w/t = 21.95 < 60$ OK (Section B1.1-(a)-(1))

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{2.568})(21.95)\sqrt{50/29500} = 0.59 < 0.673$

$b = w$ (Eq. B2.1-1)

$= 1.317$ in. (i.e. compression flange fully effective)

Compression (upper) stiffener:

$k = 0.43$ (unstiffened compression element)

$d/t = 4.93$

Also conservatively assume f = 50 ksi as for top compression fiber.

$\lambda = (1.052/\sqrt{0.43})(4.93)\sqrt{50/29500} = 0.32 < 0.673$

therefore,

$d'_s = d = 0.296$ in.

$d_s = d'_s(I_s/I_a) \le d'_s$ (Eq. B4.2-11)

$= 0.296(0.000130/0.000275)$

$= 0.140$ in. < 0.296 in.

$d_s = 0.140$ in. (i.e. compression stiffener is not fully effective)

Effective section properties about x-axis:

Element		L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Web		5.692	3.000	17.076	51.228	15.368
Upper Corners	2x0.195 =	0.390	0.075	0.029	0.002	—
Lower Corners	2x0.195 =	0.390	5.925	2.311	13.691	—
Compression Flange		1.317	0.030	0.040	0.001	—
Upper Stiffener		0.140	0.224	0.031	0.007	—
Tension Flange		1.317	5.970	7.862	46.939	—
Lower Stiffener		0.296	5.698	1.687	9.610	0.002
Sum		9.542		29.036	121.478	15.370

Distance from top fiber to x-axis is

$y_{cg} = 29.036/9.542 = 3.043$ in.

Since the distance of top compression fiber from neutral axis is greater than one half the beam depth, a compression stress of 50 ksi will govern as assumed (i.e., initial yield is in compression).

To check if web is fully effective (Section B2.3):

$f_1 = [(3.043-0.154)/3.043]x50 = 47.47$ ksi(compression)

$f_2 = -[(2.957-0.154)/3.043]x50 = -46.06$ ksi(tension)

$\psi = f_2/f_1 = -46.06/47.47 = -0.970$

$k = 4+2(1-\psi)^3+2(1-\psi)$

$= 4+2[1-(-0.970)]^3+2[1-(-0.970)]$

$= 23.231$

h = w = 5.692 in., h/t = w/t = 5.692/0.060 = 94.87

h/t = 94.87 < 200 OK (Section B1.2-(a))

$\lambda = (1.052/\sqrt{23.231})(94.87)\sqrt{47.47/29500} = 0.831 > 0.673$

$\rho = [1-(0.22/\lambda)]/\lambda$ (Eq. B2.1-3)

$= [1 - (0.22/0.831)]/0.831 = 0.885$

$b_e = \rho w$ (Eq. B2.1-2)

$= 0.885 \times 5.692 = 5.037$ in.

$b_2 = b_e/2$ (Eq. B2.3-2)

$= 5.037/2 = 2.519$ in.

$b_1 = b_e/(3-\psi)$ (Eq. B2.3-1)

$= 5.037/[3-(-0.970)] = 1.269$ in.

$b_1+b_2 = 1.269 + 2.519 = 3.788$ in.

Compression portion of the web calculated on the basis of the effective section = $y_{cg} - 0.154 = 3.043 - 0.154 = 2.889$ in.

Since $b_1+b_2 = 3.788$ in. > 2.889 in., b_1+b_2 shall be taken as 2.889 in. This verifies the assumption that the web is fully effective.

$I'_x = Ly^2+I'_1-Ly^2_{cg}$

$= 121.478 + 15.370 - 9.542(3.043)^2$

$= 48.491$ in.^{3}

Actual $I_x = I'_x t$

$= 48.491 \times 0.060$

$= 2.909$ in.^{4}

$S_e = I_x/y_{cg} = 2.909/3.043$

$= 0.956$ in.^{3}

$M_n = S_e F_y$ (Eq. C3.1.1-1)

$= 0.956 \times 50$

$= 47.80$ kip-in.

$\phi_b = 0.95$ (Section C3.1.1)

$\phi_b M_n = 0.95 \times 47.80 = 45.41$ kip-in.

2. Calculation of the effective moment of inertia based on procedure I for deflection determination at the service moment M_s:

$\phi_b M_n = 1.2M_{DL} + 1.6M_{LL}$

$= [1.2(M_{DL}/M_{LL})+1.6]M_{LL}$

$= [1.2(1/5)+1.6]M_{LL}$

$= 1.84M_{LL}$

$M_{LL} = \phi_b M_n/1.84 = 45.41/1.84 = 24.68$ kip-in.

$M_s = M_{DL} + M_{LL}$

$= (1/5+1)M_{LL}$

$= 1.2(24.68) = 29.62$ kip-in.

where

M_{DL} = Moment determined on the basis of nominal dead load

M_{LL} = Moment determined on the basis of nominal live load

The procedure is iterative: one assumes the actual compressive stress f under this service moment M_s. Knowing f, one proceeds as usual to obtain S_e and checks to see if ($f \times S_e$) is equal to M_s as it should. If not, reiterate until one obtains the desired level of accuracy. (Section B2.1-(b)-(1))

a. For the first iteration, assume a compression stress of $f = F_y/2 = 25$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

$S = 1.28\sqrt{29500/25} = 43.97$ (Eq. B4-1)

$S/3 = 14.66 < (w/t) = 21.95 < S = 43.97$

$I_a = 399(0.060)^4[(21.95/43.97)-0.33]^3$ (Eq. B4.2-6)

$= 0.000025$ in.^{4}

$I_s/I_a = 0.000130/0.000025 = 5.20$

$k = [4.82-5(0.342)](5.20)^{1/2}+0.43 = 7.522 > 3.540$ (Eq. B4.2-9)

$k = 3.540$

$\lambda = (1.052/\sqrt{3.540})(21.95)\sqrt{25/29500} = 0.357 < 0.673$ (Eq. B2.1-4)

$b_d = 1.317$ in. (i.e. compression flange fully effective)

Compression (upper) stiffener:

Again assume conservatively f = 25 ksi as in top compression fiber

$\lambda = (1.052/\sqrt{0.43})(4.93)\sqrt{25/29500} = 0.230 < 0.673$

therefore $d'_s = 0.296$ in.

Since $I_s/I_a = 5.20 > 1.0$, it follows that $d_s = d'_s = 0.296$ in. (i.e. compression stiffener fully effective).

Thus, one concludes that the section is fully effective.

$y_{cg} = 6/2 = 3.000$ in. (from symmetry)

Full section properties about x axis:

Element	L (in.)	y Distance from Centerline of Section (in.)	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Web	5.692	—	—	15.368
Stiffeners	2 x 0.296 = 0.592	2.698	4.309	0.004
Corners	4 x 0.195 = 0.780	2.925	6.673	—
Flanges	2 x 1.317 = 2.634	2.970	23.234	—
Sum			34.216	15.372

Since section is singly symmetric about x-axis and fully effective, top compression fiber (and also bottom tension fiber) may be used in computing S_e.

To check if web is fully effective:

f_1 = [(3.000-0.154)/3.000]x25 = 23.72 ksi(compresion)

f_2 = -23.72 ksi (tension)

ψ = -23.72/23.72 = -1.000

k = $4+2\,[1-(-1)]^3+2\,[1-(-1)] = 24.000$

λ = $(1.052/\sqrt{24})(94.87)\sqrt{23.72/29500} = 0.578 < 0.673$

b_e = w (Eq. B2.1-1)

= 5.692 in.

b_2 = 5.692/2 = 2.846 in.

b_1 = 5.692/ [3-(-1)] = 1.423 in. (Eq. B2.3-1)

$b_1 + b_2$ = 4.269 in.

Compression portion of the web = 3.000 - 0.154 = 2.846 in.

Since b_1+b_2 = 4.269 in. > 2.846 in., b_1+b_2 shall be taken as 2.846 in.. This verifies the assumption that the web is fully effective.

I'_x = 34.216 + 15.372 = 49.588 in.^{3}

Actual I_x = 49.588 x 0.060 = 2.975 in.^{4}

S_e = 2.975/3.000 = 0.992 in.^{3}

M = f x S_e = 25 x 0.992

= 24.80 kip-in. < M_s = 29.62 kip-in.

Need to do another iteration and also to increase f.

b. After several iterations, assume a compression stressof f = 29.86 ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

$S = 1.28\sqrt{29500/29.86} = 40.23$

$S/3 = 13.41 < (w/t) = 21.95 < S = 40.23$

$I_a = 399(0.060)^4[(21.95/40.23)-0.33]^3$

$= 0.000052 \text{ in.}^4$

$I_s/I_a = 0.000130/0.000052 = 2.50$

$k = [4.82-5(0.342)](2.50)^{1/2}+0.43 = 5.347 > 3.540$

$k = 3.540$

$\lambda = (1.052/\sqrt{3.540})(21.95)\sqrt{29.86/29500} = 0.390 < 0.673$

$b_d = 1.317$ in. (i.e. compression flange fully effective)

Compression (upper) stiffener:

f conservatively taken as for top compression fiber

$\lambda = (1.052/\sqrt{0.43})(4.93)\sqrt{29.86/29500} = 0.252 < 0.673$

$d'_s = 0.296$ in.

Since $I_s/I_a = 2.50 > 1.0$, it follows that $D_s = d'_s = 0.296$ in. (i.e. compression stiffener fully effective).

Thus, the section is fully effective.

$y_{cg} = 6/2 = 3.000$ in. (from symmetry)

Full section properties are the same as were found in the first iteration. Thus, as before, top compression fiber may be used in computing S_e.

To check if web is fully effective:

$f_1 = [(3.000-0.154)/3.000]\text{x}29.86 = 28.33$ ksi(compression)

$f_2 = -28.33$ ksi(tension)

$\psi = -28.33/28.33 = -1.000$

$k = 24.000$

$\lambda = (1.052/\sqrt{24})(94.87)\sqrt{28.33/29500} = 0.631 < 0.673$

$b_e = w = 5.692$ in.

Hence, as in first iteration, $b_1 + b_2 = 2.846$ in. and thus the web is fully effective as assumed.

$I_x = 2.975 \text{ in.}^4$

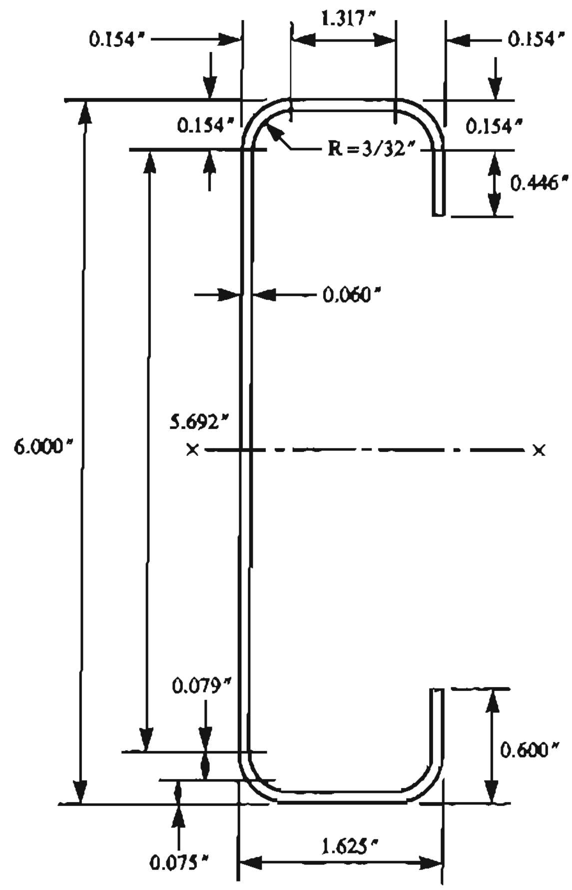
$S_e = 0.992 \text{ in.}^3$

$M = f \text{ x } S_e = 29.86 \text{ x } 0.992$

$= 29.62$ kip-in. $= M_s$ OK

Thus $I_x = 2.975 \text{ in.}^4$ using procedure I for deflection determination.

EXAMPLE NO. 3.
BRACED C-SECTION



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: 6 x 1.625 x 0.060 channel with stiffened flanges.
3. Compression flange braced against lateral buckling.
4. Dead load to live load ratio D/L = 1/5 and 1.2D + 1.6L governs the design.

Required:

1. Design flexural strength, $\phi_b M_n$, based on initiation of yielding.
2. Effective moment of inertia based on procedure I for deflection determination at the service moment.

Solution:

1. Calculation of the design flexural strength, $\phi_b M_n$:
   Properties of 90° corners:

$r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.

Length of arc, $u = 1.57r = 1.57 \times 0.124 = 0.195$ in. Distance of c.g. from center of radius, $c = 0.637r = 0.637 \times 0.124 = 0.079$ in.

Computation of I_x:

For the first approximation, assume a compression stress of $f = F_y = 50$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

$w = 1.317$ in.

$w/t = 1.317/0.060 = 21.95$

$S = 1.28\sqrt{E/f}$ (Eq. B4-1)

$= 1.28\sqrt{29500/50} = 31.09$

$S/3 = 10.36 < (w/t) = 21.95 < S = 31.09$

$I_a = 399t^4\{[(w/t)/S]-0.33\}^3$ (Eq. B4.2-6)

$= 399(0.060)^4[(21.95/31.09)-0.33]^3$

$= 0.000275$ in.^{4}

$D = 0.600$ in.

$d = 0.446$ in. $d/t = 0.446/0.060 = 7.43 < 14$ OK

(Section B4 of the Commentary)

$I_s = d^3t/12$ (Eq. B4-2)

$= (0.446)^3(0.060)/12 = 0.000444$ in.^{4}

$D/w = 0.600/1.317 = 0.456$, $0.25 < D/w = 0.456 < 0.80$ (Fig. B4-2)

$k = [4.82-5(D/w)](I_s/I_a)^n+0.43 \leq 5.25-5(D/w)$ (Eq. B4.2-9)

$n = 1/2$

$[4.82-5(0.456)](0.000444/0.000275)^{1/2}+0.43 = 3.657$

$5.25-5(0.456) = 2.970$

$k = 2.970$

Since $I_s > I_a$ and $D/w < 0.8$, the stiffener is not considered as a simple lip.

$w/t = 21.95 < 90$ OK (Section B1.1-(a)-(1))

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{2.970})(21.95)\sqrt{50/29500} = 0.55 < 0.673$

$b = w$ (Eq. B2.1-1)

$= 1.317$ in. (i.e. compression flange fully effective)

Compression (upper) stiffener:

$k = 0.43$ (unstiffened compression element)

$d/t = 7.43$

f conservatively taken equal to 50 ksi as in top compression fiber

$\lambda \quad = (1.052/\sqrt{0.43})(7.43)\sqrt{50/29500} = 0.489 < 0.673$

therefore,

$d'_s \quad = d = 0.446$ in.

$d_s \quad = d'_s(I_s/I_a) \leq d'_s$ (Eq. B4.2-11)

$= 0.446(0.000444/0.000275)$

$= 0.720$ in. > 0.446 in.

$d_s \quad = 0.446$ in. (i.e. compression stiffener is fully effective)

Thus, one concludes that the section is fully effective.

$y_{cg} = 6/2 = 3.000$ in. (from symmetry)

Full section properties about x axis:

Element	L (in.)	y Distance from Centerline of Section (in.)	Ly^2 ($in.^3$)	I'_1 About Own Axis ($in.^3$)
Web	5.692	—	—	15.368
Stiffeners	2 x 0.446 = 0.892	2.623	6.137	0.015
Corners	4 x 0.195 = 0.780	2.925	6.673	—
Flanges	2 x 1.317 - 2.634	2.970	23.234	—
Sum			36.044	15.383

Since section is singly symmetric about x-axis and fully effective, a compression stress of 50 ksi will govern as assumed. (At the bottom tension fibers a tensile stress of 50 ksi will develop simultaneously from symmetry).

To check if web is fully effective: (Section B2.3)

$f_1 \quad = [(3.000-0.154)/3.000]x50 = 47.43$ ksi(compression)

$f_2 \quad = -47.43$ ksi(tension)

$\psi \quad = f_2/f_1 = -47.43/47.43 = -1.000$

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

$= 4+2\,[1-(-1)]^3 + 2\,[1-(-1)]$

$k \quad = 24.000$

$h \quad = w = 5.692$ in., $h/t = w/t = 5.692/0.060 = 94.87$

$h/t \quad = 94.87 < 200$ OK (Section B1.2-(a))

$\lambda \quad = (1.052/\sqrt{24})(94.87)\sqrt{47.43/29500} = 0.817 > 0.673$

$\rho = [1-(0.22/\lambda)]/\lambda$ (Eq.B2.1-3)

$= [1-(0.22/0.817)]/0.817 = 0.894$

$b_e = \rho w$ (Eq. B2.1-2)

$= 0.894 \times 5.692 = 5.089$ in.

$b_2 = b_e/2$ (Eq. B2.3-2)

$= 5.089/2 = 2.545$ in.

$b_1 = b_e/(3-\psi)$ (Eq. B2.3-1)

$= 5.089/[3-(-1)] = 1.272$ in.

$b_1+b_2 = 1.272 + 2.545 = 3.817$ in.

Compression portion of the web = $y_{cg} - 0.154$

$= 3.000 - 0.154$

$= 2.846$ in.

Since $b_1+b_2 = 3.817$ in. > 2.846 in., b_1+b_2 shall be taken as 2.846 in.. This verifies the assumption that the web is fully effective.

$I'_x = Ly^2+I'_1$

$= 36.044 + 15.383$

$= 51.427$ in.^{3}

Actual $I_x = I'_x t$

$= 51.427 \times 0.060$

$= 3.086$ in.^{4}

$S_e = I_x/y_{cg}$

$= 3.086/3.000$

$S_e = 1.029$ in.^{3}

$M_n = S_e F_y$ (Eq. C3.1.1-1)

$= 1.029 \times 50$

$= 51.45$ kip-in.

$\phi_b = 0.95$

$\phi_b M_n = 0.95 \times 51.45 = 48.88$ kip-in.

2. Calculation of the effective moment of inertia based on procedure I for deflection determination at the service moment M_s:

$\phi_b M_n = 1.2M_{DL} + 1.6M_{LL}$

$= [1.2(M_{DL}/M_{LL})+1.6]M_{LL}$

$= [1.2(1/5)+1.6]M_{LL}$

$= 1.84M_{LL}$

$M_{LL} = \phi_b M_n/1.84 = 48.88/1.84 = 26.57$ kip-in.

$M_s = M_{DL} + M_{LL}$

$= (1/5+1)M_{LL}$

$= 1.2(26.57) = 31.88$ kip-in.

where

M_{DL} = Moment determined on the basis of nominal dead load

M_{LL} = Moment determined on the basis of nominal live load

The procedure is iterative: one assumes the actual compressive stress f under this service moment M_s. Knowing f, one proceeds as usual to obtain S_e. and checks to see if ($f \times S_e$) is equal to M_s as it should. If not, reiterate until one obtains the desired level of accuracy. (Section B2.1-(b)-(1))

a. For the first iteration, assume a compression stress of f = 30.98 ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

$S = 1.28\sqrt{29500/30.98} = 39.50$ (Eq B4-1)

$S/3 = 13.17 < w/t = 21.95 < S = 39.50$

$I_a = 399(0.060)^4\,[(21.95/39.50)\text{-}0.33]^3$

$= 0.000059$ in.^{4}

$I_s/I_a = 0.000444/0.000059 = 7.525$

$k = [4.82\text{-}5(0.456)](7.525)^{1/2}+0.43 = 7.398 > 2.970$ (Eq. B4.2-9)

$k = 2.970$

$\lambda = (1.052/\sqrt{2.970})(21.95)\sqrt{30.98/29500} = 0.434 < 0.673$ (Eq. B2.1-4)

$b_d = 1.317$ in. (i.e. compression flange fully effective)

Compression (upper) stiffener:

f conservatively taken equal to 30.98 ksi as in top compression fiber

$.\lambda = (1.052/\sqrt{0.43})(7.43)\sqrt{30.98/29500} = 0.386 < 0.673$ therefore, $d'_s = 0.446$ in.

Since $I_s/I_a = 7.525 > 1.0$, it follows that $d_s = d'_s = 0.446$ in. (i.e. compression stiffener fully effective).

Thus the section is fully effective.

$y_{cg} = 6/2 = 3.000$ in. (from symmetry)

And since the section is singly symmetric about x-axis, top compression fiber (and also bottom tension fiber) may be used in computing S_e.

To check if web is fully effective:

$f_1 = [(3.000\text{-}0.154)/3.000]\text{x}30.98 = 29.39$ ksi(compression)

$f_2 = -29.39$ ksi(tension)

$\psi = f_2/f_1 = -29.39/29.39 = -1.000$

$k = 24.0$

$\lambda = (1.052/\sqrt{24})(94.87)\sqrt{29.39/29500} = 0.643 < 0.673$

$b_e = w$ (Eq. B2.1-1)

$= 5.692$ in.

$b_2 = 5.692/2 = 2.846$ in.

b_1 = 5.692/ [3-(-1)] = 1.423 in.

$b_1 + b_2$ = 4.269 in. > compression portion of the web = 2.846 in. thus b_1+b_2 shall be taken as 2.846 in.. This verifies the assumption that the web is fully effective.

Full section properties are the same as were found in determination of $\phi_b M_n$ since the section is fully effective.

I_x = 3.086 in.^{4}

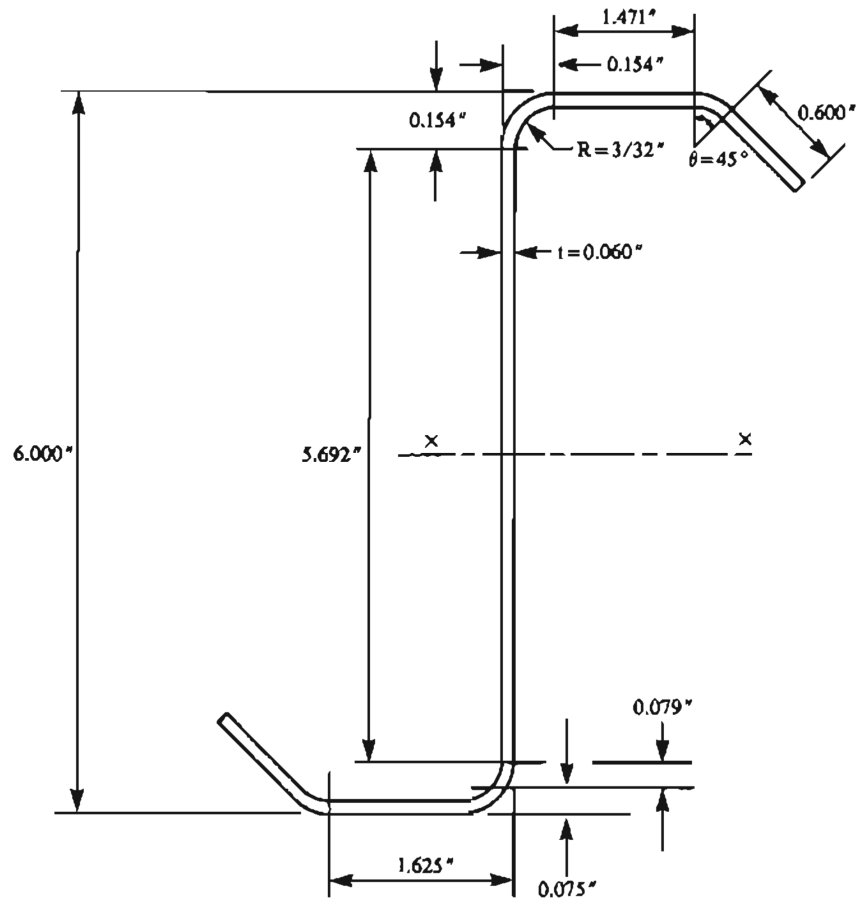
S_e = 1.029 in.^{3}

M = f x S_e = 30.98 x 1.029

= 31.88 kip-in. = M_s OK

Thus, I_x = 3.086 in.^{4} using procedure I for deflection determination.

EXAMPLE NO. 4.
BRACED Z-SECTION



Given:

1. Steel: F_y = 50 ksi.
2. Section: 6 x 1.625 x 0.060 Z-section with stiffened flanges.
3. Compression flange braced against lateral buckling.
4. Dead load to live load ratio D/L = 1/5 and 1.2D + 1.6L governs the design.

Required:

1. Design flexural strength, $\phi_b M_n$, based on initiation of yielding.
2. Effective moment of inertia based on procedure I for deflection determination at the service moment.

Solution:

1. Calculation of the design flexural strength, $\phi_b M_n$:

Properties of 90° corners:

$r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.

Length of arc, $u = 1.57r = 1.57 \times 0.124 = 0.195$ in.

Distance of c.g. from center of radius,

$c = 0.637r = 0.637 \times 0.124 = 0.079$ in.

Properties of 135° corners:

$r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.

Length of arc, $u = (45°/180°)(3.14)r = 0.785r = 0.785 \times 0.124$

$= 0.097$ in.

Distance of c.g. from center of radius,

$c_1 = r\sin\theta/\theta = [(0.124 \times \sin 45°)/0.785] = 0.112$ in.

Computation of I_x:

For the first approximation, assume a compression stress of $f = F_y = 50$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

w	$= 1.471$ in.	
w/t	$= 1.471/0.060 = 24.52$	
S	$= 1.28\sqrt{E/f}$	(Eq. B4-1)
	$= 1.28\sqrt{29500/50} = 31.09$	
S/3	$= 10.36 < w/t = 24.52 < S = 31.09$	
I_a	$= 399t^4\{[(w/t)/S] - 0.33\}^3$	(Eq. B4.2-6)
	$= 399(0.060)^4[(24.52/31.09)-0.33]^3$	
	$= 0.000499$ in.^{4}	
d	$= 0.600$ in., $d/t = 0.600/0.060 = 10 < 14$ OK	

(Section B4 of the Commentary)

D	$= d+0.154\tan(\theta/2) = 0.600+0.154\tan(45°/2) = 0.664$ in.	
I_s	$= d^3 t\sin^2\theta/12$	(Eq. B4-2)
	$= (0.600)3(0.060)\sin^2(45o)/12 = 0.000540$ in.^{4}	
I_s/I_a	$= 0.000540/0.000499 = 1.082$	
D/w	$= 0.664/1.471 = 0.451$, $0.25 < D/w = 0.451 < 0.80$	
k	$= [4.82-5(D/w)](I_s/I_a)^n+0.43 \leq 5.25-5(D/w)$	(Eq. B4.2-9)
n	$= 1/2$	
	$[4.82-5(0.451)](1.082)^{1/2}+0.43 = 3.098$	
	$5.25-5(0.451) = 2.995$	
k	$= 2.995$	

Since $I_s > I_a$ and $D/w < 0.8$, the stiffener is not considered as a simple lip.

$w/t = 24.52 < 90$ OK (Section B1.1-(a)-(1))

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{2.995})(24.52)\sqrt{50/29500} = 0.614 < 0.673$

$b = w$ (Eq. B2.1-1)

$= 1.471$ in. (i.e. compression flange fully effective)

Compression (upper) stiffener:

$k = 0.43$ (unstiffened compression element)

$d/t = 10.00$

f conservatively taken equal to 50 ksi as in top compression fiber

$\lambda = (1.052/\sqrt{0.43})(10.00)\sqrt{50/29500} = 0.660 < 0.673$

therefore,

$d'_s = d = 0.600$ in.

$d_s = d'_s(I_s/I_a) \leq d'_s$ (Eq. B4.2-11)

$= 0.600(1.082)$

$= 0.649$ in. > 0.600 in.

$d_s = 0.600$ in. (i.e. compression stiffener is fully effective)

Thus, one concludes that the section is fully effective.

$y_{cg} = 6/2 = 3.000$ in. (from symmetry)

Full section properties about x axis:

Element	L (in.)	y Distance from Centerline of Section (in.)	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Web	5.692	—	—	15.368
Stiffeners	2 x 0.600 = 1.200	2.743	9.029	0.018
90° corners	2 x 0.195 = 0.390	2.925	3.337	—
135° corners	2 x 0.097 = 0.194	2.958	1.697	—
Flanges	2 x 1.471 = 2.942	2.970	25.951	—
Sum			40.014	15.386

Since section is singly symmetric about x-axis and fully effective, a compression stress of 50 ksi will govern as assumed. (At the bottom tension fibers a tensile stress of 50 ksi will develop simultaneously from geometry).

To check if web is fully effective: (Section B2.3)

f_1 = [(3.000-0.154)/3.000]x50 = 47.43 ksi(compression)

f_2 = -47.43 ksi(tension)

ψ = f_2/f_1 = -47.43/47.43 = -1.000

k = $4+2(1-\psi)^3+2(1-\psi)$

= $4+2\,[1-(-1)]^3+2\,[1-(-1)]$

= 24.000

h = w = 5.692 in., h/t = w/t = 5.692/0.060 = 94.87

h/t = 94.87 < 200 OK (Section B1.2-(a))

λ = $(1.052/\sqrt{24})(94.87)\sqrt{47.43/29500}$ = 0.817 > 0.673

ρ = $[1-(0.22/\lambda)]/\lambda$ (Eq. B2.1-3)

= [1-(0.22/0.817)]/0.817 = 0.894

b_e = ρw (Eq. B2.1-2)

= 0.894 x 5.692 = 5.089 in.

b_2 = $b_e/2$ (Eq. B2.3-2)

= 5.089/2 = 2.545 in.

b_1 = $b_e/(3-\psi)$ (Eq. B2.3-1)

= 5.089/ [3-(-1)] = 1.272 in.

b_1+b_2 = 1.272 + 2.545 = 3.817 in.

Compression portion of the web = y_{cg} - 0.154

= 3.000 - 0.154= 2.846 in.

Since b_1+b_2 = 3.817 in. > 2.846 in., b_1+b_2 shall be taken as 2.846 in. This verifies the assumption that the web is fully effective.

I'_x = $Ly^2+I'_1$

= 40.014 + 15.386

= 55.400 in.^{3}

Actual I_x = $I'_x t$

= 55.400 x 0.060

= 3.324 in.^{4}

S_e = I_x/y_{cg}

= 3.324/3.0 = 1.108 in.^{3}

M_n = S_eF_y (Eq. C3.1.1-1)

= 1.108 x 50 = 55.40 kip-in.

ϕ_b = 0.95

ϕ_bM_n = 0.95 x 55.400 = 52.63 kip-in.

2. Calculation of the effective moment of inertia based on procedure I for deflection determination at the service moment M_s:

$\phi_b M_n = 1.2M_{DL} + 1.6M_{LL}$

$= [1.2(M_{DL}/M_{LL})+1.6]M_{LL}$

$= [1.2(1/5)+1.6]\ M_{LL}$

$= 1.84M_{LL}$

$M_{LL} = \phi_b M_n/1.84 = 52.63/1.84 = 28.603$ kip-in.

$M_s = M_{DL} + M_{LL}$

$= (1/5+1)M_{LL}$

$= 1.2(28.603) = 34.324$ kip-in.

where

M_{DL}= Moment determined on the basis of nominal dead load

M_{LL}= Moment determined on the basis of nominal live load

The procedure is iterative: one assumes the actual compressive stress f under this service moment M_s. Knowing f, one proceeds as usual to obtain S_e and checks to see if (f x S_e) is equal to M_s as it should. If not, reiterate until one obtains the desired level of accuracy. (Section B2.1-(b)-(1))

a. For the first iteration, assume a compression stress of f = 30.99 ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

$S = 1.28\sqrt{29500/30.99} = 39.49$

$S/3 = 13.16 < w/t = 24.52 < S = 39.49$

$I_a = 399(0.060)^4\,[(24.52/39.49)-0.33]^3$

$= 0.000127$ in.^{4}

$I_s/I_a = 0.000540/0.000127 = 4.252$

$k = [4.82-5(0.443)](4.252)^{1/2}+0.43 = 5.802 > 3.035$

$k = 3.035$

$\lambda = (1.052/\sqrt{3.035})(24.52)\sqrt{30.99/29500} = 0.480 < 0.673$

$b_d = 1.471$ in. (i.e. compression flange fully effective)

Compression (upper) stiffener:

f conservatively taken equal to 30.99 ksi as in top compression fiber

$\lambda = (1.052/\sqrt{0.43})(10.00)\sqrt{30.99/29500} = 0.520 < 0.673$

therefore, $d'_s = 0.600$ in.

Since $I_s/I_a = 4.252\ \ 1.0$, it follows that $D_s = d'_s = 0.600$ in. (i.e. compression stiffener fully effective).

Thus the section is fully effective.

$y_{cg} = 6/2 = 3.000$ in. (from symmetry)

And since the section is singly symmetric about x-axis, top compression fiber (and also bottom tension fiber) may be used in computing S_e.

To check if web is fully effective:

f_1 = [(3.000-0.154)/3.000]x30.99 = 29.40 ksi(compression)

f_2 = -29.40 ksi(tension)

ψ = f_2/f_1 = -29.40/29.40 = -1.000

k = 24.000

λ = $(1.052/\sqrt{24})(94.87)\sqrt{29.40/29500} = 0.643 < 0.673$

b_e = w (Eq. B2.1-1)

= 5.692 in.

b_2 = 5.692/2 = 2.846 in.

b_1 = 5.692/ [3-(-1)] = 1.423 in.

$b_1 + b_2$ = 4.269 in. > compression portion of the web = 2.846 in. thus $b_1 + b_2$ shall be taken as 2.846 in.. This verifies the assumption that the web is fully effective.

Full section properties are the same as were found in determination of $\phi_b M_n$ since the section is fully effective.

I_x = 3.324 in.^{4}

S_e = 1.108 in.^{3}

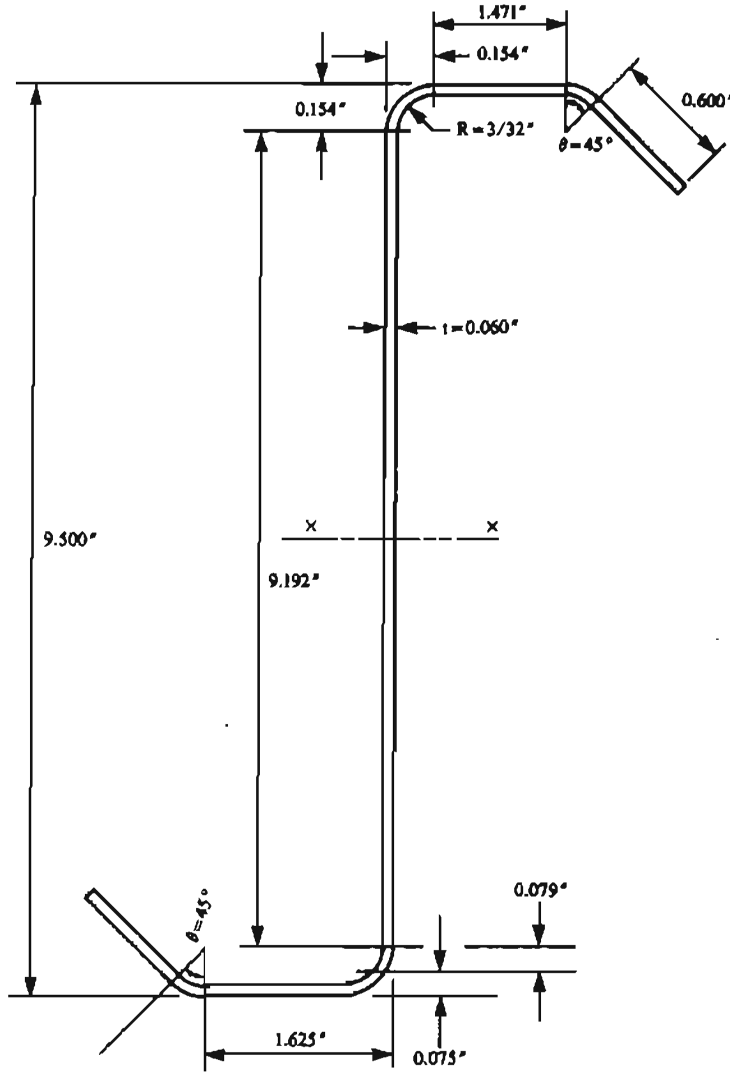
M = f x S_e = 30.99 x 1.108

= 34.34 kip-in. = M_s OK

Thus, I_x = 3.324 in.^{4} using procedure I for deflection determination.

EXAMPLE NO. 4A.

DEEP-Z SECTION WITH STIFFENED FLANGE



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: 9.5 x 1.625 x 0.060 Z-section with stiffened flanges.
3. Compression flange braced against lateral buckling.
4. Dead load to live load ratio D/L = 1/5 and 1.2D + 1.6L governs the design.

Required:

1. Design flexural strength, $\phi_b M_n$, based on initiation of yielding.
2. Effective moment of inertia based on procedure I for deflection determination at the service moment.

Solution:

1. Calculation of the design flexural strength, $\phi_b M_n$:
   Properties of 90° corners:

$r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.

Length of arc, $u = 1.57r = 1.57 \times 0.124 = 0.195$ in.

Distance of c.g. from center of radius,

$c = 0.637r = 0.637 \times 0.124 = 0.079$ in.

Properties of 135° corners:

$r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.

Length of arc, $u = (45^\circ/180^\circ)(3.14)r = 0.785r = 0.785 \times 0.124 = 0.097$ in.

Distance of c.g. from center of radius,

$c_1 = r\sin\theta/\theta = [(0.124 \times \sin 45^\circ)/0.785] = 0.112$ in.

Computation of I_x:

For the first approximation, assume a compression stress of $f = F_y = 50$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

$w = 1.471$ in.

$w/t = 1.471/0.060 = 24.52$

$S = 1.28\sqrt{E/f}$ (Eq. B4-1)

$= 1.28\sqrt{29500/50} = 31.09$

$S/3 = 10.36 < w/t = 24.52 < S = 31.09$

$I_a = 399t^4\{[(w/t)/S] - 0.33\}^3$ (Eq. B4. 2-6)

$= 399(0.060)^4[(24.52/31.09) - 0.33]^3$

$= 0.000499$ in.^{4}

$d = 0.600$ in., $d/t = 0.600/0.060 = 10 < 14$ OK

(Section B4 of the Commentary)

$D = d + 0.154\tan(\theta/2) = 0.600 + 0.154\tan(45^\circ/2) = 0.664$ in.

$I_s = d^3 t \sin^2\theta/12$ (Eq. B4-2)

$= (0.600)3(0.060)\sin2(45^\circ)/12 = 0.000540$ in.^{4}

$I_s/I_a = 0.000540/0.000499 = 1.082$

$D/w = 0.664/1.471 = 0.451$, $0.25 < D/w = 0.451 < 0.80$

$k = [4.82 - 5(D/w)](I_s/I_a)^n + 0.43 \le 5.25 - 5(D/w)$ (Eq. B4.2-9)

$n = 1/2$

$[4.82 - 5(0.451)](1.082)^{1/2} + 0.43 = 3.098$

$5.25 - 5(0.451) = 2.995$

$k = 2.995$

Since $I_s > I_a$ and $D/w < 0.8$, the stiffener is not considered as a simple lip.

$w/t = 24.52 < 90$ OK (Section B1.1-(a)-(1))

λ $= (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{2.995})(24.52)\sqrt{50/29500} = 0.614 < 0.673$

b $= w$ (Eq. B2.1-1)

$= 1.471$ in. (i.e. compression flange fully effective)

Compression (upper) stiffener:

k $= 0.43$ (unstiffened compression element)

d/t $= 10.00$

f conservatively taken equal to 50 ksi as in top compression fiber

λ $= (1.052/\sqrt{0.43})(10.00)\sqrt{50/29500} = 0.660 < 0.673$

therefore,

d'_s $= d = 0.600$ in.

d_s $= d'_s(I_s/I_a) \leq d'_s$ (Eq. B4.2-11)}

since $I_s/I_a = 1.082 > 1.000$

d_s $= d'_s = 0.600$ in. (i.e. compression stiffener is fully effective)

Thus, one concludes that the section is fully effective.

y_{cg} $= 9.5/2 = 4.750$ in. (from symmetry)

It follows that a compression stress of 50 ksi will govern as assumed.

To check if web is fully effective (Section B2.3):

f_1 $= [(4.750-0.154)/4.750]x50 = 48.38$ ksi(compression)

f_2 $= -48.38$ ksi(tension)

ψ $= f_2/f_1 = -48.38/48.38 = -1.000$

k $= 4+2(1-\psi)^3+2(1-\psi)$

$= 4+2\,[1-(-1)]^3 + 2\,[1-(-1)]$

$= 24.000$

h $= w = 9.192$ in., $h/t = w/t = 9.192/0.060 = 153.20$

h/t $= 153.20 < 200$ OK (Section B1.2-(a))

λ $= (1.052/\sqrt{24})(153.20)\sqrt{48.38/29500} = 1.332 > 0.673$

ρ $= [1-(0.22/\lambda)]/\lambda;$ (Eq. B2.1-3)

$= [1-(0.22/1.332)]/1.332 = 0.627$

b_e $= \rho w$ (Eq. B2.1-2)

$= 0.627 \times 9.192 = 5.763$ in.

b_2 $= b_e/2$ (Eq. B2.3-2)

$= 5.763/2 = 2.882$ in.

b_1 $= b_e/(3-\psi)$ (Eq. B2.3-1)

$= 5.763/[3-(-1)] = 1.441$ in.

$b_1 + b_2 = 1.441 + 2.882 = 4.323$ in.

Compression portion of the web = $y_{cg} - 0.154$
= $4.750 - 0.154$
= 4.596 in.

Since $b_1 + b_2 = 4.323$ in. < 4.596 in., it follows that the web is not fully effective. Hence $y_{cg} \neq 4.750$; as assumed.

The procedure to determine the location of the neutral axis (N.A.) based on partially effective web is iterative. We start with $y_{cg} = 4.750$ and from Figure B2.3-1 scale b_1, b_2 already computed with respect to $y_{cg} = 4.750$ in.. Then we proceed to compute a new N.A. and hence $b_1 + b_2$. If $b_1 + b_2$ same as before, the solution stabilizes and the location of N.A. is calculated according to this (b_1+b_2). If $(b_1 + b_2)$ differ than before, one reiterates in the same before mentioned manner until $b_1 + b_2$ stabilizes.

Thus, for the first iteration, the web is divided into three segments:

$b_1 = 1.441$ in., ineffective portion of web, and $b_2(=2.882)+4.750-0.154 = 7.478$ in.. Thus the ineffective portion of web = 9.192-1.441-7.478 = 0.273 in.

The compression flange and stiffener remain fully effective since nothing is altered in their calculations.

Effective section properties about x-axis:

Element	L (in.)	y Distance from Top Fiber (in.)		Ly (in.^{2})
b_1	1.441	0.154+(1.441/2)	= 0.875	1.261
$b_2+(9.5-y_{cg})-0.154$	7.478	9.5-0.154-(7.478/2)	= 5.607	41.929
Compression flange	1.471		0.030	0.044
Compresson stiffener	0.600	0.064-0.124cos45° +(0.600/2)cos45°	= 0.257	0.154
Top 90° corner	0.195		0.075	0.015
Top 135° corner	0.097	0.154-0.112	= 0.042	0.004
Bottom 135° corner	0.097	9.5-(0.154-0.112)	= 9.458	0.917
Bottom 90° corner	0.195	9.5-0.075	= 9.425	1.838
Bottom stiffener	0.600	9.5-0.257	= 9.243	5.546
Tension flange	1.471	9.5-(0.060/2)	= 9.470	13.930
Sum	13.645			65.638

y_{cg} = Ly/L = 65.638/13.645= 4.810 in. (measured from top compression fiber)

f_1 = [(4.810-0.154)/4.810](50) = 48.40 ksi(compression)

f_2 = -[(9.5-4.810-0.154)/4.810](50) = -47.15 ksi(tension)

ψ = -47.15/48.40 = -0.974

$k = 4+2\,[1-(-0.974)]^3 + 2\,[1-(-0.974)] = 23.332$

λ = $(1.052/\sqrt{23.332})(153.20)\sqrt{48.40/29500} = 1.351 > 0.673$

ρ = $[1-(0.22/1.351)]/1.351 = 0.620$

b_e = $0.620 \times 9.192 = 5.699$ in.

b_2 = $5.699/2 = 2.850$ in.

b_1 = $5.699/[3-(-0.974)] = 1.434$ in.

$b_1 + b_2$ = 4.284 in. $\neq 4.323$ in. therefore need to reiterate

For the second iteration:

$b_1 = 1.434$ in.

$b_2+(9.5-y_{cg})-0.154 = 2.850+9.5-4.810-0.154 = 7.386$ in.

ineffective portion of web = $9.192-1.434-7.386 = 0.372$ in.

Effective section properties about x-axis:

Element	L (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})
b_1	1.434	0.871	1.249
$B_2+(9.5-y_{cg})-0.154$	7.386	5.653	41.753
Compression flange	1.471	0.030	0.044
Compression stiffener	0.600	0.257	0.154
Top 90° corner	0.195	0.075	0.015
Top 135° corner	0.097	0.042	0.004
Bottom 135° corner	0.097	9.458	0.917
Bottom 90° corner	0.195	9.425	1.838
Bottom stiffener	0.600	9.243	5.546
Tension flange	1.471	9.470	13.930
Sum	13.546		65.450

y_{cg} = $65.450/13.546 = 4.832$ in. (measured from top compression fiber)

f_1 = $[(4.832-0.154)/4.832](50) = 48.41$ ksi

f_2 = $-[(9.5-4.832-0.154)/4.832](50) = -46.71$ ksi

ψ = $-46.71/48.41 = -0.965$

k = $4+2[1-(-0.965)]^3+2[1-(-0.965)] = 23.105$

λ = $(1.052/\sqrt{23.105})(153.20)\sqrt{48.41/29500} = 1.358 > 0.673$

ρ = $[1-(0.22/1.358)]/1.358 = 0.617$

b_e = $0.617 \times 9.192 = 5.671$ in.

b_2 = $5.671/2 = 2.836$ in.

b_1 = $5.671/[3-(-0.965)] = 1.430$ in.

b_1+b_2 = 4.266 in. $\neq$ 4.284 in. therefore need to reiterate

For the third iteration:

b_1 =1.430 in.

$b_2+(9.5-y_{cg})-0.154 = 2.836+9.5-4.832-0.154 = 7.350$ in.

ineffective portion of web = 9.192-1.430-7.350 = 0.412 in.

Effective section properties about x-axis:

L = 13.506 in.

Ly = 65.373 in.^{2}

y_{cg} = 65.373/13.506 = 4.840 in.

f_1 = [(4.840-0.154)/4.840](50) = 48.41 ksi

f_2 = -[(9.5-4.840-0.154)/4.840](50) = -46.55 ksi

ψ = -46.55/48.41 = -0.962

k = $4+2\,[1-(-0.962)]^3 +2\,[1-(-0.962)] = 23.029$

λ = $(1.052/\sqrt{23.029})\,(153.20)\,\sqrt{48.41/29500}$ = 1.360 > 0.673

ρ = [1-(0.22/1.360)]/1.360 = 0.616

b_e = 0.616 x 9.192 = 5.662 in.

b_2 = 5.662/2 = 2.831 in.

b_1 = 5.662/ [3-(-0.962)] = 1.429 in .

b_1+b_2 = 4.260 in. $\neq$ 4.266 in. therefore need to reiterate

For the fourth iteration:

b_1 =1.429 in.

$b_2 + (9.5-y_{cg})-0.154 = 2.831+9.5-4.840-0.154 = 7.337$ in.

in effective portion of web = 9.192-1.429-7.337 = 0.426 in.

Effective section properties about x-axis:

L = 13.492 in.

Ly = 65.345 in.^{2}

y_{cg} = 65.345/13.492 = 4.843 in.

f_1 = [(4.843-0.154)/4.843](50) = 48.41 ksi

f_2 = - [(9.5-4.843-0.154)/4.843](50) = -46.49 ksi

ψ = -46.49/48.41 = -0.960

k = $4+2\,[1-(-0.960)]^3 +2\,[1-(-0.960)]$ = 22.979

λ = $(1.052/\sqrt{22.979})(153.20)\,\sqrt{48.41/29500}$ = 1.362 > 0.673

= [1-(0.22/1.362)]/1.362 = 0.616

b_e = 0.616 x 9.192 = 5.662 in.

b_2 = 5.662/2 = 2.831 in.

b_1 = 5.662/ [3-(-0.960)] = 1.430 in.

b_1+b_2 = 4.261 in. close enough to 4.260 in. thus the solution stabilizes.

Hence we now compute the location of N.A. and moment of inertia using b_1 = 1 .430 in. and b_2 = 2.831 in.

Effective section properties about x-axis:

Element	L (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
b_1	1.430	0.869	1.243	1.080	0.244
$B_2+(9.5-y_{cg})-0.154$	7.334	5.679	41.650	236.578	32.860
Compression flange	1.471	0.030	0.044	0.001	—
Compression stiffener	0.600	0.257	0.154	0.040	0.009
Top 90° corner	0.195	0.075	0.015	0.001	—
Top 135° corner	0.097	0.042	0.004	—	—
Bottom 135° corner	0.097	9.458	0.917	8.673	—
Bottom 90° corner	0.195	9.425	1.838	17.322	—
Bottom stiffener	0.600	9.243	5.546	51.262	0.009
Tension flange	1.471	9.470	13.930	131.917	—
Sum	13.490		65.341	446.875	33.122

Distance of x-axis from top fiber is y_{cg}= 65.341/13.490 = 4.844 in.

Since distance of top compression fiber from neutral axis is greater than one half the beam depth (= 4.750 in.), a compression stress of 50 ksi will govern as assumed.

I'_x $= Ly^2+I'_1-Ly^2_{cg}$

$= 446.875 + 33.122 - 13.490(4.844)^2$

$= 163.487$ in.^{3}

Actual I_x $= I'_x t$

$= 163.487 \times 0.060$

$= 9.809$ in.$_4$

S_e $= I_x/y_{cg}$

$= 9.809/4.844$

$= 2.025$ in.^{3}

M_n $= S_e F_y$ (Eq. C3.1.1-1)

$= 2.025 \times 50$

$= 101.25$ kip-in.

ϕ_b $= 0.95$

$\phi_b M_n = 0.95 x 101.25 = 96.19$ kip-in.

2. Calculation of the effective moment of inertia based on procedure I for deflection determination at the service moment M_s:

$$\begin{aligned}
\phi_b M_n &= 1.2M_{DL} + 1.6M_{LL} \\
&= [1.2(M_{DL}/M_{LL})+1.6]\, M_{LL} \\
&= [1.2(1/5)+1.6]\, M_{LL} \\
&= 1.84M_{LL} \\
M_{LL} &= \phi_b M_n/1.84 = 96.19/1.84 = 52.28 \text{ kip-in.} \\
M_s &= M_{DL} + M_{LL} \\
&= (1/5+1)M_{LL} \\
&= 1.2(52.28) = 62.73 \text{ kip-in.}
\end{aligned}$$

where

M_{DL}= Moment determined on the basis of nominal dead load

M_{LL}= Moment determined on the basis of nominal live load

The procedure is iterative: one assumes the actual compressive stress f under this service moment M_s. Knowing f, one proceeds as usual to obtain S_e and checks to see if (f x S_e) is equal to M_s as it should. If not, reiterate until one obtains the desired level of accuracy. (Section B2.1-(b)-(1))

a. For the first iteration, assume a compression stress of f = 30 ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

$$\begin{aligned}
S &= 1.28\sqrt{29500/30} = 40.14 \\
S/3 &= 13.38 < w/t = 24.52 < S = 40.14 \\
I_a &= 399(0.060)^4[(24.52/40.14)-0.33]^3 \\
&= 0.000115 \text{ in.}^4 \\
I_s/I_a &= 0.000540/0.000115 = 4.696 \\
k &= [4.82-5(0.451)](4.696)^{1/2}+0.43 = 5.988 > 2.995 \\
k &= 2.995 \\
\lambda &= (1.052/\sqrt{2.995})(24.52)\sqrt{30/29500} = 0.475 < 0.673 \\
b_d &= 1.471 \text{ in. (i.e. compression flange fully effective)}
\end{aligned}$$

Compression (upper) stiffener:

f conservatively taken equal to 30 ksi as in the top compression fiber.

$$\lambda = (1.052/\sqrt{0.43})(10.00)\sqrt{30/29500} = 0.512 < 0.673$$

therefore, $d'_s = 0.600$ in.

Since $I_s/I_a = 4.696 > 1.0$, it follows that $D_s = d'_s = 0.600$ in.
(i.e. compression stiffener fully effective).

Thus section is fully effective (since web was assumed fully effective).

$y_{cg} = 9.5/2 = 4.750$ in. (from symmetry)

To check if web is fully effective:

$f_1 =$ [(4.750-0.154)/4.750] ;(30) = 29.03 ksi

$f_2 =$ -29.03 ksi

$\psi =$ -29.03/29.03 =1.000

$k =$ 24.000

$\lambda =$ $(1.052/\sqrt{24})(153.20)\sqrt{29.03/29500} =$ 1.032 >0.673

$r =$ [1-(0.22/1.032)]/1.032 = 0.762

$b_e =$ 0.762 x 9.192 = 7.004 in.

$b_2 =$ 7.004/2 = 3.502 in.

$b_1 =$ 7.004/ [3-(-1)] ; = 1.751 in.

compression portion of the web = y_{cg} - 0.154

= 4.750 - 0.154 = 4.596 in.

$b_1 + b_2$ = 5.253 in. > 4.596 in.

Thus $b_1 + b_2$ shall be taken as 4.596 in. This verifies the assumption that the web is fully effective.

Full section properties about x-axis:

Element	L (in.)	y Distance from Centerline of Section (in.)	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Web	9.192	—	—	64.722
Stiffeners	2 x 0.600 = 1.200	4.493	24.224	0.018
90° corners	2 x 0.195 = 0.390	4.675	8.524	—
135° corners	2 x 0.097 = 0.194	4.708	4.300	—
Flanges	2 x 1.471 = 2.942	4.720	65.543	—
Sum			102.591	64.740

$I'_x =$ $Ly^2 + I'_1$

= 102.591 + 64.740 = 167.331 in.^{3}

$I_x =$ 167.331(0.060) = 10.040 in.$_4$

$S_e =$ I_x/y_{cg} = 10.040/4.750 = 2.114 in.^{3}

$M =$ f x S_e = 30 x 2.114

= 63.42 kip-in. not equal to M_A = 60.63 kip-in. thus need to reiterate.

However, one sees that we need to assume a smaller stress than 30 ksi and since the section was fully effective for f = 30 ksi, it will be fully effective for f < 30 ksi.

Thus S_e = 2.114 in.^{3}

Therefore, the correct actual f at $M_A = M_A/S_e = 60.63/2.114 = 28.68$ ksi. And $I_x = 10.026$ in.^{4} using procedure I for deflection determination.

Remark:

It was clearly seen that in the calculation of $\phi_b M_n$ the assumption of the web being fully effective was not true. However, it would be interesting to see the percentage of error if one neglected the partial effectiveness of the web and proceeded with the assumption of a fully effective web.

To demonstrate: neglect in the first approximation in the calculation of $\phi_b M_n$ the partial effectiveness of the web

Thus the whole section is fully effective. Full section properties about x-axis (from part 2):

I_x = 10.040 in.^{4}

S_e = 2.114 in.^{3}

$\phi_b M_n$ = 0.95(2.114x50) = 100.42 kip-in.

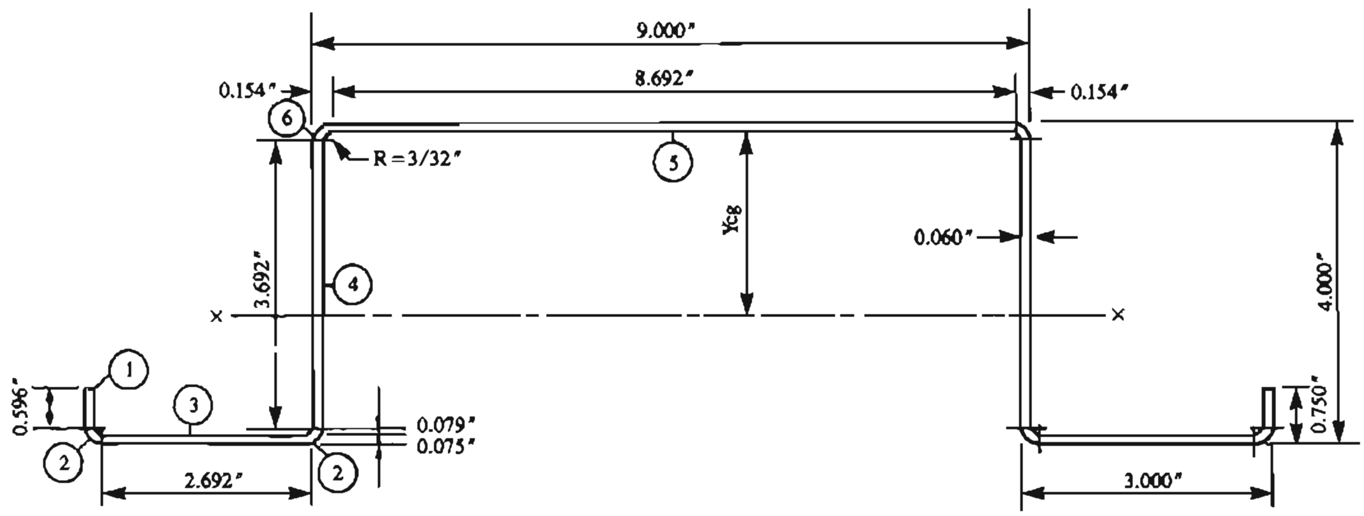
% error = [(100.42-96.00)/96.00]x100% = 4.60%

Since the percentage of error is small, one could rationalize that in practical cases to get a first-hand quick answer one could assume the web being fully effective.

EXAMPLE NO. 5

HAT SECTION

Complete Flexural Design,
Stiffened Compression Flange



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: As shown in sketch.
3. Span: L = 8 ft., with simple supports, no overhang, and 6-in. support bearing lengths.
4. Loading: Live = 300 lb/ft.; Dead = 20 lb/ft.

Required:

Check adequacy of section for:

1. Bending moment
2. Shear
3. Web Crippling
4. Deflection

Solution:

1. Properties of 90° corners:

Radius to centerline,

$r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.

Length of arc, $u = 1.57r = 1.57 \times 0.124 = 0.195$ in.

Distance of c.g. from center of radius,

$c = 0.637r = 0.637 \times 0.124 = 0.079$ in.

I' of corner about its own centroidal axis is negligible

2. Nominal Section Strength, M_n (Section C3.1.1)

a. Procedure I - Based on Initiation of Yielding
Computation of I_x, first approximation:

* Assume a compressive stress of $f = F_y = 50$ ksi in the top fibers of the section.

* Also assume web is fully effective.

Element 4:

$h/t = 3.692/0.060 = 61.53 < (h/t)_{max} = 200$ OK (Section B1.2-(a))

Assumed fully effective

Element 5:

$w/t = 8.692/0.060 = 144.9 < 500$ OK (Section B1.1 -(a)-(2))

$k = 4$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{4})(144.9)\sqrt{50/29500} = 3.138 > 0.673$

$\rho = [1-(0.22/\lambda)]/\lambda$ (Eq. B2.1-3)

$= [1-(0.22/3.138)]/3.138 = 0.296$

$b = \rho w$ (Eq. B2.1-2)

$= 0.296 \times 8.692$

$= 2.573$ in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in^3)	I'_1 About Own Axis (in^3)
1	2 x 0.596 = 1.192	3.548	4.229	15.005	0.035
2	4 x 0.195 = 0.780	3.925	3.062	12.016	—
3	2 x 2.692 = 5.384	3.970	21.375	84.857	—
4	2 x 3.692 = 7.384	2.000	14.768	29.536	8.388
5	2.573	0.030	0.077	0.002	—
6	2 x 0.195 = 0.390	0.075	0.029	0.002	—
Sum	17.703		43.540	141.418	8.423

Distance of neutral axis from top fiber, $y_{cg} = Ly/L = 43.540/17.703 = 2.460$ in.

Since the distance of the top compression fiber from the neutral axis is greater than one half the beam depth, a compressive stress of F_y will govern as assumed.

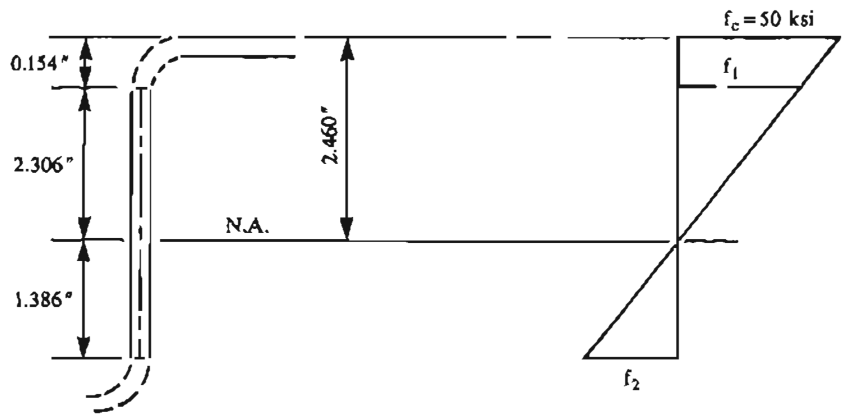
$I'_x = Ly^2 + I'_1 - Ly^2_{cg}$

$= 141.418 + 8.423 - 17.703(2.460)^2$

$= 42.71 \text{ in.}^3$

Actual $I_x = tI'_x = (0.060)(42.71) = 2.56 \text{ in.}^4$

Check Web



$f_1 = (2.306/2.460)(50) = 46.87$ ksi(compression)

$f_2 = -(1.386/2.460)(50) = -28.17$ ksi(tension)

$\psi = f_2/f_1 = -28.17/46.87 = -0.601$

$k = 4+2(1-\psi)^3+2(1-\psi)$ (Eq. B2.3-4)

$= 4+2[1-(-0.601)]^3+2[1-(-0.601)]$

$= 15.41$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$, $f = f_1$ (Eq. B2.1-4)

$= (1.052/\sqrt{15.41})(61.53)\sqrt{46.87/29500} = 0.657 < 0.673$

$b = w$ (Eq. B2.1-1)

$b_e = 3.692$ in.

$b_2 = b_e/2$ (Eq. B2.3-2)

$= 3.692/2 = 1.846$ in.

$b_1 = b_e/(3-\psi)$ (Eq. B2.3-1)

$= 3.692/[3-(-0.601)] = 1.025$ in.

$b_1+b_2 = 1.025 + 1.846 = 2.871 \text{ in.} > 2.306$ in. (compression portion of web)

Therefore, web is fully effective.

$S_e = I_x/y_{cg}$

$= 2.56/2.46 = 1.04 \text{ in.}^3$

$M_n = S_eF_y$ (Eq. C3.1.1-1)

$= (1.04)(50)$

$= 52.0$ kip-in.

b. Procedure II - Based on Inelastic Reserve Capacity

$\lambda_1 = (1.11/\sqrt{f_y/E})$ (Eq. C3.1.1-2)

$= (1.11/\sqrt{50/29500}) = 26.96$

$\lambda_2 = (1.28/\sqrt{F_y/E})$ (Eq. C3.1.1-3)

$= (1.28/\sqrt{50/29500}) = 31.09$

$w/t = 8.692/0.06 = 144.9$

For $w/t > \lambda_2$, $C_y = 1$

Maximum compressive strain $= C_ye_y = e_y$

Therefore, the nominal moment, M_n is the same as the M_n determined by procedure I because the compression flange will yield first.

3. Design Flexural Strength, ϕ_bM_n (Section C3.1)

$\phi_b = 0.95$, $\phi_bM_n = 0.95 \times 52.0 = 49.4$ kip-in.

$w_u = 1.2w_{DL} + 1.6w_{LL} = 1.2(0.02)+1.6(0.3) = 0.504$ kip/ft.

Maximum required flexural strength $= w_uL^2/8 = 0.504(8)^2(12)/8 = 48.38$ kip-in.

$M_u = 48.38$ kip-in. $< \phi_bM_n = 49.4$ kip-in. OK

4. Strength for Shear Only (Section C3.2)

The required shear strength at any section shall not exceed the design shear strength ϕ_vV_n:

$\sqrt{EK_v/F_y} = \sqrt{29500(5.34)/50} = 56.13$

$1.415\sqrt{EK_v/F_y} = 1.415(56.13) = 79.42$

$h/t = 61.53$

For $\sqrt{EK_v/F_y} < h/t < 1.415\sqrt{EK_v/F_y}$

$\phi_v = 0.90$

$V_n = 0.64t^2\sqrt{K_vF_yE}$ (Eq. C3.2-2)

$= 0.64(0.06)^2\sqrt{5.34(50)(29500)} = 6.47$ kips (per web)

Total V_n for section:

$V_n = 2(6.47) = 12.94$ kips

$\phi_vV_n = 0.90(12.94) = 11.65$ kips

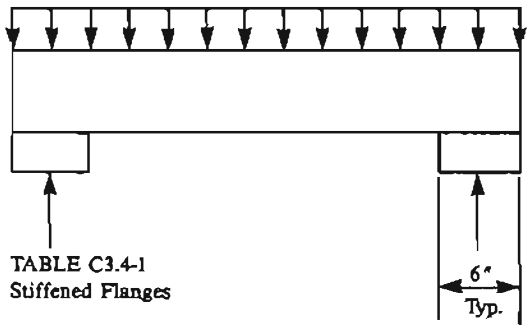
Maximum Required Shear Strength = Reaction

$V_u = w_uL/2 = 0.504(8)/2 = 2.02 \text{ k} < \phi_vV_n = 11.65 \text{ k}$ OK

5. Web Crippling Strength (Section C3.4)

   $R/t = (3/32)/0.06 = 1.563 < 6$ OK

   $h/t = 3.692/0.06 = 61.53 < 200$



$P_n = t^2kC_3C_4C_\theta\,[331\text{-}0.61(h/t)][1+0.01(N/t)]$ (Eq. C3.4-1)

$k = F_y/33 = 50/33 = 1.515$ (Eq. C3.4-21)

$C_3 = (1.33\text{-}0.33k)$ (Eq. C3.4-12)

$= [1.33\text{-}0.33(1.515)] = 0.830\ (1.15\text{-}0.15R/t) = [1.15\text{-}0.15(1.563)]$

$C_4 = (1.15\text{-}0.15R/t) \leq 1.0$ but not less than 0.50 (Eq. C3.4-13)

$= 0.916 \leq 1.0$ OK

> 0.50 OK

$C_4 = 0.916$

$C_\theta = 0.7+0.3(\theta/90)^2$ (Eq. C3.4-20)

$= 0.7+0.3(90/90)^2 = 1.0$

$P_n = (0.06)^2(1.515)(0.830)(0.916)(1.0)[331\text{-}0.61(61.53)]$

$x[1+0.01(6/0.06)] = 2.43$ k/web

$P_n = (2 \text{ webs})(2.43 \text{ k/web}) = 4.86$ k

$\phi_w = 0.75$

$\phi_wP_n = 0.75(4.86) = 3.65$ k

Reaction $= 2.02\text{ k} < \phi_wP_n = 3.65$ k OK

6. Deflection Determination at Service Moment M_s

   Find I_{eff} at $M_s = wL^2/8 = 0.32(8)^2(12)/8 = 30.72$ kip-in.

   a. Procedure I

   Computation of I_{eff}, first approximation

   * Assume a compressive stress of $f = 0.6F_y = 30$ ksi in the top fibers of the section.
   * Also assume web is fully effective.

Element 5:

$\lambda = (1.052/\sqrt{4})(144.9)\sqrt{30/29500} = 2.431 > 0.673$ (Eq. B2.1-4)

$\rho = [1-(0.22/2.431)]/2.431 = 0.374$ (Eq. B2.1-3)

$b_d = \rho w$ (Eq. B2.1-6)

$= 0.374(8.692) = 3.251$ in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
1	2 x 0.596 = 1.192	3.548	4.229	15.005	0.035
2	4 x 0.195 = 0.780	3.925	3.062	12.016	—
3	2 x 2.692 = 5.384	3.970	21.375	84.857	—
4	2 x 3.692 = 7.384	2.000	14.768	29.536	8.388
5	3.251	0.030	0.098	0.003	—
6	2 x 0.195 = 0.390	0.075	0.029	0.002	—
Sum	18.381		43.561	141.419	8.423

Distance of neutral axis from top fiber,

$ycg = Ly/L = 43.561/18.381 = 2.370$ in.

$I'eff = Ly2 + I'_1 - Ly^2_{cg}$

$= 141.419 + 8.423 - 18.381(2.370)^2$

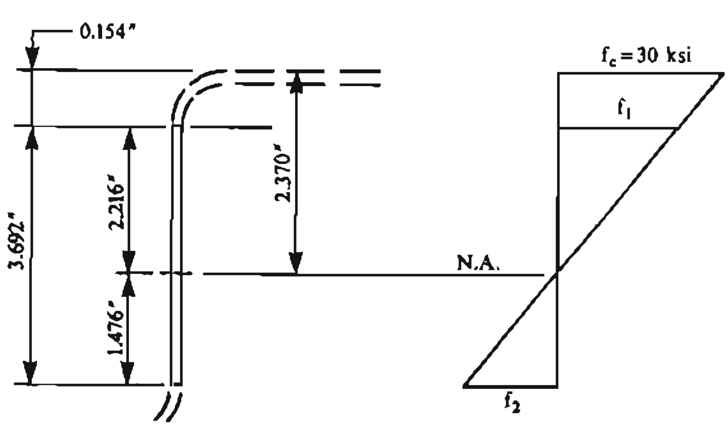
$= 46.60$ in.^{3}

Actual $I_{eff} = tI'_{eff}$

$= (0.060)(46.60) = 2.80$ in.^{4}

Check Web

* Should be fully effective



$f_1 = (2.216/2.370)(30) = 28.05$ ksi(compression)

$f_2 = -(1.476/2.370)(30) = -18.68$ ksi(tension)

$\psi = f_2/f_1 = -18.68/28.05 = -0.666$

$k = 4+2(1-\psi)^3+2(1-\psi)$ (Eq. B2.3-4)

$= 4+2[1-(-0.666)]^3+2[1-(-0.666)] = 16.58$

$\lambda = (1.052/\sqrt{k}(w/t)\sqrt{f/E}$, $f=f_1$ (Eq. B2.1-4)

$= (1.052/\sqrt{16.58})(61.53)\sqrt{28.05/29500} = 0.490 < 0.673$

$b = w$ (Eq. B2.1-1)

$b_e = 3.692$ in.

$b_2 = b_e/2$ (Eq. B2.3-2)

$= 3.692/2 = 1.846$ in.

$b_1 = b_e/(3-\psi)$ (Eq. B2.3-1)

$= 3.692/[3-(-0.666)] = 1.007$ in.

$b_1+b_2 = 1.007 + 1.846 = 2.853$ in. $>$ 2.216 in. (compression portion of web)

Therefore, web is fully effective.

$S_{eff} = I_{eff}/y_{cg} = 2.80/2.370 = 1.18$ in.^{3}

$M = S_{eff}(0.6F_y)$

$= (1.18)(30) = 35.4$ kip-in.

To determine I_{eff} at M = 30.72 kip-in., extrapolate using

(1) M= 52.00 kip-in., I = 2.56 in.^{4}

(2) M= 35.40 kip-in., I= 2.80 in.^{4}

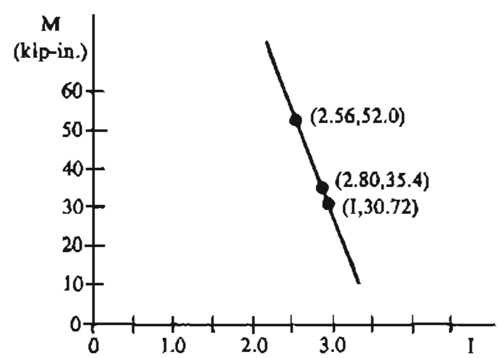
(3) M = 30.72 kip-in., I= ?

$(30.72-35.4)/(I-2.80) = (35.4-52.0)/(2.80-2.56)$

$-4.68 = -69.17(I-2.80)$

$0.0676= I-2.80$

$I = 2.87$ in.^{4}



Use I = 2.87 in.^{4} in deflection calculations

Deflection = $5wL^4/384EI$

b. Procedure II

$\lambda = 0.256+0.328(w/t)\sqrt{F_y/E}$ (Eq. B2.1-10)

$= 0.256+0.328(144.9)\sqrt{50/29500} = 2.213$

Computation of I_{eff}: Check case of $f = F_y$

$\lambda = 3.138 > \lambda_c$

$\rho = (0.41+0.59\sqrt{F_y/f} - 0.22\lambda)/\lambda$ (Eq. B2.1-9)

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

$\rho = 0.296$, which is the same value for Load Capacity Determination

Computation of I_{eff}: Assume a compressive stress $f = 0.6F_y$

$= 30$ ksi in top fiber of section

Note: Web is fully effective

$\lambda \quad = 2.431 > \lambda c$

$\rho \quad = (0.41+0.59\sqrt{F_y/f} - 0.22/\lambda)/\lambda$ (Eq. B2.1-9)

$= (0.41+0.59\sqrt{50/30} - 0.22/2.431)/2.431 = 0.445$

For $\lambda > 0.673$

$b_d \quad = \rho w$ (Eq. B2.1-6)

$= 0.445(8.692) = 3.868$ in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I′1 About Own Axis (in.^{3})
1	2 x 0.596 = 1.192	3.548	4.229	15.005	0.035
2	4 x 0.195 = 0.780	3.925	3.062	12.016	—
3	2 x 2.692 = 5.384	3.970	21.375	84.857	—
4	2 x 3.692 = 7.384	2.000	14.768	29.536	8.388
5	3.868	0.030	0.116	0.004	—
6	2 x 0.195 = 0.390	0.075	0.029	0.002	—
Sum	18.998		43.579	141.420	8.423

Distance of neutral axis from top fiber,

$y_{cg} \quad = Ly/L = 43.579/18.998 = 2.294$ in.

$I'_{eff} \quad = Ly^2 + I'_1 - Ly^2_{cg}$

$= 141.420 + 8.423 - 18.998(2.294)^2$

$= 49.87$ in.^{3}

Actual $I_{eff} = tI'_{eff}$

$= (0.060)(49.87) = 2.99$ in.^{4}

$S_{eff} \quad = I_{eff}/y_{cg} = 2.99/2.294 = 1.30$ in.^{3}

$M \quad = S_{eff}(0.6F_y)$

$= (1.30)(30)$

$= 39.0$ kip-in.

To determine I_{eff} at $M = 30.72$ kip-in., extrapolate using

(1) M = 52.00 kip-in., I = 2.56 $in.^4$

(2) M = 39.00 kip-in., I =2.99 $in.^4$

(3) M = 30.72 kip-in., I = ?

$(30.72-39.0)/(I-2.99) = (39.0-52.0)/(2.99-2.56)$

$-8.28 = -30.23(I-2.99)$

$0.27 = I-2.99, I = 3.26 in.^4$

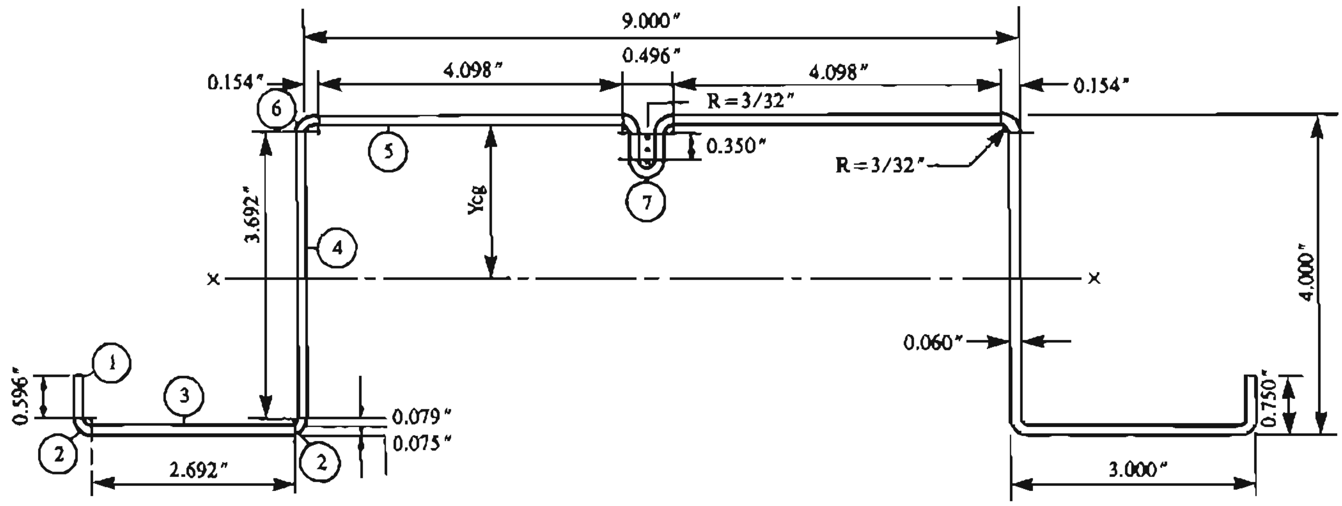
Use I = 3.26 $in.^4$ in deflection calculations

Deflection = $5wL^4/384EI$

EXAMPLE NO. 6

HAT SECTION

Flexural Design, Compression Flange
with Intermediate Stiffener



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: As shown in the sketch.
3. Dead load to live load ratio D/L = 1/5 and 1.2D + 1.6L governs the design.

Required:

1. For the section, determine the design flexural strength, $\phi_b M_n$, and the moment of inertia for deflection calculation.
2. Compare structural economy of this section with an almost identical section without an intermediate stiffener computed in previous example.

Solution:

1. Properties of 90° corners:
   Radius to centerline,

   $r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.

   Length of arc, $u = 1.57r = 1.57 \times 0.124 = 0.195$ in.

Distance of c.g. from center of radius,

$c = 0.637r = 0.637 \times 0.1\ 24 = 0.079$ in.

I' of corner about its own centroidal axis is negligible

2. Nominal Section Strength, M_n (Section C3.1.1)

a. Procedure I - Based on Initiation of Yielding

Computation of I_x, first approximation:

* Assume a compressive stress of $f = F_y = 50$ ksi in the top fibers of the section.

* Also assume web is fully effective.

Element 4:

$h/t = 3.692/0.060 = 61.53 < 200$ OK (Section B1.2-(a)) Assumed fully effective

Element 5:

$S = 1.28\sqrt{E/f}$ (Eq. B4-1)

$= 1.28\sqrt{29500/50} = 31.09$

$b_o/t = 8.692/0.060 = 144.9 < 500$ OK (Section B1.1-(a)-(2))

$3S = 3(31.09) = 93.27$

For $b_o/t > 3S$ (Case III)

$I_a = t^4\{[128(b_o/t)/S] - 285\}$ (Eq. B4.1-9)

$= (0.06)^4\{[128(144.9)/31.09] - 285\} = 0.004038\ in^4$

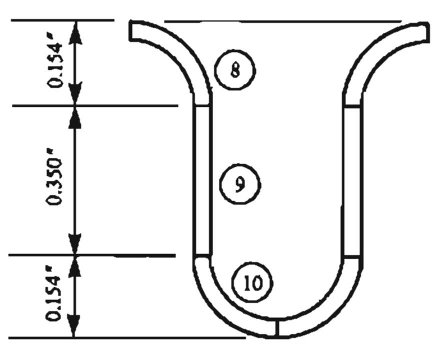
Determine full section properties of stiffener 7:

All inner radii = 3/32 in.

$r = R + t/2 = 3/32 + 0.060/2 = 0.124$ in.

$u = 1.57r = 1.57(0.124) = 0.195$ in.

$c = 0.637r = 0.637(0.124) = 0.0\ 79$ in.



Element	L Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
8	2 x 0.195 = 0.390	0.075	0.0293	0.0022	—
9	2 x 0.350 = 0.700	0.329	0.2303	0.0758	0.0071
10	2 x 0.195 = 0.390	0.583	0.2274	0.1326	—
Sum	1.480		0.4870	0.2106	0.0071

Distance of neutral axis from top fiber,

y_{cg} = Ly/L = 0.4870/1.480 = 0.329 in.

Total area of section, Lt = (1.480)(0.060) = 0.0888 in.^{2}

$I'_s = Ly^2 + I'_1 - Ly^2_{cg}$

$= 0.2106 + 0.0071 - 1.480(0.329)^2$

$= 0.0575$ in.^{3}

Actual $I_s = tI'_s = (0.060)(0.0575) = 0.00345$ in.^{4}

Reduced Area of Stiffener

Element 9:

Stiffened element, k = 4

$f = F_y = 50$ ksi

w/t = 0.350/0.060 = 5.83 < 500 OK (Section B1.1-(a)-(2))

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$(1.052/\sqrt{4})(5.83)\sqrt{50/29500} = 0.126 < 0.673$

b = w (Eq. B2.1-1)

= 0.3 50 in. (fully effective)

$A'_s = Lt = 0.0888$ in.^{2}

$A_s = A'_s(I_s/I_a) \le A'_s$ (Eq. B4.1-11)

= 0.0888(0.00345/0.004038)

= 0.0888(0.8544)= 0.0759 in.^{2} < A'_s OK

$L_s = (A_s/t) = (0.0759/0.060) = 1.265$ in.

Continuing with element 5:

$k = 3(I_s/I_a)^{1/3}+1 \le 4$ (Eq. B4.1-10)

$= 3(0.8544)^{1/3}+1 = 3.847 < 4$ OK

w/t = 4.098/0.060 = 68.30

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{3.847})(68.30)\sqrt{50/29500} = 1.508 > 0.673$

$\rho \quad = (1-0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1-0.22/1.508)/1.508 = 0.566$

$b \quad = \rho w$ (Eq. B2.1-2)

$= 0.566(4.098) = 2.320$ in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly ($in.^2$)	Ly^2 ($in.^3$)	I'_1 About Own Axis ($in.^3$)
1	2 x 0.596 = 1.192	3.548	4.229	15.005	0.035
2	4 x 0.195 = 0.780	3.925	3.062	12.016	—
3	2 x 2.692 = 5.384	3.970	21.375	84.857	—
4	2 x 3.692 = 7.384	2.000	14.768	29.536	8.388
5	2 x 2.320 = 4.640	0.030	0.139	0.004	—
6	2 x 0.195 = 0.390	0.075	0.029	0.002	—
7	Stiffener 1.265	0.329	0.416	0.137	0.058
Sum	21.035		44.018	141.557	8.481

Distance of neutral axis from top fiber, $y_{cg} = Ly/L = 44.018/21.035 = 2.093$ in.

Since the distance of the top compression fiber from the neutral axis is greater than one half the beam depth, a compressive stress of F_y will govern as assumed.

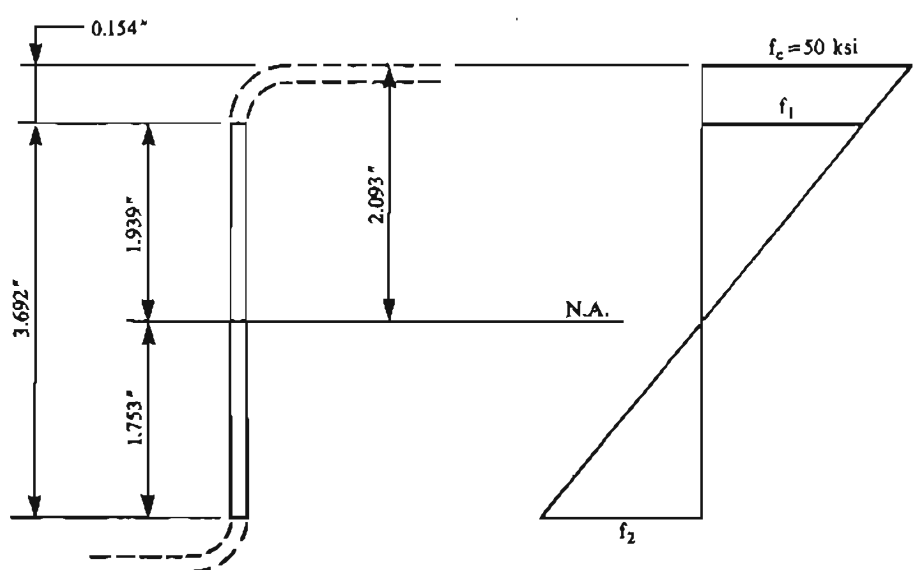
$I'_x \quad = Ly^2 + I'_1 - Ly^2_{cg}$

$= 141.557 + 8.481 - 21.035(2.093)^2$

$= 57.89 \text{ in.}^3$

Actual $I_x \quad = tI'_x = (0.060)(57.89) = 3.47 \text{ in.}^4$

Check Web



$f_1 = (1.939/2.093)(50) = 46.32$ ksi(compression)

$f_2 = -(1.753/2.093)(50) = -41.88$ ksi(tension)

$\psi = f_2/f_1 = -41.88/46.32 = -0.904$

$k = 4+2(1-\psi)^3+2(1-\psi)$ (Eq. B2.3-4)

$= 4+2[1-(-0.904)]^3+2[1-(-0.904)]$

$= 21.61$

$\lambda = (1.052/\sqrt{k})\,(w/t)\,\sqrt{f/E},\ f = f_1$ (Eq. B2.1-4)

$= (1.052/\sqrt{21.61})(61.53)\sqrt{46.32/29500} = 0.552 < 0.673$

$b = w$ (Eq. B2.1-1)

$b_e = 3.692$ in.

$b_2 = b_e/2$ (Eq. B2.3-2)

$= 3.692/2 = 1.846$ in.

$b_1 = b_e/(3-\psi)$ (Eq. B2.3-1)

$= 3.692/[3-(-0.904)] = 0.946$ in.

$b_1+b_2 = 0.946 + 1.846 = 2.792$ in. $>$ 1.939 in. (compression portion of web)

Therefore, web is fully effective.

$S_e = I_x/y_{cg}$

$= 3.47/2.093 = 1.66$ in.^{3}

$M_n = S_eF_y$ (Eq. C3.1.1-1)

$= (1.66)(50) = 83.0$ kip-in.

b. Procedure II - Based on Inelastic Reserve Capacity
For multiple -stiffened compression elements

$C_y = 1$

Maximum compressive strain $= C_ye_y = e_y$

Therefore, the nominal moment, M_n, is the same as the M_n determined by procedure I because the compression flange will yield first.

3. Design Flexural Strength, ϕ_bM_n (Section C3.1)

$\phi_b = 0.95$

$\phi_bM_n = 0.95\text{x}83.0 = 78.85$ kip-in.

4. For complete design, one should also check:
   a. Shear Strength
   b. Combined Bending and Shear, if applicable
   c. Web Crippling Strength

5. Deflection Determination at Service Moment M_s

$\phi_bM_n = 1.2M_{DL} + 1.6M_{LL}$

$= [\,1.2(M_{DL}/M_{LL})+1.6]\,M_{LL}$

$= [\,1.2(1/5)+1.6]\,M_{LL}$

$\phi_b M_n = 1.84 M_{LL}$

$M_{LL} = \phi_b M_n/1.84 = 78.85/1.84 = 42.85$ kip-in.

$M_s = M_{DL} + M_{LL}$

$= (1/5+1)M_{LL}$

$= 1.2(42.85) = 51.42$ kip-in.

where

M_{DL} = Moment determined on the basis of nominal dead load

M_{LL} = Moment determined on the basis of nominal live load

Find I_{eff} at $M_s = 51.42$ kip-in.

Note: Procedure II of Section B2.1-(b)-(2) does not apply.

Computation of I_{eff}, first approximation

* Assume a compressive stress of $f = 0.6F_y = 30$ ksi in the top fibers of the section.
* Web is fully effective, because it was fully effective at a higher stress gradient.
* Element 9 of the stiffener, which was fully effective at f = 50 ksi will also be fully effective at f = 30 ksi.

Element 5:

$S = 1.28\sqrt{E/f}$, $f = 30$ (Eq. B4-1)

$= 1.28\sqrt{29500/30} = 40.14$

$b_o/t = 144.9$

$3S = 3(40.14) = 120.42$

For $b_o/t > 3S$ (Case III)

$I_a = t^4\{[\,128(b_o/t)/S]-285\}$ (Eq. B4.1-9)

$= (0.06)^4\{[\,128(144.9)/40.14]-285\} = 0.002295\text{ in.}^4$

$I_s = 0.00345\text{ in.}^4$

$k = 3(I_s/I_a)^{1/3}+1 \le 4$ (Eq. B4.1-10)

$= 3(0.00345/0.002295)^{1/3}+1 = 4.437 > 4$

$k = 4$

$w/t = 68.30$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E} = 30$ ksi (Eq. B2.1-4)

$= (1.052/\sqrt{4})(68.30)\sqrt{30/29500} = 1.146 > 0.673$

$\rho = (1-0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1-0.22/1.146)/1.146 = 0.705$

$b = \rho w$ (Eq. B2.1-2)

$= 0.705(4.098) = 2.889$ in.

Stiffener, Element 7:

$A_s \quad = A'_s(I_s/I_a) \le A'_s$ (Eq. B4.1-11)

$\quad = 0.0888(0.00345/0.002295)$

$\quad = 0.133 \text{ in.}^2 > A'_s$

$A_s \quad = A'_s = 0.0888 \text{ in.}^2$

$L_s \quad = A_s/t = 0.0888/0.060 = 1.480 \text{ in.}$

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'$_1$ About Own Axis (in.^{3})
1	2 x 0.596 = 1.192	3.548	4.229	15.005	0.035
2	4 x 0.195 = 0.780	3.925	3.062	12.016	—
3	2 x 2.692 = 5.384	3.970	21.375	84.857	—
4	2 x 3.692 = 7.384	2.000	14.768	29.536	8.388
5	2 x 2.889 = 5.778	0.030	0.173	0.005	—
6	2 x 0.195 = 0.390	0.075	0.029	0.002	—
7	Stiffenerd 1.480	0.329	0.487	0.160	0.058
Sum	22.388		44.123	141.581	8.481

Distance of neutral axis from top fiber,

$y_{cg} \quad = Ly/L = 44.123/22.388 = 1.971 \text{ in.}$

$I'_{eff} \quad = Ly^2 + I'_1 - Ly^2_{cg}$

$\quad = 141.581 + 8.481 - 22.388(1.971)^2$

$\quad = 63.09 \text{ in.}^3$

Actual $I_{eff} = tI'_{eff}$

$\quad = (0.060)(63.09) = 3.79 \text{ in.}^4$

$S_{eff} \quad = I_{eff}/y_{cg} = 3.79/1.971 = 1.92 \text{ in.}^3$

$M \quad = S_{eff}(0.6F_y)$

$\quad = (1.92)(30)$

$\quad = 57.6 \text{ kip-in.} > M_s = 51.42 \text{ kip-in.}$

Computation of I_{eff}: second approximation

Extrapolate to obtain the stress value

(1) $M = 83.00$ kip-in., $f = F_y = 50$ ksi

(2) $M = 57.60$ kip-in., $f = 0.6F_y = 30$ ksi

(3) $M = 51.42$ kip-in., $f = ?$

$(f-30)/(30-50) = (51.42-57.6)/(57.6-83.0)$

$f = 25.13$ ksi

* Compressive stress of $f = 25.13$ ksi in the top fiber of section
* Web is fully effective
* Element 9 of stiffener is fully effective

Element 5:

$S = 1.28\sqrt{E/f}$, $f = 25.13$ ksi (Eq. B4-1)

$= 1.28\sqrt{29500/25.13} = 43.86$

$b_o/t = 144.9$

$3S = 3(43.86) = 131.58$

For $b_o/t > 3S$ (Case III)

$I_a = t^4\{[128(b_o/t)/S] - 285\}$ (Eq. B4.1-9)

$= (0.06)^4\{[128(144.9)/43.86] - 285\}; = 0.00179$ in.^{4}

$I_s = 0.00345$ in.^{4}

$k = 3(I_s/I_a)^{1/3} + 1 \leq 4$ (Eq. B4.1-10)

Since $I_s/I_a > 1$, $k = 4$

$w/t = 68.30$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$, $= 25.13$ ksi (Eq. B2.1-4)

$= (1.052/\sqrt{4})(68.30)\sqrt{25.13/29500} = 1.049 > 0.673$

$\rho = (1-0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1-0.22/1.049)/1.049 = 0.753$

$b = \rho w$ (Eq. B2.1-2)

$= 0.753(4.09\ 8) = 3.086$ in.

Stiffener, Element 7:

$A_s = A'_s(I_s/I_a) \leq A'_s$ (Eq. B4.1-11)

Since $I_s/I_a > 1$

$A_s = A'_s = 0.0888$ in.^{2}

$L_s = A_s/t = 0.0888/0.060 = 1.480$ in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly ($in.^2$)	Ly^2 ($in.^3$)	I'_1 About Own Axis ($in.^3$)
1	2 x 0.596 = 1.192	3.548	4.229	15.005	0.035
2	4 x 0.195 = 0.780	3.925	3.062	12.016	—
3	2 x 2.692 = 5.384	3.970	21.375	84.857	—
4	2 x 3.692 = 7.384	2.000	14.768	29.536	8.388
5	2 x 3.086 = 6.172	0.030	0.185	0.006	—
6	2 x 0.195 = 0.390	0.075	0.029	0.002	—
7	Stiffener 1.480	0.329	0.487	0.160	0.058
Sum	22.782		44.135	141.582	8.481

Distance of neutral axis from top fiber,

y_{cg} = Ly/L = 44.135/22.782 = 1.937 in.

I'_{eff} = $Ly^2 + I'_1 - Ly^2_{cg}$

= $141.582 + 8.481 - 22.782(1.937)^2$

= 6 4.59 $in.^3$

Actual I_{eff} = tI'_{eff} = (0.060)(64.59) = 3.88 $in.^4$

S_{eff} = I_{eff}/y_{cg} = 3.88/1.937 = 2.00 $in.^3$

M = (2.00)(25.13) = 50.26 kip-in. Close enough OK

Use I = 3.88 $in.^4$ in deflection calculations

7. Comparison of sections with and without intermediate stiffeners

Section	Total Area ($in.^2$)	Design Flexural Strength (kip-in.)
Without Stiffener	1.43	49.40
With Stiffener	1.49	78.85

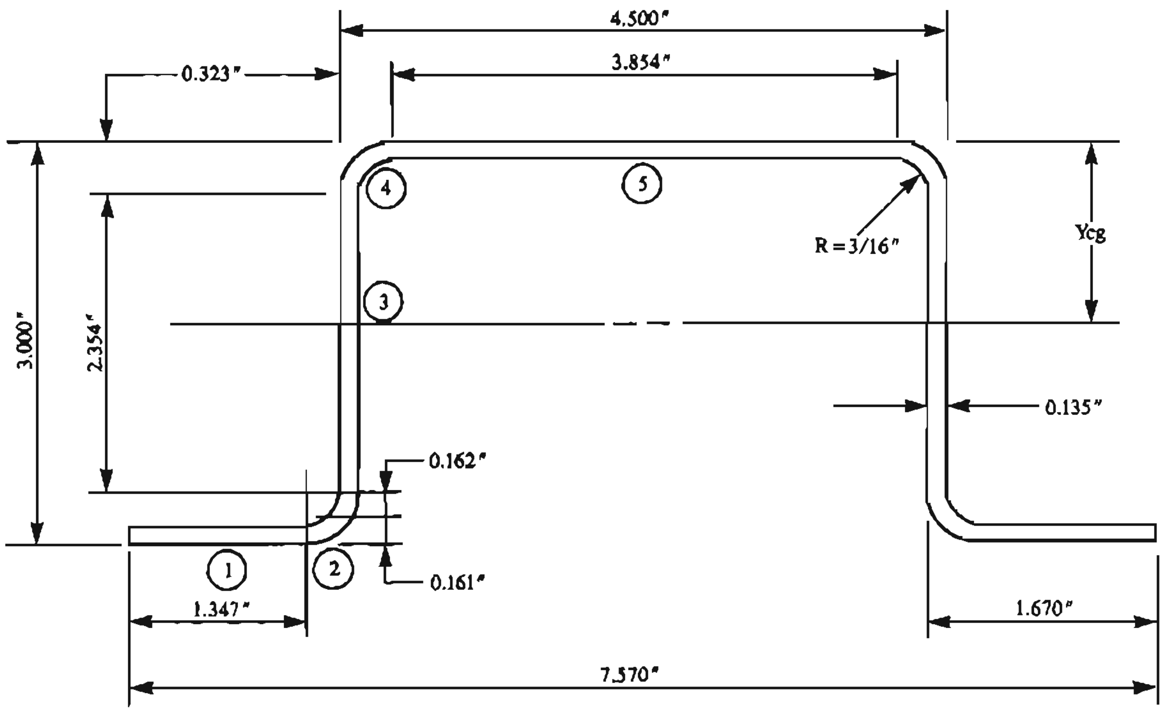
Increase in weight = [(1.49-1.43)/1.43]x100% = 4.2%

Increase in moment capacity = [(78.85-49.40)/49.40] x100% = 59.6%

EXAMPLE NO. 7

HAT SECTION

Using Inelastic Reserve Capacity



Given :

1. Steel: $F_y = 50$ ksi.
2. Section: As shown in sketch.
3. Top flange continuously supported.
4. Span = 8 ft. (simply supported).

Required:

Determine design flexural strength, $\phi_b M_n$.

Solution:

1. Properties of 90° corners:

   Radius to centerline,

   $r = R + t/2 = 3/16 + 0.135/2 = 0.255$ in.

   Length of arc, $u = 1.57r = 1.57 \times 0.255 = 0.400$ in.

   Distance of c.g. from center of radius,

   $c = 0.637r = 0.637 \times 0.255 = 0.162$ in.

I' of corner about its own centroidal axis = $0.149r^3 = 0.149(0.255)^3 = 0.003$ in.^{3}. This is negligible.

2. Nominal Section Strength (Section C3.1.1)

a. Procedure I - Based on Initiation of Yielding

Computation of I_x, first approximation:

* Assume a compressive stress of $f = F_y = 50$ ksi in the top fibers of the section.
* Assume web is fully effective.

Element 3:

h/t $= 2.354/0.135 = 17.44 < 200$ OK (Section B1.2-(a)). Assumed fully effective

Element 5:

w/t $= 3.854/0.135 = 28.55 < 500$ OK (Section B1.1-(a)-(2))

k $= 4$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{4})(28.55)\sqrt{50/29500} = 0.618 < 0.673$

b $= w$ (Eq. B2.1-1)

$= 3.854$ in. (Fully effective)

Effect ive section properties about x axis:=

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
1	2 x 1.347 = 2.694	2.933	7.902	23.175	—
2	2 x 0.400 = 0.800	2.839	2.271	6.448	—
3	2 x 2.354 = 4.708	1.500	7.062	10.593	2.174
4	2 x 0.400 = 0.800	0.161	0.129	0.021	—
5	3.854	0.068	0.262	0.018	—
Sum	12.856		17.626	40.255	2.174

Distance of neutral axis from top fiber,

$y_{cg} = Ly/L = 17.626/12.856 = 1.371$ in.

Since the distance of the top compression fiber from the neutral axis is less than one half of the beam depth, a compressive stress of F_y will not govern as assumed. The actual compressive stress will be less than F_y and so the flange will still be fully effective. The tension flange will yield first. Section properties will not change.

Therefore,

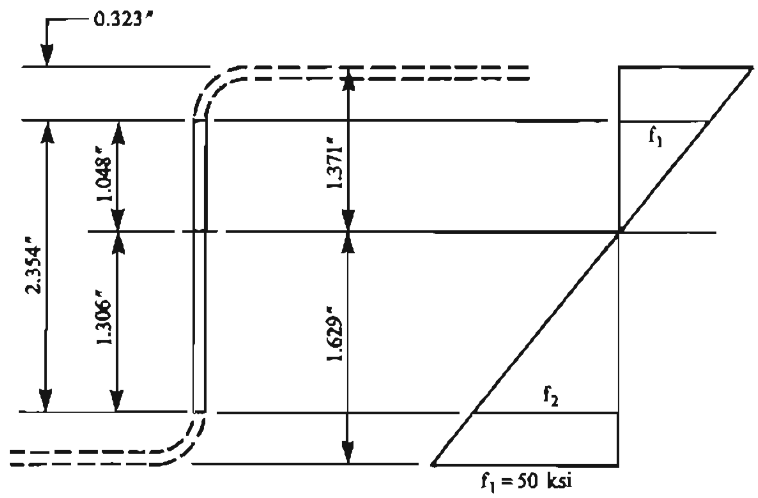
$$I'_x = Ly^2 + I'_1 - Ly^2_{cg}$$

$= 40.255 + 2.\ 174 - 12.856(1.371)^2$

$= 18.26 \text{ in.}^3$

Actual $I_x = tI'_x = (0.135)(18.26) = 2.47 \text{ in.}^4$

Check Web



$f_1 = (1.048/1.629)(50) = 32.17$ ksi(compression)

$f_2 = -(1.306/1.629)(50) = -40.09$ ksi(tension)

$\psi = f_2/f_1 = -40.09/32.17 = -1.246$

$k = 4+2(1-\psi)^3+2(1-\psi)$ (Eq. B2.3-4)

$= 4+2[1-(-1.246)]^3+2[1-(-1.246)]$

$= 31.15$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E},\ f = f_1$ (Eq. B2.1-4)

$= (1.052/\sqrt{31.15})(17.44)\sqrt{32.17/29500} = 0.109 < 0.673$

$b = w$ (Eq. B2.1-1)

$b_e = 2.354$ in.

$b_2 = b_e/2$ (Eq. B2.3-2)

$= 2.354/2 = 1.177$ in.

$b_1 = b_e/(3-\psi)$ (Eq. B2.3-1)

$= 2.354/[3-(-1.246)] = 0.554$ in.

$b_1+b_2 = 0.554 + 1.177 = 1.731 \text{ in.} > 1.048$ in. (compression portion of web)

Therefore, web is fully effective.

$S_e = I_x/(d-y_{cg}) = 2.47/(3-1.371) = 1.516 \text{ in.}^3$

$M_n = S_eF_y$ (Eq. C3.1.1-1)

$= (1.516)(50)$

$= 75.8$ kip-in.

b. Procedure II - Based on Inelastic Reserve Capacity

$\lambda_1 = (1.11/\sqrt{F_y/E})$ (Eq. C3.1.1-2)

$= (1.11/\sqrt{50/29500}) = 26.96$

$\lambda_2 = (1.28/\sqrt{F_y/E})$ (Eq. C3.1.1-3)

$= (1.28/\sqrt{50/29500}) = 31.09$

$w/t = 28.55$

For $26.96 = \lambda_1 < w/t < \lambda_2 = 31.09$

$C_y = 3\text{-}2[(w/t\text{-}\lambda_1)/(\lambda_2 - \lambda_1)]$

$= 3\text{-}2[(28.55\text{-}26.96)/(31.09\text{-}26.96)] = 2.23$

Compute location of e_y on strain diagram, the summation of longitudinal forces should be zero. Using equations from Reck, Pekoz, and Winter, "Inelastic Strength of Cold-Formed Steel Beams," Journal of the Structural Division, November 1975, ASCE.

Distance from neutral axis to the outer compression fiber, y_c:

$t = 0.135$ in.

$b_t = 2(1.670) = 3.340$ in.

$b_c = 4.500$ in.

$d = 3.000$ in.

$y_c = (1/4)(b_t\text{-}b_c+2d)$

$y_c = (1/4)[3.340\text{-}4.500+2(3.000)] = 1.210$ in.

$y_p = y_c/C_y$

$= 1.21/2.23 = 0.543$ in.

$y_t = d\text{-}y_c$

$= 3.000 - 1.210 = 1.790$ in.

$y_{cp} = y_c\text{-}y_p$

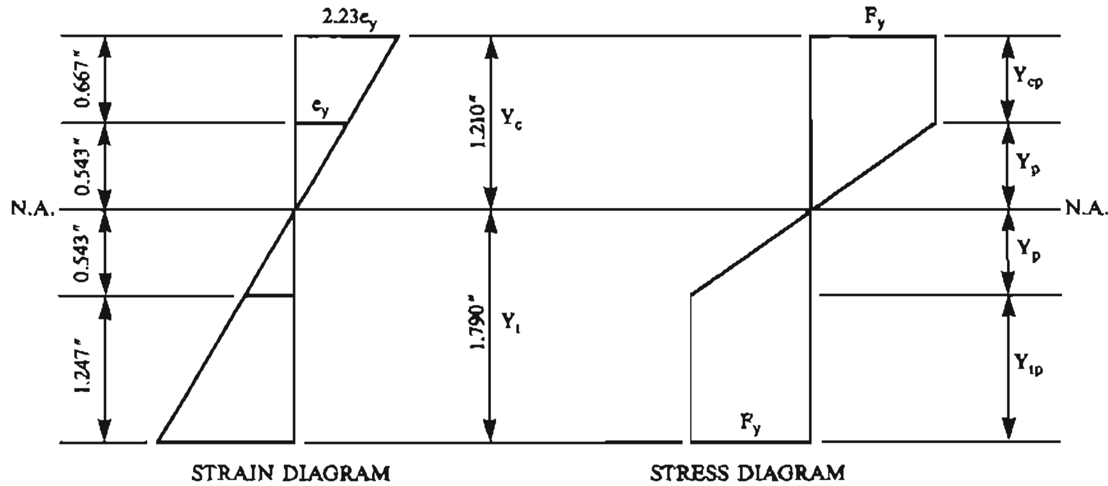
$= 1.210\text{-}0.543 = 0.667$ in.

$y_{tp} = y_t\text{-}y_p = 1.790\text{-}0.543 = 1.247$ in.

Summing moments of stresses in component plates:

$$M_n = F_y t\{b_c y_c + 2y_{cp}[y_p + (y_{cp}/2)] + (4/3)y_p^2$$

$$+2y_{tp}[y_p + (y_{tp}/2)] + b_t y_t\}$$



$$M_n = 50(0.135)\{4.500(1.210)+2(0.667)[0.543+(0.667/2)] +(4/3)(0.543)^2+2(1.247)[0.543+(1.247/2)]+3.340(1.790)\}$$

M_n = 107.3 kip-in.

M_n shall not exceed $1.25S_eF_y = 1.25(75.8) = 94.75$ kip-in.

Therefore

$M_n = 1.25S_eF_y = 94.75$ kip-in.

The inelastic reserve capacity can be used because:

(1) Member is not subject to twisting, 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 depth of the compressed portion of the web to its thickness does not exceed λ_1, $(1.210-0.323)/0.135 = 6.57 < l_1 = 26.96$ OK

(4) The shear force does not exceed $0.35F_y$ times the web area, h x t.
This still needs to be checked for a complete design.

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

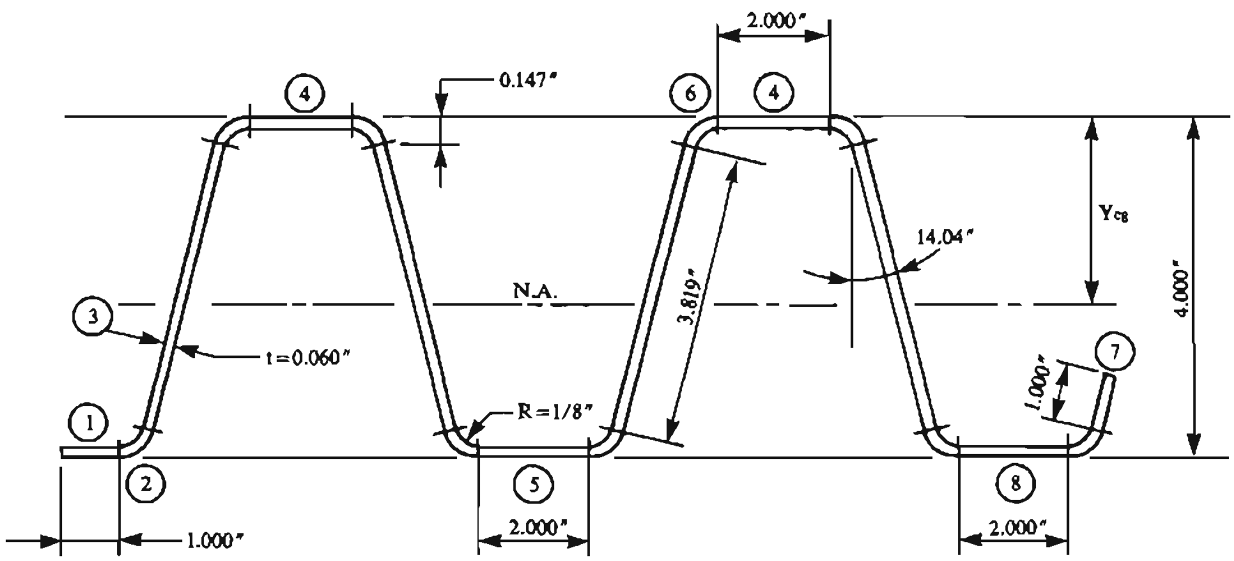
3. Design Flexural Strength, ϕ_bM_n

$\phi_b = 0.95$

$\phi_bM_n = 0.95 \times 94.75 = 90.01$ kip-in.

EXAMPLE NO. 8

DECK SECTION



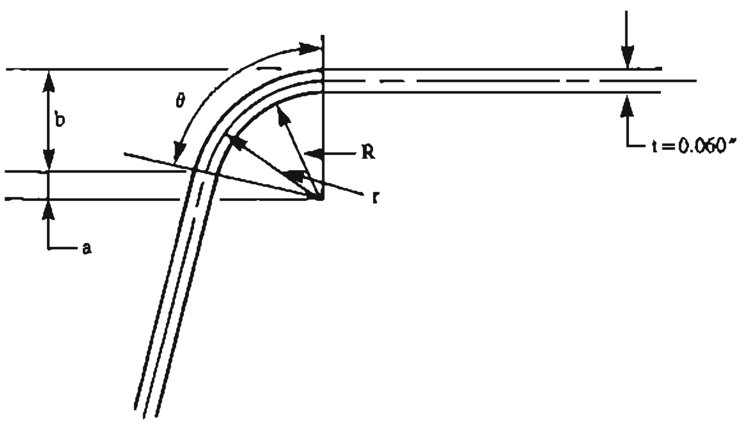
Given:

1. Steel: F_y = 50 ksi.
2. Section: Shown in sketch above.
3. Dead load to live load ratio D/L = 1/5 and 1.2D + 1.6L governs the design.

Required:

1. Design flexural strengths, $\phi_b M_n$, for positive and negative bending.
2. Factored uniform load, w_u, as controlled either by bending or deflection when deck is continuous over three 10'-0" spans. Deflection due to service live load is to be limited to 1/240 of the span.

Corner Properties:



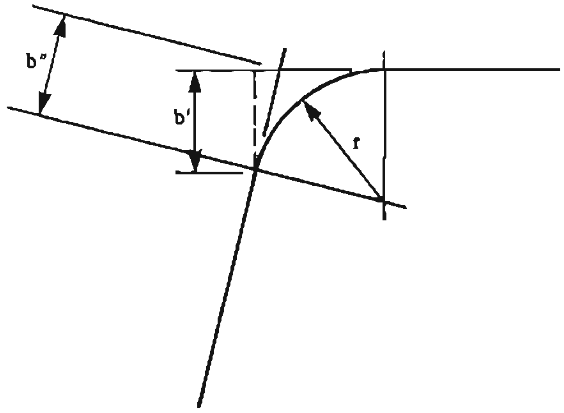
$\theta = 75.96°$

$R = 1/8''$

$r = 0.155''$

$a = r\sin(90° - 75.96°)$
$= 0.155''\sin 14.04°$
$= 0.0376''$

$b = t/2 + r - a$
$= 0.060''/2 + 0.155'' - 0.0376''$
$= 0.147''$

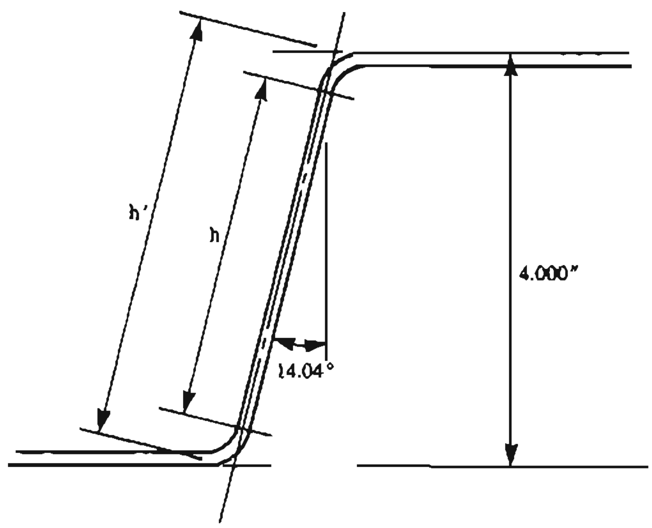


$b' = b - t/2$
$= 0.147'' - 0.060''/2$
$= 0.117''$

$b'/b'' = \cos(90° - 75.96°)$

$b'' = b'/\cos 14.04°$
$= 0.117''/\cos 14.04°$
$= 0.112''$

Flat Portion of Web



h' = 4.000"/cos 14.04°

= 4.123"

h = h'-2b"-t/cos14.04°

= 4.123"-2(0.121")

-0.06"/cos14.04°

= 3.819"

Solution:

1. Full Section Properties:

Elements 2 and 6:

Radius to centerline,

$r = R + t/2 = 1/8 + 0.060/2 = 0.155$ in.

Angle, $\theta = 75.96° = 1.326$ rad

Length of arc, $u = \theta r = 1.326(0.155) = 0.206$ in.

Distance of c.g. from center of radius,

$c_1 = r\sin\theta/\theta = 0.155\sin 75.96°/1.326 = 0.113$ in.

I'_1 of arc element about its own centroidal axis is negligible.

Element 3:

$l = 3.819$ in.

$\theta = 14.04°$

$\cos\theta = 0.9701$

$I'_1 = (\cos^2\theta l^3)/12 = [(0.9701)2\ (3.819)\ 3]/12 = 4.368\ \text{in.}^3$

Element 7:

$l = 1.000$ in.

$\theta = 14.04°$

$\cos\theta = 0.9701$

$I'_1 = (\cos^2\theta l^3)/12 = [\,(0.9701)^2\,(1)^3]\,/12 = 0.0784$ in.3

Distance of centroid of full section from top fiber,

y = 4-0.147-(1.000/2)cos14.04° = 3.368 in.

2. Section Modulus for Load Determination - Positive Bending (Based on Procedure I)

Since the effective design width of flat compressive elements is a function of stress, iteration is required.

Computation of I_x, first approximation:

* Assume a compressive stress of $f = F_y = 50$ ksi in the top fibers of the section.
* Assume web is fully effective.

Element 3:

h/t = 3.819/0.060 = 63.65 200 OK (Section B1.2-(a)). Assumed fully effective

Element 4:

w/t = 2.000/0.060 = 33.33 < 500 OK (Section B1.1-(a)-(2))

k = 4

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E},\ f = F_y$ (Eq. B2.1-4)

$= (1.052/\sqrt{4})(33.33)\sqrt{50/29500} = 0.722 > 0.673$

$\rho = (1-0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1-0.22/0.722)/0.722 = 0.963$

$b = \rho w$ (Eq. B2.1-2)

$= 0.963 \times 2 = 1.926$ in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
1	1.000	3.970	3.970	15.761	—
2	5 x 0.206 = 1.030	3.928	4.046	15.892	—
3	4 x 3.819 =15.276	2.000	30.552	61.104	17.472
4	2 x 1.926 = 3.852	0.030	0.116	0.004	—
5 &8	2 x 2.000 = 4.000	3.970	15.880	63.044	—
6	4 x 0.206 = 0.824	0.072	0.059	0.004	—
7	1.000	3.368	3.368	11.343	0.078
Sum	26.982		57.991	167.152	17.550

Distance of neutral axis from top fiber,

$y_{cg} = Ly/L = 57.991/26.982 = 2.149$ in.

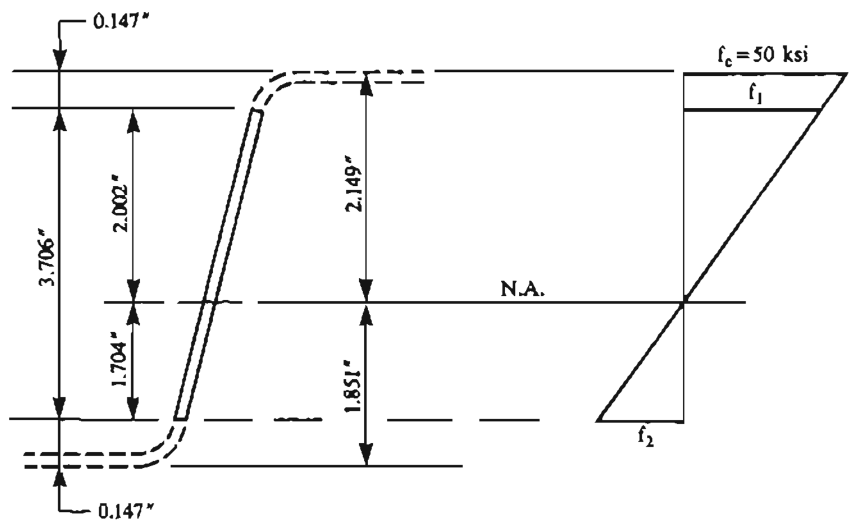
Since the distance of the top compression fiber from the neutral axis is greater than one half of the deck depth, a compressive stress of F_y will govern as assumed.

$I'_x = Ly^2 + I'_1 - Ly^2_{cg} = 167.152 + 17.550 - 26.982(2.149)^2$

$= 60.09$ in.^{3}

Actual $I_x = tI'_x = (0.060)(60.09) = 3.61$ in.^{4}

Check Web



$f_1 = (2.002/2.149)(50) = 46.58$ ksi(compression)

$f_2 = -(1.704/2.149)(50) = -39.65$ ksi(tension)

$\psi = f_2/f_1 = -39.65/46.58 = -0.851$

$k = 4+2(1-\psi)^3+2(1-\psi)$ (Eq. B2.3-4)

$= 4+2[1-(-0.851)]^3 +2[1-(-0.851)] = 20.39$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E},\ f= f_1$ (Eq. B2.1-4)

$= (1.052/\sqrt{20.39})(63.65)\sqrt{46.58/29500} = 0.589 < 0.673$

$b = w$ (Eq. B2.1-1)

$b_e = 3.819$ in.

$b_2 = b_e/2$ (Eq. B2.3-2)

$= 3.819/2 = 1.910$ in.

$b_1 = b_e/(3-\psi)$ (Eq. B2.3-1)

$= 3.819/[3-(-0.851)] = 0.992$ in.

$b_1+b_2 = 0.992 + 1.910 = 2.902$ in. > 2.002 in. (compression portion of web)

Therefore, web is fully effective.

$S_e \;= I_x/y_{cg}$

$\quad = 3.61/2.149 = 1.68 \text{ in.}^3$

$M_n \;= S_eF_y$ (Eq. C3.1.1-1)

$\quad = (1.68)(50)$

$\quad = 84.0$ kip-in

$\phi_b \;= 0.95$

$\phi_bM_n = 0.95 \times 84.0 = 79.8$ kip-in.

3. Moment of Inertia for Deflection Determination - Positive Bending

$\phi_bM_n = 1.2M_{DL} + 1.6M_{LL}$

$\quad = [\,1.2(M_{DL}/M_{LL}) + 1.6]M_{LL}$

$\quad = [\,1.2(1/5) + 1.6]\,M_{LL}$

$\quad = 1.84M_{LL}$

$M_{LL} \;= \phi_bM_n/1.84 = 79.80/1.84 = 43.37$ kip-in.

$M_s \;= M_{DL} + M_{LL}$

$\quad = (1/5+1)M_{LL}$

$\quad = 1.2(43.37) = 52.04$ kip-in.

where

M_{DL}= Moment determined on the basis of nominal dead load

M_{LL}= Moment determined on the basis of nominal live load

Computation of I_{eff}, first approximation:

* Assume a compressive stress of $f = 31$ ksi in the top fibers of the section.
* Since the web was fully effective at a higher stress gradient, it will be fully effective at this stress level.

Element 4:

$w/t \;= 33.33$

$k \;= 4$

$\lambda \;= (1.052/\sqrt{k})(w/t)\sqrt{f/E}$, (Eq. B2.1-4)

$\quad = (1.052/\sqrt{4})(33.33)\sqrt{31/29500} = 0.568 < 0.673$

$b_d \;= w$ (Eq. B2.1-5)

$\quad = 2.000$ in. (Fully effective)

Note: All elements are fully effective.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly ($in.^2$)	Ly^2 ($in.^3$)	I'_1 About Own Axis ($in.^3$)
1	1.000	3.970	3.970	15.761	—
2	5 x 0.206 = 1.030	3.928	4.046	15.892	—
3	4 x 3.819 =15.276	2.000	30.552	61.104	17.472
4	2 x 2.000 = 4.000	0.030	0.120	0.004	—
5 & 8	2 x 2.000 = 4.000	3.970	15.880	63.044	—
6	4 x 0.206 = 0.824	0.072	0.059	0.004	—
7	1.000	3.368	3.368	11.343	0.078
Sum	27.130		57.995	167.152	17.550

Distance of neutral axis from top fiber,

y_{cg} = Ly/L = 57.995/27.130 = 2.138 in.

Since the distance of the top compression fiber from teh neutral axis is greater than one half the deck depth, the compressive stress of 31 ksi will govern as assumed.

$I'_{eff} = Ly^2 + I'_1 - Ly^2_{cg}$

$= 167.152 = 17.550 - 27.130(2.138)^2$

$= 60.69 \text{ in.}^3$

Actual $I_{eff} = tI'_{eff}$

$= (0.060)(60.69) = 3.64 \text{ in.}^4$

$S_{eff} = I_{eff}/y_{cg} = 3.64/2.138 = 1.70 \text{ in.}^3$

$M = S_{eff}(31) = (1.70)(31)$

$= 52.7 \text{ kip-in.} > M_s = 52.04$

$I_{eff} = 3.64 \text{ in.}^4$ can be used for deflection calculations for the following reasons:

(1) M (I_{eff} = 3.64) ≈ M_s

(2) In order to achieve M (I_{eff}) = M_s a lower compressive stress would be required, but the section is already fully effective.

$I_{eff} = 3.64 \text{ in.}^4$

4. Section Modulus for Load Determination - Negative Bending (Based on Procedure I)

Following a similar procedure as in positive bending.

Computation of I_x, first approximation:

* Assume a compressive stress of $f = F_y$ = 50 ksi in the bottom fiber of the section.
* Assume web is fully effective.

Element 3:

h/t = 3.819/0.060 = 63.65 < 200 OK (Section B1.2-(a)). Assumed fully effective

Element 1:

$w/t = 1.000/0.060 = 16.67 \;\; 60$ OK (Section B1.1-(a)-(3))

$k = 0.43$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{0.43})(16.67)\sqrt{50/29500} = 1.101 > 0.673$

$\rho \;\; (1-0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1-0.22/1.101)/1.101 = 0.727$

$b = \rho w$ (Eq. B2.1-2)

$= 0.727 \times 1.000$

$= 0.727$ in.

Element 5:

Same as element 4 in positive bending case

$b = 1.926$ in.

Element 8:

$w/t = 2.000/0.060 = 33.33 < 60$ OK (Section B1.1-(a)-(3))

$S = 1.28\sqrt{E/f}$ (Eq. B4-1)

$= 1.28\sqrt{29500/50} = 31.09$

For $w/t > S$

$I_a = t^4\{[115(w/t)/S] + 5\}$ (Eq. B4.2-13)

$= (0.060)^4\{[115(33.33)/31.09] + 5\}$

$= 0.00166 \text{ in.}^4$

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

$= (1.000)^3(0.060)(\sin 75.96°)/12 = 0.00471$ in.4

$D = 1.000 + 0.185\tan(75.96°/2) = 1.144$ in.

$D/w = 1.144/2.000 = 0.572$

For $0.25 < D/w < 0.80$

$k = [4.82-5(D/w)]\,(Is/Ia)1/3+0.43 \leq 5.25-5(D/w)$ (Eq. B4.2-9)

$[4.82-5(0.572)]\,(0.00471/0.00166)^{1/3}+0.43 = 3.205$

$5.25-5(0.572) = 2.390$

$k = 2.390$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{2.390})(33.33)\sqrt{50/29500} = 0.934 > 0.673$

$\rho = (1-0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1-0.22/0.934)/0.934 = 0.818$

$b = \rho w$ (Eq. B2.1-2)

$= 0.818(2.000) = 1.636$ in.

Element 7:

I_s $= 0.00471$ in.^{4} (calculated previously)

I_a $= 0.00166$ in.^{4} (calculated previously)

d $= 1.000$ in.

Assume max. stress in element, $f = F_y = 50$ ksi although it will be actually less.

k $= 0.43$

w/t $= 1.000/0.060 = 16.67 > 14$ (Section B4 of the Commentary)

λ $= (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{0.43})(16.67)\sqrt{50/29500} = 1.101 > 0.673$

ρ $= (1-0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1-0.22/1.101)/1.101 = 0.727$

b $= \rho w$ (Eq. B2.1-2)

$= 0.727(1.000) = 0.727$ in.

d'_s $= 0.727$ in.

$d_s = d'_s(I_s/I_a) \leq d'_s$ (Eq. B4.2-11)

Since $I_s/I_a > 1$

d_s $= d'_s = 0.727$ in.

I'_1 $= (d_s)^3\sin^2\theta/12 = (0.727)^3(\sin 75.96°)2/12 = 0.030$ in.3

Distance of centroid of reduced section from top fiber,

y $= 4-0.147-(0.727/2)\cos 14.04° = 3.500$ in.

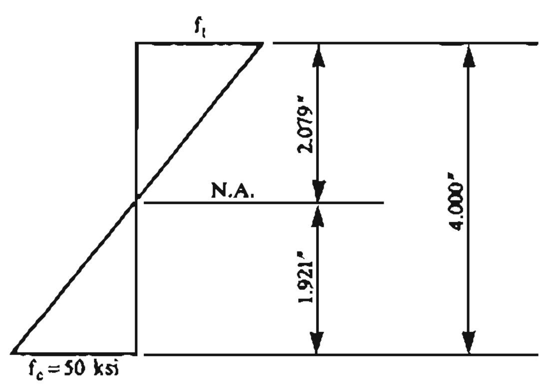
Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly ($in.^2$)	Ly^2 ($in.^3$)	I'_1 About Own Axis ($in.^3$)
1	0.727	3.970	2.886	11.458	—
2	5 x 0.206 = 1.030	3.928	4.046	15.892	—
3	4 x 3.819 =15.276	2.000	30.552	61.104	17.472
4	2 x 2.000 = 4.000	0.030	0.120	0.004	—
5	1.926	3.970	7.646	30.355	—
6	4 x 0.206 = 0.824	0.072	0.059	0.004	—
7	0.727	3.500	2.545	8.906	0.030
8	1.636	3.970	6.495	25.785	—
Sum	26.146		54.349	153.508	17.502

Distance of neutral axis from top fiber,

$y_{cg} = Ly/L = 54.349/26.146 = 2.079$ in.

Since the distance of the top fiber from the neutral axis is greater than one half the deck depth, a compressive stress of F_y will not govern as assumed. The compressive stress will be slightly less.



$f_t = (2.079/1.921)(50)$
$= 54.11 \text{ ksi} > F_y$

Computation of I_x, second approximation:

* Assume a compressive stress of f = 45 ksi
* Assume web is fully effective.

Element 1:

w/t = 16.67

k = 0.43

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{0.43})(16.67)\sqrt{45/29500} = 1.045 > 0.673$

$\rho = (1\text{-}0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1\text{-}0.22/1.045)/1.045 = 0.756$

$b = \rho w$ (Eq. B2.1-2)

$= 0.756 \times 1.000$

$= 0.756$ in.

Element 5:

$w/t = 33.33$

$k = 4$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{4})(33.33)\sqrt{45/29500} = 0.685 > 0.673$

$\rho = (1\text{-}0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1\text{-}0.22/0.685)/0.685 = 0.991$

$b = \rho w$ Eq. B2.1-2)

$= 0.991 \times 2.000 = 1.982$ in.

Element 8:

$w/t = 33.33$

$S = 1.28\sqrt{E/f}$ (Eq. B4-1)

$= 1.28\sqrt{29500/45} = 32.77$

For $w/t > S$

$I_a = t^4\{[115(w/t)/S] + 5\};$ (Eq. B4.2-13)

$= (0.060)^4\{[115(33.33)/32.77] + 5\} = 0.00158$ in.^{4}

$I_s = 0.00471$ in.^{4} (calculated previously)

$D = 1.144$ in. (calculated previously)

$D/w = 0.572$ (calculated previously)

For $0.25 < D/w < 0.80$

$k = [4.82\text{-}5(D/w)](I_s/I_a)^{1/3} + 0.43 \leq 5.25\text{-}5(D/w)$ (Eq. B4.2-9)

$[4.82\text{-}5(0.572)](0.00471/0.00158)^{1/3} + 0.43 = 3.251$

$5.25\text{-}5(0.572) = 2.390$

$k = 2.390$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{2.390})(33.33)\sqrt{45/29500} = 0.886 > 0.673$

$\rho = (1\text{-}0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1\text{-}0.22/0.886)/0.886 = 0.848$

$b = \rho w$ (Eq. B2.1-2)

$= 0.848(2.000) = 1.696$ in.

Element 7:

I_s = 0.00471 in.^{4} (calculated previously)

I_a = 0.00158 in.^{4} (calculated previously)

d = 1.000 in.

Assume max. stress in element, f = 45 ksi although it will be actually less.

k = 0.43

w/t = 16.67

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{0.43})(16.67)\sqrt{45/29500} = 1.045 > 0.673$

$\rho = (1-0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1-0.22/1.045)/1.045 = 0.756$

$b = \rho w$ (Eq. B2.1-2)

$= 0.756(1.000) = 0.756$ in.

$d'_s = 0.756$ in.

$d_s = d'_s(I_s/I_a) \leq d'_s$ (Eq. B4.2-11)

Since $I_s/I_a > 1$

$d_s = d'_s = 0.756$ in.

$I'_1 = (d_s)^3\sin^2\theta/12 = (0.756)^3(\sin 75.96°)2/12 = 0.034$ in.3

Distance of centroid of reduced section from top fiber,

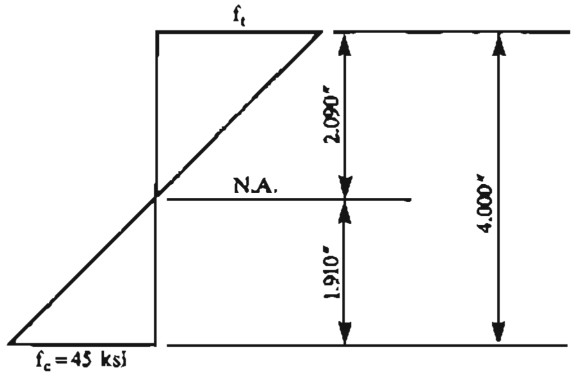
$y = 4-0.147-(0.756/2)\cos 14.04° = 3.486$ in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
1	0.756	3.970	3.001	11.915	—
2	5 x 0.206 = 1.030	3.928	4.046	15.892	—
3	4 x 3.819 =15.276	2.000	30.552	61.104	17.472
4	2 x 2.000 = 4.000	0.030	0.120	0.004	—
5	1.982	3.970	7.869	31.238	—
6	4 x 0.206 = 0.824	0.072	0.059	0.004	—
7	0.756	3.486	2.635	9.187	0.034
8	1.696	3.970	6.733	26.730	—
Sum	26.320		55.015	156.074	17.506

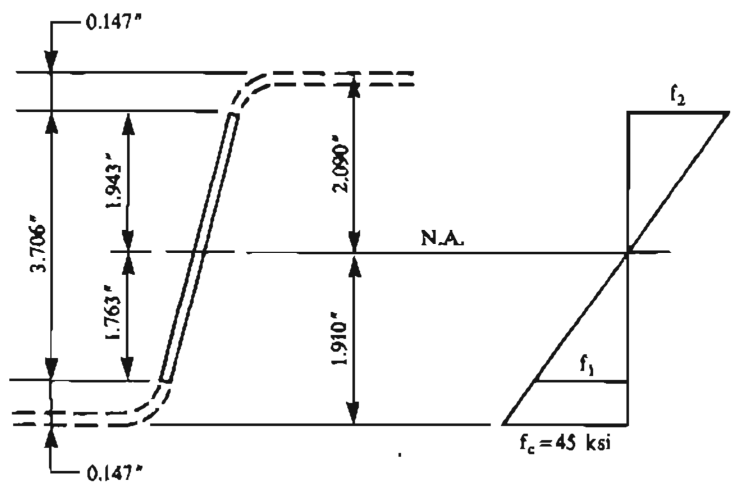
Distance of neutral axis from top fiber,

$y_{cg} = Ly/L = 55.015/26.320 = 2.090$ in.



$f_t = (2.090/1.910)(45)$
$= 49.24$ ksi Satisfactory

Check Web



$f_1 = (1.763/1.910)(45) = 41.54$ ksi (compression)

$f_2 = -(1.943/1.910)(45) = -45.78$ ksi (tension)

$\psi = f_2/f_1 = -45.78/41.54 = -1.102$

$k = 4+2(1-\psi)^3+2(1-\psi)$ (Eq. B2.3-4)

$= 4+2[1-(-1.102)]^3+2[1-(-1.102)]$

$= 26.78$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$, $f = f_1$ (Eq. B2.1-4)

$= (1.052/\sqrt{26.78})(63.65)\sqrt{41.54/29500} = 0.486 < 0.673$

$b = w$ (Eq. B2.1-1)

b_e = 3.819 in.

b_2 = $b_e/2$ (Eq. B2.3-2)

= 3.819/2 = 1.910 in.

b_1 = $b_e/(3-\psi)$ (Eq. B2.3-1)

= 3.819/[3-(-1.102)] = 0.931 in.

b_1+b_2 = 0.931 + 1.910 = 2.841 in. > 1.763 in. (compressionportion of web)

Therefore, web is fully effective.

Check Element 7:

Maximum stress in element, f = 41.54 ksi

k = 0.43, w/t = 16.67

λ = $(1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

= $(1.052/\sqrt{0.43})(16.67)\sqrt{41.54/29500}$ = 1.004 > 0.673

ρ = $(1-0.22/\lambda)/\lambda$ (Eq. B2.1-3)

= (1-0.22/1.004)/1.004 = 0.778

b = ρw (Eq. B2.1-2)

= 0.778(1.000) = 0.778 in.

d'_s = 0.778 in.

d_s = $d'_s(I_s/I_a) \leq d'_s$ (Eq. B4.2-11)

Since $I_s/I_a > 1$

d_s = d'_s = 0.778 in.

I'_1 = $(d_s)^3\sin^2\theta/12 = (0.778)^3(\sin 75.96°)2/12$ = 0.037 in.3

Distance of centroid of reduced section from top fiber,

y = 4-0.147-(0.778/2)cos14.04° = 3.476 in.

Determine section properties, but only the properties of element 7 have changed

ΔL = 0.778-0.756 = 0.022 in.

ΔLy = (0.778)(3.476)-2.635 = 0.069 in.^{2}

ΔLy^2 = $0.778(3.476)^2$-9.187 = 0.213 in.^{3}

$\Delta I'_1$ = 0.037-0.034 = 0.003 in.^{3}

Therefore,

L = 26.320+0.022 = 26.342 in.

Ly = 55.015+0.069 = 55.084 in.^{2}

Ly^2 = 156.074+0.213 = 156.287 in.^{3}

I'_1 = 17.506+0.003 = 17.509 in.^{3}

Distance of neutral axis from top fiber,

y_{cg} = Ly/L = 55.084/26.342 = 2.091 in.

f_t = (2.091/1.909)(45) = 49.29 ksi ≈ 50 ksi OK

$I'_x = Ly^2 + I'_1 - Ly^2_{cg}$
$= 156.287 + 17.509 - 26.342(2.091)^2$
$= 58.62 \text{ in.}^3$

Actual $I_x = tI'_x$
$= (0.060)(58.62) = 3.52 \text{ in.}^4$

$S_e = I_x/y_{cg}$
$= 3.52/2.091$
$= 1.68 \text{ in.}^3$

$M_n = S_eF_y$ (Eq. C3.1.1-1)
$= (1.68)(50)$
$= 84.0$ kip-in.

$\phi_b = 0.90$

$\phi_bM_n = 0.90 \times 84.0 = 75.6$ kip-in.

5. Moment of Inertia for Deflection Determination - Negative Bending

$\phi_bM_n = 1.2M_{DL} + 1.6M_{LL}$
$= [\ 1.2(M_{DL}/M_{LL}) + 1.6]\ M_{LL}$
$= [\ 1.2(1/5) + 1.6]\ M_{LL}$
$= 1.84M_{LL}$

$M_{LL} = \phi_bM_n/1.84 = 75.60/1.84 = 41.09$ kip-in.

$M_s = M_{DL} + M_{LL} = (1/5+1)M_{LL} = 1.2(41.09) = 49.31$ kip-in.

Computation of I_{eff}, first approximation:

* Assume a compressive stress of f = 27 ksi in the bottom fiber of the section.
* Since the web was fully effective at a higher stress gradient, it will be fully effective at this stress level.

Element 1:

$w/t = 16.67$

$k = 0.43$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)
$= (1.052/\sqrt{0.43})(16.67)\sqrt{27/29500} = 0.809 > 0.673$

$\rho = (1-0.22/\lambda)/\lambda$ (Eq. B2.1-3)
$= (1-0.22/0.809)/0.809 = 0.900$

$b = \rho w$ (Eq. B2.1-2)
$= 0.900 \times 1.000$
$= 0.900$ in.

Element 5:

$w/t = 33.33$

$k \quad = 4$

$\lambda \quad = (1.052/\sqrt{k}(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$\quad = (1.052/\sqrt{4})(33.33)\sqrt{27/29500} = 0.530 > 0.673$

$b_d \quad = w$ (Eq. B2.1-5)

$\quad = 2.000$ (Fully effective)

Element 8:

$w/t \quad = 33.33$

$S \quad = 1.28\sqrt{E/f}$ (Eq. B4-1)

$\quad = 1.28\sqrt{29500/27} = 42.31$

For $S/3 < w/t < S$,

$I_a \quad = t^4\, 399\{[(w/t)/S] - 0.33\}^3$ (Eq. B4.2-6)

$\quad = (0.060)^4(399)[(33.33/42.31) - 0.33]^3$

$\quad = 0.000496 \text{ in.}^4$

$I_s \quad = 0.00471 \text{ in.}^4$ (calculated previously)

$I_s/I_a \quad = 0.00471/0.000496 = 9.5 > 1$

$D \quad = 1.144$ in. (calculated previously)

$D/w \quad = 0.572$ (calculated previously)

For $0.25 < D/w < 0.80$

$k \quad = [4.82 - 5(D/w)](I_s/I_a)^{1/2} + 0.43 \leq 5.25 - 5(D/w)$ (Eq. B4.2-9)

Since $I_s/I_a < 1$

$k \quad = 5.25 - 5(D/w) = 5.25 - 5(0.572) = 2.390$

$\lambda \quad = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$\quad = (1.052/\sqrt{2.390})(33.33)\sqrt{27/29500} = 0.680 > 0.673$

$\rho \quad = (1 - 0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$\quad = (1 - 0.22/0.680)/0.680 = 0.994$

$b \quad = \rho w$ (Eq. B2.1-2)

$\quad = 0.994(2.000) = 1.988$ in.

Element 7:

$I_s/I_a > 1$

$d = 1.000$ in.

Assume max. stress in element, $f = 27$ ksi although it will be actually less.

$k \quad = 0.43$

$w/t \quad = 16.67$

$\lambda \quad = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$\quad = (1.052/\sqrt{0.43})(16.67)\sqrt{27/29500} = 0.809 > 0.673$

$\rho \quad = (1 - 0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1-0.22/0.809)/0.809 = 0.900$

$b \quad = \rho w$ (Eq. B2.1-2)

$= 0.900(1.000) = 0.900$ in.

$d'_s \quad = 0.900$ in.

$d_s \quad = d'_s(I_s/I_a) \leq d'_s$ (Eq. B4.2-11)

Since $I_s/I_a > 1$

$d_s \quad = d'_s = 0.900$ in.

$I'_1 \quad = (d_s)^3 \sin^2\theta/12 = (0.900)^3(\sin 75.96°)2/12 = 0.057$ in.3

Distance of centroid of reduced section from top fiber,
$y = 4-0.147-(0.900/2)\cos 14.04° = 3.416$ in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly ($in.^2$)	Ly^2 ($in.^3$)	I'_1 About Own Axis ($in.^3$)
1	0.900	3.970	3.573	14.185	—
2	5 x 0.206 = 1.030	3.928	4.046	15.892	—
3	4 x 3.819 =15.276	2.000	30.552	61.104	17.472
4	2 x 2.000 = 4.000	0.030	0.120	0.004	—
5	2.000	3.970	7.940	31.522	—
6	4 x 0.206 = 0.824	0.072	0.059	0.004	—
7	0.900	3.416	3.074	10.50	0.057
8	1.988	3.970	7.892	31.331	—
Sum	26.918		57.256	164.544	17.529

Distance of neutral axis from top fiber,

$y_{cg} \quad = Ly/L = 57.256/26.918 = 2.127$ in.

$I'_{eff} \quad = Ly^2 + I'_1 - Ly^2_{cg}$

$= 164.544 + 17.529 - 26.918(2.127)^2 = 60.29\ in.^3$

Actual $I_{eff} = tI'_{eff} = (0.060)(60.29) = 3.62\ in.^4$

$S_{eff} \quad = I_{eff}/(d-y_{cg}) = 3.62/(4-2.127) = 1.93\ in.^3$

$M \quad = (1.93)(27) = 52.11\ ksi > M_s = 49.31$ ksi N.G.

Computation of I_{eff}, second approximation:

* Assume a compressive stress in the bottom fiber of the section using extrapolation.

(1) f = 45 ksi, M = 84.00 kip-in.

(2) f = 27 ksi, M = 52.11 kip-in.

(3) f = ? M = 49.31 kip-in.

$(f-27)/(49.31-52.11) = (27-45)/(52.11-84.00)$

$f- = -1.58 \quad f = 27 - 1.58$

$f = 25.42$ ksi

Element 1:

$w/t = 16.67$

$k \quad 0.43$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{0.43})(16.67)\sqrt{25.42/29500} = 0.785 > .673$

$\rho = (1-0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1-0.22/0.785)/0.785 = 0.917$

$b = \rho w$ (Eq. B2.1-2)

$= 0.917 \times 1 = 0.917$ in.

Element 5:

Fully effective at f = 27 ksi

It will also be fully effective at f = 25.42 ksi

b = 2.000 in.

Element 8:

$w/t = 33.33$

$S = 1.28\sqrt{f/E}$ (Eq. B4-1)

$= 1.28\sqrt{29500/25.42} = 43.60$

For $S/3 < w/t < S$,

$I_s/I_a > 1$ by observation

$D/w = 0.572$

Since $I_s/I_a > 1$

$k = 2.390$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{0.43})(33.33)\sqrt{25.42/29500} = 0.666 < 0.673$

$b = w$ (Eq. B2.1-1)

$= 2.000$ in. (Fully effective)

Element 7:

$I_s/I_a > 1$

d = 1.000 in.

Assume max. stress in element, f = 25.42 ksi although it will be actually less.

$k = 0.43$

$w/t = 16.67$

$\lambda \quad = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{0.43})(16.67)\sqrt{25.42/29500} = 0.785 > 0.673$

$\rho \quad = (1\text{-}0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1\text{-}0.22/0.785)/0.785 = 0.917$

$b \quad = \rho w$ (Eq. B2.1-2)

$= 0.917(1.000) = 0.917$ in.

$d'_s \quad = 0.917$ in.

Since $I_s/I_a > 1$

$d_s \quad = d'_s = 0.917$ in.

$I'_1 \quad = (d_s)^3\sin^2\theta/12 = (0.917)^3(\sin 75.96°)2/12 = 0.060$ in.^{3}

Distance of centroid of reduced section from top fiber,

$y \quad = 4\text{-}0.147\text{-}(0.917/2)\cos 14.04° = 3.408$ in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
1	0.917	3.970	3.640	14.453	—
2	5 x 0.206 = 1.030	3.928	4.046	15.892	—
3	4 x 3.819 = 15.276	2.000	30.552	61.104	17.472
4	2 x 2.000 = 4.000	0.030	0.120	0.004	—
5	2.000	3.970	7.940	31.522	—
6	4 x 0.206 = 0.824	0.072	0.059	0.004	—
7	0.917	3.408	3.125	10.650	0.060
8	2.000	3.970	7.940	31.522	—
Sum	26.964		57.422	165.151	17.532

Distance of neutral axis from top fiber,

y_{cg} Ly/L = 57.422/26.964 = 2.130 in.

$I'_{eff} \quad = L_y2 + I'_1 - L_y2_{cg}$

$= 165.151 + 17.532 - 26.964(2.130)^2 = 60.35$ in.^{3}

Actual $I_{eff} = tI'_{eff} = (0.060)(60.35) = 3.62$ in.^{4}

$S_{eff} \quad = I_{eff}/(d\text{-}y_{cg}) = 3.62/(4\text{-}2.130) = 1.94$ in.^{3}

$M \quad = (1.94)(25.42) = 49.31$ ksi $= M_t$ OK

Note : A slight adjustment could be made for element 7 since the actual maximum stress is less than f = 25.42 ksi, but the net effect will be negligible.

6. Summary Positive Bending : $\phi_b M_n$ = 79.8 kip-in.

   I_{eff} = 3.64 in.^{4}

   Negative Bending : $\phi_b M_n$ = 75.6 kip-in.

   I_{eff} = 3.62 in.^{4}

7. Compute Factored Uniform Load for a continuous deck over three equal spans, the maximum bending moment is negative and occurs over the interior supports. It is given by:

   $M_u = 0.100 w_u L^2$

   Therefore, the maximum factored uniform load is

   $w_u = M_u/0.100L^2 = 75.6/0.100(10' \text{x} 12''/1)^2 = 0.0525$ kip/in.

   w_u = 0.630 kip/ft

   The maximum deflection occurs at a distance of 0.446L from the exterior supports. It is given by:

   $\Delta = 0.0069wL^4/EI$

   This deflection is limited to $\Delta = L/240$ for live load. Therefore,the maximum live load which will satisfy the deflection requirement is)

   $w_{LL} = EI/[240(0.0069)L^3] = 29500(3.64)/[240(0.0069)(10\text{x}12)^3]$

   = 0.0375 kip/in.

   w_{LL} = 0.450 kip/ft

   $w_u = 1.2DLL + 1.6\, w_{LL}$

   $= [1.2(w_{DL}/w_{LL}) + 1.6]\, w_{LL}$

   $= [1.2(1/5) + 1.6]\, w_{LL}$

   $= 1.84 w_{LL} = 1.84(0.450) = 0.828$ kip/ft > 0.630 kip/ft

   Therefore, design flexural strength governs.

   Factored Uniform Load = 0.630 kip/ft.

8. Check Shear Strength (Section C3.2)

   k_v = 5.34, unreinforced web

   $\sqrt{Ek_v/F_y} = \sqrt{29500(5.34/50} = 56.13$

   $1.415\sqrt{Ek_v/F_y} = 1.415\,(56.13) = 79.42$

   $h/t = 3.819/0.060 = 63.65$

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

   $\theta_v = 0.90$

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

   $= 0.64(0.06)^2\sqrt{5.34(50)(29500)} = 6.47$ kips (per web)

   Total V_n for section:

   $V_n = 4(6.47) = 25.88$ kips

   $\phi_v V_n = 0.90(25.88) = 23.29$ kips

   Vertical component $(\phi_v V_n)_v = 23.29\cos 14.04° = 22.59$ kips

The maximum required shear strength is given by

$V_u \quad = 0.600\ w_uL$

$\quad = (0.600)(0.630)(10) = 3.78 \text{ kips} < (\phi_v V_n)_v = 22.59 \text{ kips OK}$

9. Check Strength for Combined Bending and Shear (Section C3.3) At the interior supports there is a combination of web bending and web shear:

$\phi_b M_n x_o = 75.6 \text{ kip-in.} \qquad M_u = 0.100 w_u L^2$

$(\phi_v V_n)_v = 22.59 \text{ kips} \qquad V_u = 0.600 w_u L$

For unreinforced webs.

$(M_u/\phi_b M_n x_o)^2 + [\ V_u/(\phi_v Vn\ )_v]^2 \leq 1.0$ (Eq. C3.3-1)

Solve for w_u:

$\{[\ 0.100 w_u\ (10\text{x}12)^2]\ /75.6\}^2 + \{[\ 0.600 w_u\ (10\text{x}12)]\ /22.59\}^2 = 1.0$

$362.81 w_u 2 + 10.16 w_u 2 = 1.0$

$372.97 w_u 2 = 1.0$

$w_u = 0.0518 \text{ kip/in.}$

$= 0.621 \text{ kip/ft.}$

Factored Uniform Load = 0.621 kip/ft.

10. Check Web Crippling Strength (Section C3.4)

$h \quad = 3.819 \text{ in.}$

$t \quad = 0.060 \text{ in.}$

$h/t \quad = 3.819/0.06 = 63.65 < 200 \text{ OK}$

$R \quad = 1/8 \text{ in.}$

$R/t \quad = 0.125/0.06 = 2.083 < 7 \text{ OK}$

Let N =6 in.

$N/t \quad = 6/0.06 = 100 < 210 \text{ OK}$

$N/h \quad = 6/3.819 = 1.57 < 3.5 \text{ O K}$

Table C3.4-1 applies

For end reactions: (Eq. C3.4-2)

For interior reaction: (Eq. C3.4-4)

$k \quad = F_y/33 = 50/33 = 1.515$ (Eq. C3.4-21)

$C_1 \quad = (1.22-0.22k)$ (Eq. C3.4-10)

$\quad = [\ 1.22-0.22(1.515)] = 0.887$

$C_2 \quad = (1.06-0.06R/t)$ (Eq. C3.4-11)

$\quad = [\ 1.06-0.06(2.083)] = 0.935 < 1.0 \text{ OK}$

$C_3 \quad = (1.33-0.33k)$ (Eq. C3.4-12)

$\quad = [\ 1.33-0.33(1.515)] = 0.830$

$C_4 \quad = (1.15-0.15R/t) \leq 1.0$ but not less than 0.50 (Eq. C3.4-13)

$(1.15\text{-}0.15R/t) = [1.15\text{-}0.15(2.083)] = 0.838 \leq 1.0$ OK

$C_4 = 0.838$

$\theta = 75.96°$

$C_\theta = 0.7+0.3(\theta/90)^2$ (Eq. C3.4-20)

$= 0.7+0.3(75.96/90)^2 = 0.914$

For end reaction:

$P_n = t^2kC_3C_4C_\theta[217\text{-}0.28(h/t)][1+0.01(N/t)]$ (Eq. C3.4-2)

$= (0.06)^2(1.515)(0.830)(0.838)(0.914)[217\text{-}0.28(63.65)]$

$\times[1+0.01(100)] = 1.38$ kips/web

Total P_n for section:

$P_n = (4 \text{ webs})(1.38 \text{ k/web}) = 5.52$ kips

$\theta_w = 0.75$

$\theta_w P_n = 0.75(5.52) = 4.14$ kips

End reaction is given by

$R = 0.400w_uL$

$= (0.400)(0.621)(10) = 2.48 \text{ kips} < \theta_w P_n\ 4.14$ kips OK

For interior reaction:

$P_n = t^2kC_1C_2C_\theta[538\text{-}0.74(h/t)][1+0.007(N/t)]$ (Eq. C3.4-4)

$= (0.06)^2(1.515)(0.887)(0.935)(0.914)[538\text{-}0.74(63.65)]$

$\times[1+0.007(100)] = 3.45$ kips/web

Total P_n for section:

$P_n = (4 \text{ webs})(3.45 \text{ k/web}) = 13.80$ kips

$\theta_w = 0.75$

$\theta_w P_n = 0.75(13.80) = 10.35$ kips

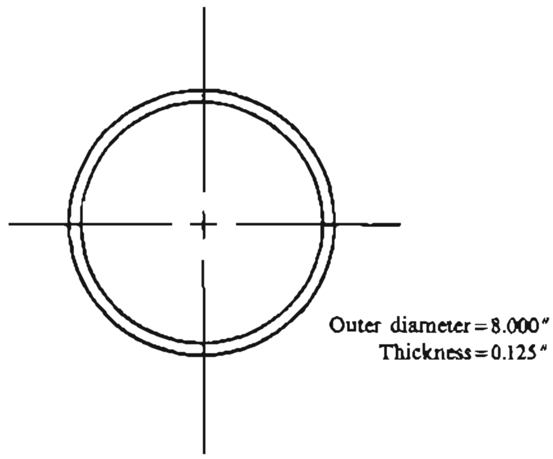
Interior reaction is given by

$R = 1.10w_uL$

$= (1.10)(0.621)(10) = 6.83 \text{ kips} < \theta_w P_n = 10.35$ kips OK

EXAMPLE NO. 9.

CYLINDRICAL TUBULAR SECTION



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: Shown in sketch above.

Required:

Design flexural strength, $\phi_b M_n$.

Solution:

Ratio of outside diameter to wall thickness,

$D/t = 8.000/0.125 = 64.00$

$D/t < 0.441E/F_y = 0.441(29500/50) = 260.2$ OK

$S_f = \pi[(O.D.)^4 - (I.D.)^4]/32(O.D.)$

$= \pi[(8)^4 - (7.75)^4]/32(8)$

$= 5.995$ in.^{3}

$0.070E/F_y = 0.070(29500/50) = 41.30$

$0.319E/F_y = 0.319(29500/50) = 188.2$

For $0.070E/F_y < D/t < 0.319E/F_y$

$M_n = [0.97+0.02(E/F_y)/(D/t)]F_y S_f$ (Eq. C6.1-2)

$= [0.97+0.02(29500/50)/64.00](50)(5.995)$

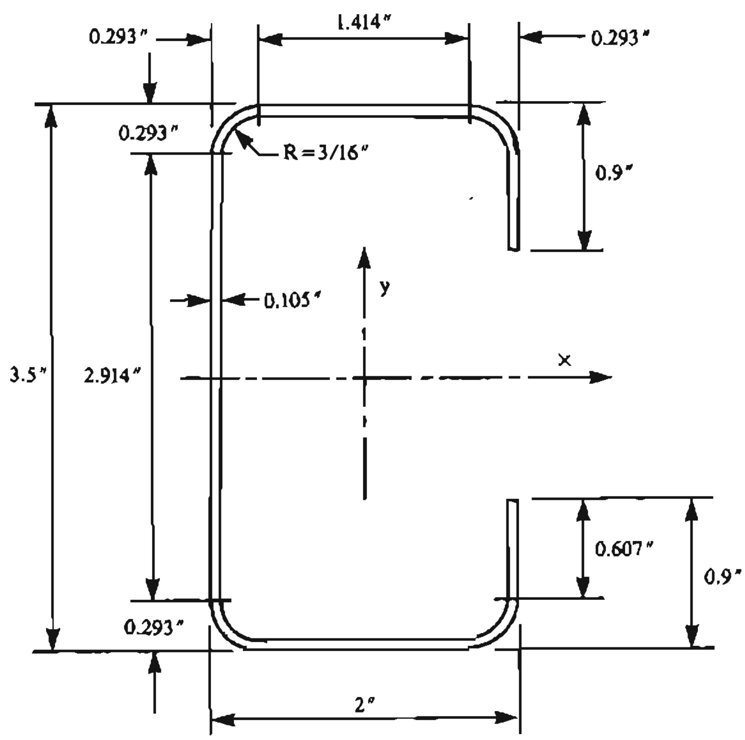
$= 346.02$ kip-in.

$\phi_b = 0.95$

$\phi_b M_n = 0.95 \times 346.02 = 328.72$ kip-in.

EXAMPLE NO. 10.

C-SECTION



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: 3.5 x 2 x 0.105 channel with stiffened flanges.
3. $K_xL_x = K_yL_y = K_tL_t = 6$ ft.

Required :

Design axial strength, ϕ_cP_n.

Solution:

1. Basic parameters:

   r $= R+t/2 = 3/16+0.105/2 = 0.240$ in.

   From the sketch and Section 1.2.2 of Part III of the Manual, a = 2.914 in., b = 1.414 in., c = 0.607 in., α=1.00 (Since the section has lips)

$\bar{a}$ $= A'-t = 3.5-0.105 = 3.395$ in.

$\bar{b}$ $= B'-[t/2+\alpha t/2] = B'-t = 2-0.105 = 1.895$ in.

$\bar{c}$ $= \alpha[C'-t/2] = C'-t/2 = 0.9-0.105/2 = 0.848$ in.

u $=1.57r = 1.57 \times 0.240 = 0.377$ in.

2. Area:

A $= t[a+2b+2u+\alpha(2c+2u)] = t[a+2b+2c+4u]$

$= 0.105[2.914+2x1.414+2x0.607+4x0.377] = 0.889$ in.^{2}

3. Moment of inertia about x-axis:

I_x $= 2t\{0.0417a^3+b(a/2+r)^2+u(a/2+0.637r)^2+0.149r^3$

$+\alpha[0.0833c^3+(c/4)(a-c)^2+u(a/2+0.637r)^2+0.149r^3]\}$

$= 2t[0.0417a^3+b(a/2+r)^2+2u(a/2+0.637r)^2+0.298r^3$

$+0.0833c^3+(c/4)(a-c)^2]$

$= 2x0.105[0.0417(2.914)^3+1.414(2.914/2+0.240)^2$

$+2x0.377(2.914/2+0.637x0.240)^2+0.2\,98(0.240)^3$

$+0.0833(0.607)^3+(0.607/4)(2.914-0.607)^2]$

$= 1.657$ in.^{4}

4. Distance from centroid of section to centerline of web:

$\tilde{x}$ $= (2t/A)\{b(b/2+r)+u(0.363r)+\alpha[u(b+1.637r)+c(b+2r)]\}$

$= [(2x0.105)/0.889]\{1.414(1.414/2+0.240)$

$+0.377(0.363 \times 0.240)+0.377(1.414+1.637x0.240)+0.607(1.414+2x0.240)\}$

$= 0.757$ in.

5. Moment of inertia about y-axis:

I_y $= 2t\{b(b/2+r)^2+0.0833b^3+0.356r^3+\alpha[c(b+2r)^2$

$+u(b+1.637r)^2+0.149r^3\}\}-A(\bar{x})^2$

$= 2x0.105\{1.414(1.414/2+0.240)^2+0.0833(1.414)^3$

$+0.356(0.240)^3+0.607(1.414+2x0.240)^2$

$+0.377(1.414+1.637x0.240)^2+0.149(0.240)^3\}-0.889(0.757)^2$

$= 0.524$ in.^{4}

6. Distance from shear center to centerline of web:

m $= (\bar{b}t/12I_x)[6\bar{c}(\bar{a})^2+3\bar{b}(\bar{a})^2-8(\bar{c})^3]$

$= [(1.895x0.105)/(12x1.657)][6x0.848(3.395)^2$

$+3x1.895(3.395)^2-8(0.848)^3]$

$= 1.194$ in.

7. Distance from centroid to shear center:

x_o $= -(\bar{x}+m) = -(0.757+1.194)$

$= -1.951$ in.

8. St. Venant torsion constant:

$J = (t^3/3)[a+2b+2u+\alpha(2c+2u)]$

$= [(0.105)^3/3][2.914+2x1.414+4x0.377+2x0.607]$

$= 0.003266$ in.^{4}

9. Warping Constant:

$C_w = (t^2/A)\{[\bar{x}A(\bar{a})^2/t][(\bar{b})^2/3+m^2-m\bar{b}]+(A/3t)[(m)^2(\bar{a})^3$

$+(\bar{b})^2(\bar{c})^2(2\bar{c}+3\bar{a})]-(I_x m^2/t)(2\bar{a}+4\bar{c})+[m(\bar{c})^2/3][8(\bar{b})^2(\bar{c})]$

$+2m(2\bar{c}(\bar{c}-\bar{a})+\bar{b}(2\bar{c}-3\bar{a}))]+[(\bar{b})^2(\bar{a})^2/6][(3\bar{c}+\bar{b})(4\bar{c}+\bar{a})-6(\bar{c})^2]$

$-[m^2(\bar{a})^4]/4]\}$

$= [(0.105)^2/0.889]\{[0.757x0.889x(3.395)^2/0.105][(1.895)^2/3$

$+(1.194)^2 - 1.194x1.895]+0.889/(3x0.105)[(1.194)^2(3.395)^3$

$+(1.895)^2(0.848)^2(2x0.848+3x3.395)]$

$-[1.657x(1.194)^2/0.105](2x3.395+4x0.848)$

$+[1.194(0.848)^2/3][8(1.895)^2(0.848)$

$+2x1.194(2x0.848(0.848-3.395)+1.895(2x0.848-3x3.395))]$

$+[(1.895)^2(3.395)^2/6][(3x0.848+1.895)(4x0.848+3.395)$

$-6(0.848)^2]-[(1.194)^2(3.395)^4/4]\}$

$= 2.050$ in.^{6}

10. Radii of gyration:

$r_x = \sqrt{(I_x/A)} = \sqrt{1.657/0.889)} = 1.365$ in

$r_y = \sqrt{(I_y/A)} = \sqrt{0.524/0.889)} = 0.768$ in

$(K_yL_y)/r_y = (6x12)/0.768 = 93.75 < 200$

$r_o = \sqrt{r_x^2 + r_y^2 + x_o^2} = \sqrt{(1.365)^2 + (0.768)^2 + (-1.951)^2}$

$= 2.502$ in.^{2}

11. Torsional-flexural constant:

$\beta = 1-(x_o/r_o)^2$ (Eq. C4.2-3)

$= 1-(-1.951/2.502)^2 = 0.392$

12. Determination of F_e:

For this singly symmetric section (x-axis is the axis of symmetry), F_e shall be taken as the smaller of either (Eq. C4.1-1) or (Eq. C4.2-1):

$(F_e)_1 = (\pi^2E)/(K_yL_y/r_y)^2$ (Eq. C4.1-1)

$= (\pi^2x29500)/(6x12/0.768)^2 = 33.13$ ksi

$\sigma_{ex} = (\pi^2E)/(K_xL_x/r_x)^2$ (Eq. C3.1.2-7)

$= (\pi^2 x29500)/(6x12/1.365)^2 = 104.65$ ksi

$\sigma_t \quad = 1/(Ar_o^2)[\,GJ+(\,\pi^2\, EC_w\,)/(K_tL_t\,)^2]$ (Eq. C3.1.2-9)

$= 1/(0.889x2.502^2)[11300x0.003266+(\pi^2 x29500x2.050)/(6x12)^2]$

$= 27.32$ ksi.

$(F_e)_2 \quad = (1/2\beta)[\,(\sigma_{ex}+\sigma_t)-\sqrt{(\sigma_{ex}+\sigma_t)^2-4\beta\sigma_{ex}\sigma_t}\,]$ (Eq. C4.2-1)

$= 1/(2x0.392)[\,(104.65+27.32)$

$-\sqrt{(104.65+27.32)^2-4x0.392x104.65x27.32}\,]$

$= 23.27$ ksi

$F_e \quad = 23.27$ ksi

13. Determination of F_n:

$F_y/2 \quad = 50/2 = 25.00$ ksi

For $F_e < F_y/2$

$F_n \quad = F_e$ (Eq. C4-3)

$= 23.27$ ksi.

14. Determination of A_e:

Flanges:

$d = 0.607$ in.

$I_s= d^3t/12 = (0.607)^3(0.105)/12$

$= 0.001957$ in.^{4}

$D \quad = 0.9$ in.

$w \quad = 1.414$ in.

$D/w \quad = 0.9/1.414 = 0.636 < 0.80$

$S \quad = 1.28\,\sqrt{E/f},\ f= F_n$ (Eq. B4-1)

$= 1.28\,\sqrt{29500/23.27} = 45.57$

$w/t \quad = 1.414/0.105 = 13.47 < S/3 = 15.19$ (Eq. B4.2-1)

$I_a \quad = 0$ (no edge stiffener needed) (Eq. B4.2-2)

$b \quad = w$ (Eq. B4.2-3)

$= 1.414$ in. (flanges fully effective)

$w/t \quad = 13.47 < 90$ (Section B1.1-(a)-(1))

Web:

$w \quad = 2.914$ in., $k = 4.00$

$\lambda \quad = (1.052/\sqrt{k}\,)(w/t)\,\sqrt{f/E},\ f = F_n$ (Eq. B2.1-4)

$= (1.052/\sqrt{4}\,)(2.914/0.105)\,\sqrt{23.27/29500}$

$= 0.410 < 0.673$

$b \quad = w$ (Eq. B2.1-1)

= 2.914 in. (web fully effective)

w/t = 2.914/0.105 = 27.75 < 500 (Section B1.1-(a)-(2))

Lips:

d = 0.607 in.

k = 0.43 (unstiffened compression element)

d_s = d'_s (Eq. B4.2-4)

λ = $(1.052/\sqrt{0.43})(0.607/0.105)\sqrt{23.27/29500}$

= 0.260 < 0.673

d'_s = d = 0.607 in., d_s = 0.607 in.

d/t = 5.78 < 14 (Section B4 of the Commentary)

Since flanges, web, and lips are fully effective

A_e = A = 0.889 in.^{2}

15. Determination of $\phi_c P_n$:

P_n = $A_e F_n$ (Eq. C4-1)

= 0.889 x 23.27

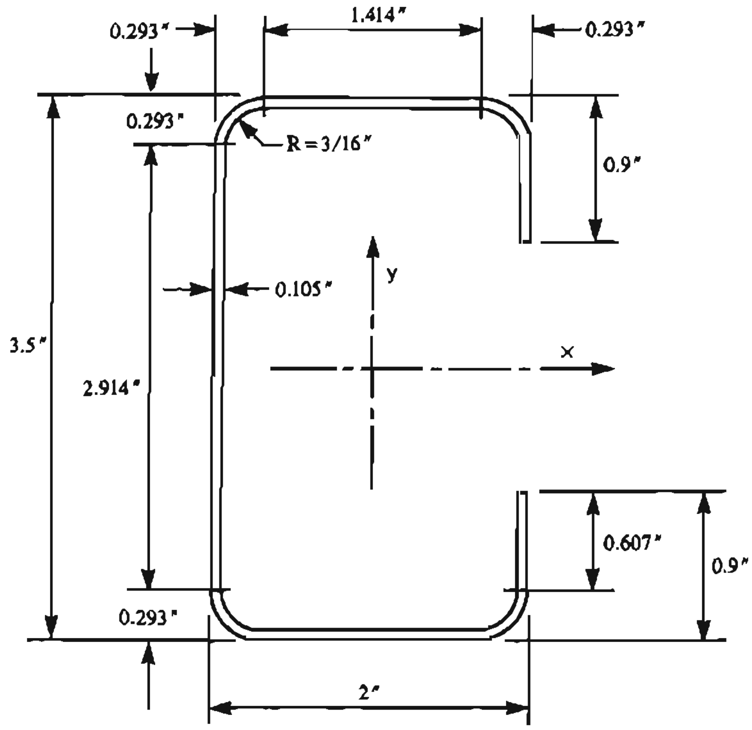
= 20.69 kips

ϕ_c = 0.85

$\phi_c P_n$ = 0.85 x 20.69= 17.59 kips

EXAMPLE NO. 11

C-SECTION WITH HOLES



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: 3.5 x 2 x 0.105 channel with stiffened flanges.
3. $K_xL_x = K_yL_y = K_tL_t = 6$ ft.
4. Web is perforated with holes for bolts of 1/2-in. diameter (in standard hole) at 4 in. spacing along the height of the column.

Required:

Design axial strength, ϕ_cP_n

Solution:

1. Basic parameters:

   $r = R+t/2 = 3/16+0.105/2 = 0.240$ in.

   From the sketch and Section 1.2.2 of Part III of the Manual, a = 2.914 in., b = 1.414 in., c = 0.607 in.,

α = 1.00 (Since the section has lips)

$\bar{a}$ = A′-t = 3.5-0.105 = 3.395 in.

$\bar{b}$ = B′-t = 2-0.105 = 1.895 in.

$\bar{c}$ = C′-t/2 = 0.9-0.105/2 = 0.848 in.

u = 1.57r = 1.57 x 0.240 = 0.377 in.

2. Area:

A = t[a+2b+2c+4u]

0.105[2.914+2x1.414+2x0.607+4x0.377]

= 0.889 in.^{2}

3. Moment of inertia about x-axis:

I_x = 2t[0.0417a^3 +b(a/2+r)^2 +2u(a/2+0.637r)^2 +0.298r^3

+0.0833c^3 +(c/4)(a-c)^2]

= 2x0.105[0.0417(2.914)^3 +1.414(2.914/2+0.240)^2

+2x0.377(2.914/2+0.637x0.240)^2+0.298(0.240)^3

+0.0833(0.607)^3 +(0.607/4)(2.914-0.607)^2]

= 1.657 in.^{4}

4. Distance from centroid of section to centerline of web:

$\bar{x}$ = (2t/A)[b(b/2+r)+u(0.363r)+u(b+1.637r)+c(b+2r)]

= (2x0.105/0.889)[1.414(1.414/2+0.240+0.377(0.363x0.240)

+0.377(1.414+1.637x0.240)+0.607(1.414+2x0.240)]

= 0.757 in.

5. Moment of inertia about y-axis:

I_y = 2t[b(b/2+r)^2 +0.0833b^3 +0.505r^3 +c(b+2r)^2

+u(b+1 .637r)^2] -A($\bar{x}$)^2

= 2x0.105[1.414(1.414/2+0.240)^2 +0.0833(1.414)^3

+0.505(0.240)^3+0.607(1.414+2x0.240)^2

+0.377(1.414+1.637x0.240)^2] -0.889(0.757)^2

= 0.524 in.^{4}

6. Distance from shear center to centerline of web:

m = ($\bar{b}$t/12I_x)[6 $\bar{c}$($\bar{a}$)^2 +3 $\bar{b}$($\bar{a}$)^2 -8($\bar{c}$)^3]

= [(1.895x0.105)/(12x1.657)][6x0.848(3.395)^2

+3x1.895(3.395)^2 -8(0.848)^3]

= 1.194 in.

7. Distance from centroid to shear center:

x_o = -($\bar{x}$+m) = -(0.757+1.194)

$= -1.951$ in.

8. St. Venant torsion constant:

$J = (t^3/3)[a+2b+2c+4u]$

$= [(0.105)^3/3][2.914+2\text{x}1.414+2\text{x}0.607+4\text{x}0.377]$

$= 0.003266 \text{ in.}^4$

9. Warping Constant:

$C_w = (t^2/A)\{[\bar{x}A(\bar{a})^2/t][(\bar{b})^2/3+m^2-m\bar{b}] + (A/3t)[(m)^2(\bar{a})^3$
$+(\bar{b})^2(\bar{c})^2(2\bar{c}+3\bar{a})] - (I_x m^2/t)(2\bar{a}+4\bar{c})+[m(\bar{c})^2/3][8(\bar{b})^2(\bar{c})$
$+2m(2\bar{c}(\bar{c}-\bar{a})+\bar{b}(2\bar{c}-3\bar{a}))] +[(\bar{b})^2(\bar{a})^2/6[(3\bar{c}+\bar{b})(4\bar{c}+\bar{a})-6(\bar{c})^2]$
$-[m^2(\bar{a})^4/4]\}$

$= [(0.105)^2/0.889]\{[0.757\text{x}0.889\text{x}(3.395)^2/0.105][(1.895)^2/3$
$+(1.194)^2-1.194\text{x}1.895]+0.889/(3\text{x}0.105)[(1.194)^2(3.395)^3$
$+(1.895)^2(0.848)^2(2\text{x}0.848+3\text{x}3.395)]$
$-[1.657\text{x}(1.194)^2/0.105](2\text{x}3.395+4\text{x}0.848)$
$+[1.194(0.848)^2/3][8(1.895)^2(0.848)$
$+2\text{x}1.194(2\text{x}0.848(0.848-3.395)+1.895(2\text{x}0.848-3\text{x}3.395))]$
$+[(1.895)^2(3.395)^2/6][(3\text{x}0.848+1.895)(4\text{x}0.848+3.395)$
$-6(0.848)^2]-[(1.194)^2(3.395)^4/4]\}$

$= 2.050 \text{ in.}^6$

10. Radii of gyration:

$r_x = \sqrt{(I_x/A)} = \sqrt{(1.657/0.889)} = 1.365$ in.

$r_y = \sqrt{(I_y/A)} = \sqrt{(0.524/0.889)} = 0.768$ in.

$(K_yL_y)/r_y = (6\text{x}12)/0.768 = 93.75 < 200$

$r_o = \sqrt{r_x^2 + r_y^2 + x_o^2} = \sqrt{(1.365)^2 + (0.768)^2 + (-1.951)^2}$

$= 2.502 \text{ in.}^2$

11. Torsional-flexural constant:

$\beta = 1-(x_o/r_o)^2$ (Eq. C4.2-3)

$= 1-(-1.951/2.502)^2$

$= 0.392$

12. Determination of F_e:

For this singly symmetric section (x-axis is the axis of symmetry), F_e shall be taken as the smaller of either (Eq. C4.1-1) or (Eq. C4.2-1):

$(F_e)_1 = (\pi^2E)/(K_yL_y/r_y)^2$ (Eq. C4.1-1)

$= (\pi^2\text{x}29500)/(93.75)^2 = 33.13$ ksi

$\sigma_{ex} = (\pi^2 E)/(K_x L_x/r_x)^2$ (Eq. C3.1.2-7)

$= (\pi^2 x29500)/(6x12/1.365)^2 = 104.65$ ksi

$\sigma_t = 1/(Ar_o^2)[\,GJ+(\pi^2 EC_w)/(K_t L_t)^2]$ (Eq. C3.1.2-9)

$= 1/(0.889x2.502^2)[11300x0.003266+(\pi^2 x29500x2.050)/(6x12)^2]$

$= 27.32$ ksi.

$(F_e)_2 = (1/2\beta)[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\,]$ (Eq. C4.2-1)

$= 1/(2x0.392)[\,(104.65+27.32)$

$-\sqrt{(104.65+27.32)^2 - 4x0.392x104.65x27.32}\,]$

$= 23.27$ ksi

$F_e = 23.27$ ksi

13. Determination of F_n:

$F_y/2 = 50/2 = 25.00$ ksi

For $F_e < F_y/2$

$F_n = F_e$ (Eq. C4-3)

$= 23.27$ ksi.

14. Determination of A_e:

Flanges:

$d = 0.607$ in.

$I_s = d^3t/12 = (0.607)^3(0.105)/12 = 0.001957$ in.^{4}

$D = 0.9$ in.

$w = 1.414$ in.

$D/w = 0.9/1.414 = 0.636 < 0.80$

$S = 1.28\sqrt{E/f},\ f = F_n$ (Eq. B4-1)

$= 1.28\sqrt{29500/23.27} = 45.57$

$w/t = 1.414/0.105 = 13.47 < S/3 = 15.19$ (Eq. B4.2-1)

$I_a = 0$ (no edge stiffener needed) (Eq. B4.2-2)

$b = w$ (Eq. B4.2-3)

$= 1.414$ in. (flanges fully effective)

$w/t = 13.47 < 90$ (Section B1.1-(a)-(1))

Web:

$w = 2.914$ in., $k = 4.00$

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E},\ f = F_n$ (Eq. B2.1-4)

$= (1.052/\sqrt{4})(2.914/0.105)\sqrt{23.27/29500}$

$= 0.410 < 0.673$

$d_h = d+1/16$ (Table E3)

d = diameter of bolt = 0.5 in.

d_h = 0.5+1/16= 0.563 in.

Number of holes in the effective length = 17
(17x0.563)/(6x12) = 0.133 > 0.015 then A_e must be determinedwith holes accounted for. (Section C4-(a))

d_h/w = 0.563/2.914 = 0.193 < 0.50

w/t = 2.914/0.105

= 27.75 < 70

$0.5w$ = 0.5x2.914 = 1.457 in.

$3d_h$ = 3x0.563 = 1.689 in.

Spacing of holes = 4 in. greater than 0.5w and $3d_h$
Effective width, b, shall be determined by using (Eq. B2.2-1)

b = $w-d_h$ (Eq. B2.2-1)

= 2.914-0.563

= 2.351 in.

w/t = 27.75 < 500 (Section B1.1-(a)-(2))

Lips:

d = w

= 0.607 in.

k = 0.43 (unstiffened compression element)

λ = $(1.052/\sqrt{0.43})(0.607/0.105)\sqrt{23.27/29500}$

= 0.260 < 0.673

d'_s = d = 0.607 in.

d_s = d'_s (Eq. B4.2-4)

= 0.607 in. (No reduction in lips area)

w/t = 0.607/0.105 = 5.78 < 14 (Section B4 of the Commentary)

A_e = $A-t(w-b)_{web}$

= 0.889-0.105(2.914-2.351)

= 0.830 in.^{2}

15. Determination of $\phi_c P_n$:

P_n = $A_e F_n$ (Eq. C4-1)

= 0.830 x 23.27

= 19.31 kips

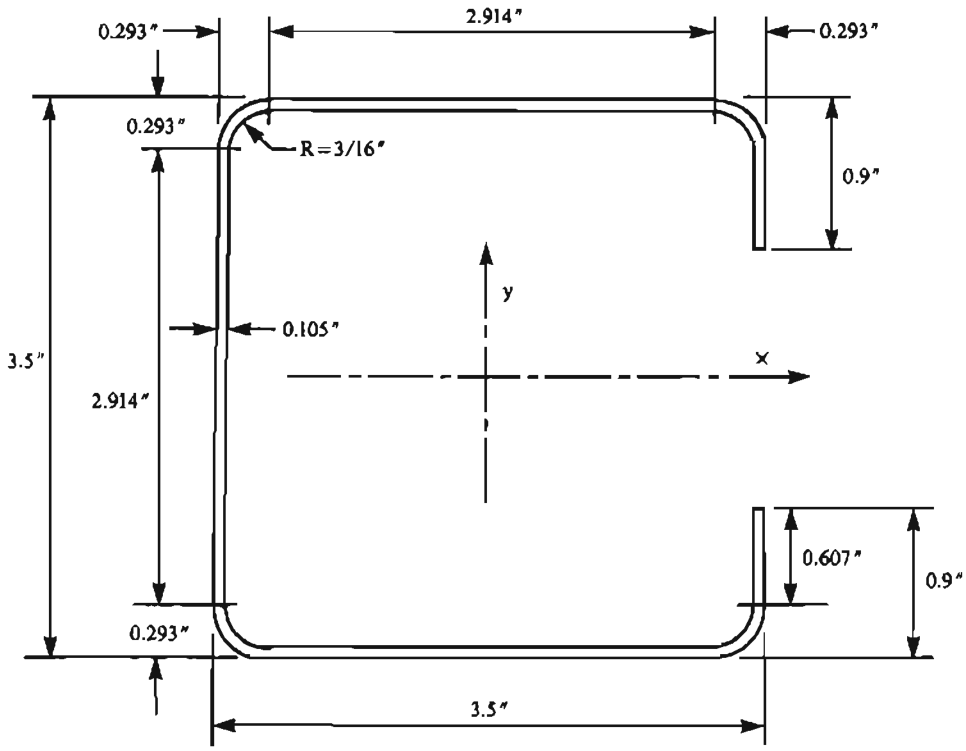
ϕ_c = 0.85

$\phi_c P_n$ = 0.85 x 19.31

= 16.41 kips

EXAMPLE NO. 12

C-SECTION WITH WIDE FLANGE



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: 3.5 x 3.5 x 0.105 channel with stiffened flanges.
3. $K_xL_x = K_yL_y = K_tL_t = 6$ ft.

Required:

Design axial strength, ϕ_cP_n.

Solution:

1. Basic parameters:

   $r = R+t/2 = 3/16+0.105/2 = 0.240$ in.

   From the sketch and Section 1.2.2 of Part III of the Manual, a = 2.914 in., b = 2.914 in., c = 0.607 in.,

   $\alpha = 1.00$ (Since the section has lips)

   $\bar{a} \quad = A'\text{-}t = 3.5\text{-}0.105 = 3.395$ in.

$\bar{b} = B' - t = 3.5 - 0.105 = 3.395$ in.

$\bar{c} = C' - t/2 = 0.9 - 0.105/2 = 0.848$ in.

$u = 1.57r = 1.57 \times 0.240 = 0.377$ in.

2. Area:

$A = t[a+2b+2c+4u]$

$= 0.105[2.914+2x2.914+2x0.607+4x0.377]$

$= 1.204 \text{ in.}^2$

3. Moment of inertia about x-axis:

$I_x = 2t[0.0417a^3 + b(a/2+r)^2 + 2u(a/2+0.637r)^2 + 0.298r^3$

$+0.0833c^3 + (c/4)(a-c)^2]$

$= 2x0.105[0.0417(2.914)^3 + 2.914(2.914/2+0.240)^2$

$+2x0.377(2.914/2+0.637x0.240)^2 + 0.298(0.240)^3$

$+0.0833(0.607)^3 + (0.607/4)(2.914-0.607)^2]$

$= 2.564 \text{ in.}^4$

4. Distance from centroid of section to centerline of web:

$\bar{x} = (2t/A)[b(b/2+r) + u(0.363r) + u(b+1.637r) + c(b+2r)]$

$= [(2x0.105)/1.204][2.914(2.914/2+0.240)+0.377(0.363x0.240)$

$+0.377(2.914+1.637x0.240)+0.607(2.914+2x0.240)]$

$= 1.445$ in.

5. Moment of inertia about y-axis:

$I_y = 2t[b(b/2+r)^2 + 0.0833b^3 + 0.505r^3 + c(b+2r)^2$

$+u(b+1.637r)^2] - A(\bar{x})^2$

$= 2x0.105[2.914(2.914/2+0.240)^2 + 0.0833(2.914)^3$

$+0.505(0.240)^3 + 0.607(2.914+2x0.240)^2$

$+0.377(2.914+1.637x0.240)^2] - 1.204(1.445)^2$

$= 2.017 \text{ in.}^4$

6. Distance from shear center to centerline of web:

$m = (\bar{b}t/12I_x)[6\bar{c}(\bar{a})^2 + 3\bar{b}(\bar{a})^2 - 8(\bar{c})^3]$

$= [(3.395x0.105)/(12x2.564)][6x0.848(3.395)^2$

$+3x3.395(3.395)^2 - 8(0.848)^3]$

$= 1.983$ in.

7. Distance from centroid to shear center:

$x_o = -(\bar{x}+m) = -(1.445+1.983) = -3.428$ in.

8. St. Venant torsion constant:

$J = (t^3/3)[a+2b+2c+4u]$

$= [(0.105)^3/3][2.914+2x2.914+2x0.607+4x0.377]$

$= 0.004424 \text{ in.}^4$

9. Warping Constant:

$C_w = (t^2/A)\{[\bar{x}A(\bar{a})^2/t][(\bar{b})^2/3+m^2-m\bar{b}]+(A/3t)[(m)^2(\bar{a})^3$
$+(\bar{b})^2(\bar{c})^2(2\bar{c}+3\bar{a})-(I_x m^2/t)(2\bar{a}+4\bar{c})+[m(\bar{c})^2/3][8(\bar{b})^2(\bar{c})$
$+2m(2\bar{c}(\bar{c}-\bar{a})+\bar{b}(2\bar{c}-3\bar{a}))]+[(\bar{b})^2(\bar{a})^2/6)][(3\bar{c}+\bar{b})(4\bar{c}+\bar{a})-6(\bar{c})^2]$
$-\{m^2(\bar{a})^4/4]\}$

$= [(0.105)^2/1.204]\{[1.445x1.204x(3.395)^2/0.105][(3.395)^2/3$
$+(1.983)^2-1.983x3.395]+1.204/(3x0.105)[(1.983)^2(3.395)^3$
$+(3.395)^2(0.848)^2(2x0.848+3x3.395)]$
$-[2.564x(1.983)^2/0.105](2x3.395+4x0.848)$
$+[1.983(0.848)^2/3][8(3.395)^2(0.848)$
$+2x1.983(2x0.848(0.848-3.395)+3.395(2x0.848-3x3.395))]$
$+[(3.395)^2(3.395)^2/6][(3x0.848+3.395)(4x0.848+3.395)$
$-6(0.848)^2]-[(1.983)^2(3.395)^4/4]\}$

$= 7.572 \text{ in.}^6$

10. Radii of gyration:

$r_x = \sqrt{(I_x/A} = \sqrt{(2.564/1.204)} = 1.459$ in.

$r_y = \sqrt{(I_y/A)} = \sqrt{(2.017/1.204)} = 1.294$ in.

$(K_yL_y)/r_y = (6x12)/1.294 = 55.64 < 200$

$r_o = \sqrt{r_x^2+r_y^2+x_o^2} = \sqrt{(1.459)^2+(1.294)^2+(-3.428)^2}$

$= 3.944 \text{ in.}^2$

11. Torsional-flexural constant:

$\beta = 1-(x_o/r_o)^2$ (Eq. C4.2-3)

$= 1-(-3.428/3.944)^2$

$= 0.244$

12. Determination of F_e:

For this singly symmetric section (x-axis is the axis of symmetry), F_e shall be taken as the smaller of either (Eq. C4.1-1) or (Eq. C4.2-1):

$(F_e)_1 = (\pi^2E)/(K_yL_y/r_y)^2$ (Eq. C4.1-1)

$= (\pi^2x29500)/(55.64)^2 = 94.05$ ksi

$\sigma_{ex} = (\pi^2E)/(K_xL_x/r_x)^2$ (Eq. C3.1.2-7)

$= (\pi^2x29500)/(6x12/1.459)^2 = 119.55$ ksi

$\sigma_t = 1/(Ar_o^2)[GJ+(\pi^2EC_w)/(K_tL_t)^2]$ (Eq. C3.1.2-9)

$= 1/(1.204x3.944^2)[11300x0.004424+(\pi^2x29500x7.572)/(6x12)^2]$

$= 25.38$ ksi.

$(F_e)_2 = (1/2\beta)[\, (\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\,]$ (Eq. C4.2-1)

$= 1/(2x0.244)[\, (119.55+25.38)$

$-\sqrt{(119.55 + 25.38)^2 - 4x0.244x119.55x25.38}\,]$

$= 21.73$ ksi

$F_e = 21.73$ ksi

13. Determination of F_n:

$F_y/2 = 50/2 = 25.00$ ksi

For $F_e < F_y/2$

$F_n = F_e$ (Eq. C4-3)

$= 21.73$ ksi.

14. Determination of A_e:

Flanges:

$d = 0.607$ in.

$I_s = d^3t/12 = (0.607)^3(0.105)/12$

$= 0.001957$ in.^{4}

$D = 0.9$ in.

$w = 2.914$ in. (for flange)

$D/w = 0.9/2.914 = 0.309 < 0.80$

$S = 1.28\sqrt{E/f},\ f = F_n$ (Eq. B4-1)

$= 1.28\sqrt{29500/21.73} = 47.16,\ S/3 = 15.72$

$w/t = 2.914/0.105 = 27.75$

$S/3 < w/t < S$

$I_a = 399t^4\{[\,(w/t)/S]-0.33\}^3$ (Eq. B4.2-6)

$= 399(0.105)^4[(27.75/47.16)-0.33]^3$

$= 0.000837$ in.$^4 < I_s = 0.001957$ in.^{4}

$C_1 = 2-(I_s/I_a) \geq 1.0$ (Eq. B4.2-8)

$= 2-(0.001957/0.000837) = -0.34 < 1.0$

$C_1 = 1.0$

$C_2 = I_s/I_a \leq 1.0$ (Eq. B4.2-7)

$I_s/I_a = (0.001957/0.000837) = 2.34 > 1.0$

$C_2 = 1.0$

$0.25 < D/w = 0.309 < 0.8$

$k = [4.82-5(D/w)](I_s/I_a)^n + 0.43 \leq 5.25-5(D/w)$ (Eq. B4.2-9)

$n = 1/2$

$[4.82\text{-}5(0.309)](0.001957/0.000837)^{1/2}+0.43 = 5.438$

$5.25\text{-}5(0.309) = 3.705 < 5.438$

k $= 3.705$

λ $= (1.052/\sqrt{k})(w/t)\sqrt{f/E}$, $f = F_n$ (Eq. B2.1-4)

$= (1.052/\sqrt{3.705})(27.75)\sqrt{21.73/29500} = 0.412 < 0.673$

b $= w$ (Eq. B2.1-1)

$= 2.914$ in. (flanges fully effective)

w/t $= 27.75 < 90$ (Section B1.1-(a)-(1))

Web:

w $= 2.914$ in., $k = 4.00$

λ $= (1.052/\sqrt{4})(2.914/0.105)\sqrt{21.73/29500}$

$= 0.396 < 0.673$

b $= w = 2.914$ in. (web fully effective)

w/t $= 2.914/0.105 = 27.75 < 500$ (Section B1.1-(a)-(2))

Lips:

d $= 0.607$ in.

k $= 0.43$ (unstiffened compression element)

λ $= (1.052/\sqrt{0.43})(0.607/0.105)\sqrt{21.73/29500}$

$= 0.252 < 0.673$

d'_s $= d = 0.607$ in.

d_s $= d'_s(I_s/I_a) \leq d'_s$ (Eq. B4.2-11)

$= 0.607(2.34) = 1.420 > d'_s = 0.607$ in.

d_s $= 0.607$ in. (Lip fully effective in computing the overall effective area)

d/t $= 5.78 < 14$ (Section B4 of the Commentary)

Since flanges, web, and lips are fully effective

A_e $= A = 1.204$ in.^{2}

15. Determination of $\phi_c P_n$:

P_n $= A_e F_n$ (Eq. C4-1)

$= 1.204 \times 21.73$

$= 26.16$ kips

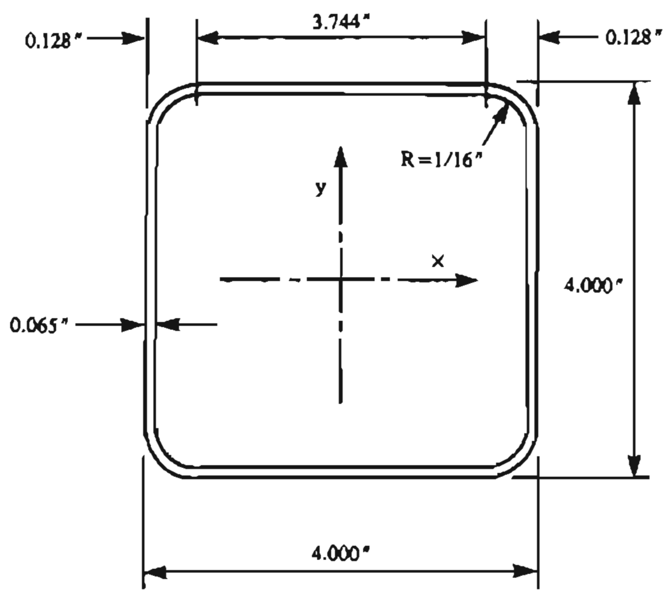
ϕ_c $= 0.85$

$\phi_c P_n$ $= 0.85 \times 26.16$

$= 22.24$ kips

EXAMPLE NO. 13

TUBULAR SECTION - SQUARE



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: 4 x 4 x 0.065 Square Tube.
3. $K_xL_x = K_yL_y = 10$ ft.

Required:

Design axial strength, $\phi_c P_n$.

Solution:

1. Properties of 90° corners:

   $r = R + t/2 = 1/16 + 0.065/2 = 0.095$ in.

   Length of arc, $u = 1.57r = 1.57 \times 0.095 = 0.149$ in.

   Distance of c.g. from center of radius,

   $c = 0.637r = 0.637 \times 0.095 = 0.061$ in.

$I_x = I_y = I$ (doubly symmetric section)

Element	(in.)		y Distance to Center of Section (in.)	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Flanges	2 x 3.744 =	7.488	2 - 0.065/2 = 1.968	29.001	—
Corners	4 x 0.149 =	0.596	3.744/2+0.061 = 1.933	2.227	—
Web	2 x 3.744 =	7.488	—	—	8.747
Sum		15.572		31.228	8.747

w/t $= 3.744/0.065 = 57.60 < 500$ (Section B1.1-(a)-(2))

A $= Lt = 15.572\text{x}0.065 = 1.012$ in.^{2}

I' $= Ly^2 + I'_1 = 31.228 + 8.747 = 39.975$ in.^{3}

I $= I't = 39.975\text{x}0.065 = 2.598$ in.^{4}

r $= \sqrt{I/A} = \sqrt{2.598/1.012} = 1.602$ in.

KL/r $= 10\text{x}12/1.602 = 74.91 < 200$ (Section C4-(d))

2. Since the square tube is a doubly symmetric closed section, provisions of Section C4.1 apply, i.e., section is not subjected to torsional flexural buckling.

$F_e = \pi^2 E/(KL/r)^2$ (Eq. C4.1-1)

$= \pi^2\text{x}29500/(74.91)^2 = 51.89$ ksi

$F_y/2 = 50/2 = 25.00$ ksi

For $F_e > F_y/2$:

$F_n = F_y(1-F_y/4F_e)$ (Eq. C4-2)

$= 50[1-50/(4\text{x}51.89)] = 37.96$ ksi

3. $\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$, $f = F_n$ (Eq. B2.1-4)

$= (1.052/\sqrt{4})(3.744/0.065)\sqrt{37.96/29500} = 1.087 > 0.673$

(Section not fully effective)

$\rho = (1-0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1-0.22/1.087)/1.087 = 0.734$

$b = \rho w$ (Eq. B2.1-2)

$= 0.734\text{x}3.744 = 2.748$ in.

$A_e = A-4(w-b)t$

$= 1.012-4(3.744-2.748)\text{x}0.065$

$= 0.753$ in.^{2}

4. $P_n = A_e F_n$ (Eq. C4-1)

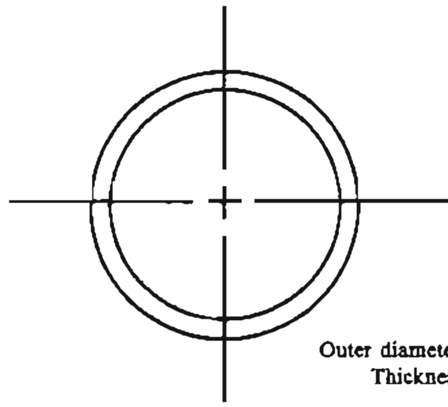
$= 0.753 \text{x} 37.96 = 28.58$ kips

$\phi_c \quad = 0.85$

$\phi_c P_n = 0.85 \times 28.58 = 24.29$ kips

EXAMPLE NO. 14

TUBULAR SECTION - ROUND



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: Shown in sketch above.
3. Height: L = 10′-0", simply supported at each end.

Required:

Design axial strength, $\phi_c P_n$.

Solution:

Ratio of outside diameter to wall thickness,

$D/t = 8.000/0.125 = 64.00$

$D/t < 0.441E/F_y = 0.441(29500/50) = 260.2$ OK

$F_e = \pi^2 E/(KL/r)^2$ (Eq. C4.1-1)

$I = (1/4)\pi[(O.R.)^4 - (I.R.)^4]$

$= (1/4)\pi[(4)^4 - (3.875)^4]$

$= 23.98 \text{ in.}^4$

$A = (1/4)\pi[(O.D.)^2 - (I.D.)^2]$

$= (1/4)\pi[(8)^2 - (7.75)^2]$

$= 3.093 \text{ in.}^2$

$r = \sqrt{I/A}$

$= \sqrt{23.98/3.093} = 2.784$ in.

$F_e = \pi^2(29500)/[10(12)/2.784]^2 = 156.71$ ksi

Since $F_e > F_y/2$

$F_n = F_y(1-F_y/4F_e)$ (Eq. C6.2-2)

$= 50[1-50/(4x156.71)] = 46.01$ ksi

$A_o = [0.037/(DF_y/tE)+0.667]A \leq A$ (Eq. C6.2-5)

$= \{0.037/[8x50/(0.125x29500)]+0.667\}(3.093)$

$= 3.118$ in.^{2}

Therefore, $A_o = A = 3.093$ in.^{2}

$A_e = [1-(1-R^2)(1-A_o/A)]A$ (Eq. C6.2-3)

Since $A_o/A = 1$, $A_e = A$

$A_e = 3.093$ in.^{2}

$P_n = F_nA_e$ (Eq. C6.2-1)

$= (46.01)(3.093)$

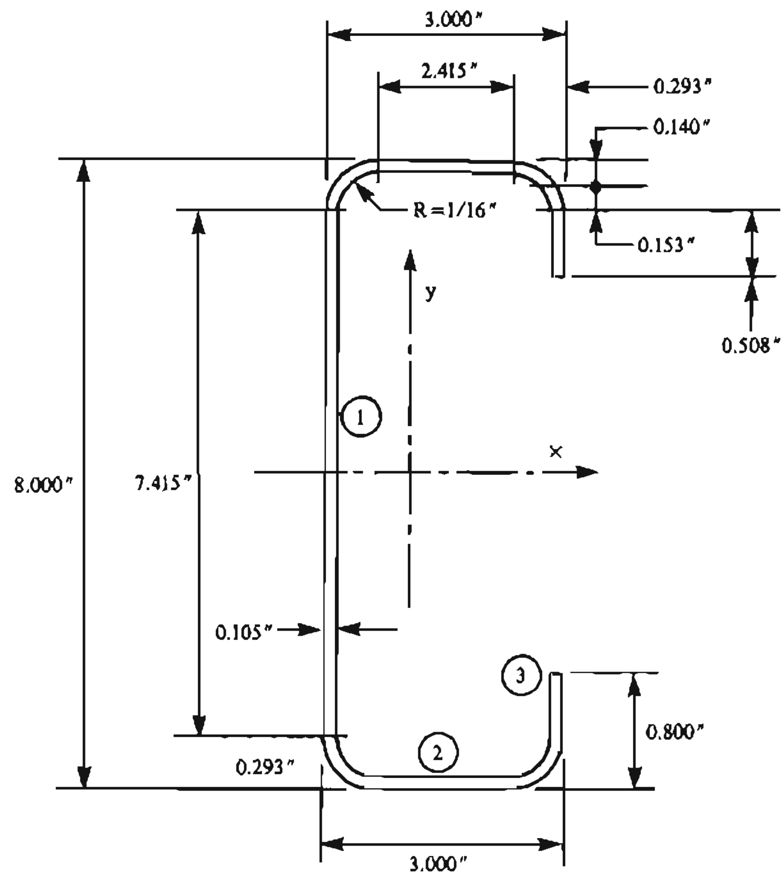
$= 142.31$ kips

$\phi_c = 0.85$

$\phi_cP_n = 0.85 \times 142.31 = 120.96$ kips

EXAMPLE NO. 15

C-SECTION



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: Channel as shown.
3. Length of Section = 16 ft.
4. $L_x = L_y = L_t = 16$ ft.
5. $K_x = K_y = K_t = 1.0$.
6. Axial loads: $P_{DL} = 0.4$ kips, $P_{LL} = 2.0$ kips.
7. Eccentricity at both ends:
   (a) Axial loads are applied 2 in. to the left of the c.g. of the full section.
   (b) Axial loads are applied 2 in. to the left and 4 in. above the c.g. of the full section.

Required:

Check the adequacy of the given section for both cases.

Solution: Part (a)

1. Full section properties:

$r = R+t/2 = 3/16+0.105/2 = 0.240$ in.

$a = A'-(2r+t) = 8.000-(2x0.240+0.105) = 7.415$ in.

$\bar{a} = A'-t = 8.000-0.105 = 7.895$ in.

$b = B'-(2r+t) = 3.000-(2x0.240+0.105) = 2.415$ in.

$\bar{b} = B'-t = 3.000-0.105 = 2.895$ in.

$c = C'-(r+t/2) = 0.800-(0.240+0.105/2) = 0.508$ in.

$\bar{c} = C'-t/2 = 0.800-(0.105/2) = 0.748$ in.

$u = 1.57r = 1.57 \times 0.240 = 0.377$ in.

Distance of corner's c.g. from center of radius = 0.637 r= 0.637(0.240) = 0.153 in.

$$A = t[a+2b+2c+4u] = 0.105[7.415+2x2.415+2x0.508+4x0.377]$$
$$= 1.551 \text{ in.}^2$$

$$I_x = 2t[0.0417a^3+b(a/2+r)^2+2u(a/2+0.637r)^2+0.298r^3$$
$$+0.0833c^3+(c/4)(a-c)^2]$$
$$= 2x0.105[0.0417(7.415)^3+2.415(7.415/2+0.240)^2]$$
$$+2x0.377(7.415/2+0.637x0.240)^2+0.298(0.240)^3$$
$$+0.0833(0.508)^3+(0.508/4)(7.415-0.508)^2]$$
$$= 15.108 \text{ in.}^4$$

$$\bar{x} = (2t/A)[b(b/2+r)+u(0.363r)+u(b+1.637r)+c(b+2r)]$$
$$= (2x0.105/1.551)[2.415(2.415/2+0.240)+0.377(0.363x0.240)$$
$$+0.377(2.415+1.637x0.240)+0.508(2.415+2x0.240)]$$
$$= 0.820 \text{ in.}$$

$$I_y = 2t[b(b/2+r)^2+0.0833b^3+0.505r^3+c(b+2r)^2$$
$$+u(b+1.637r)^2]-A(\bar{x})^2$$
$$= 2x0.105[2.415(2.415/2+0.240)^2+0.0833(2.415)^3$$
$$+0.505(0.240)^3+0.508(2.415+2x0.240)^2$$
$$+0.377(2.415+1.637x0.240)^2]-1.551(0.820)^2$$
$$= 1.786 \text{ in.}^4$$

$$m = (\bar{b}t/12I_x)[6\bar{c}(\bar{a})^2+3\bar{b}(\bar{a})^2-8(\bar{c})^3]$$
$$= [(2.895x0.105)/(12x15.108)][6x0.748(7.895)^2$$
$$+3x2.895(7.895)^2-8(0.748)^3]$$
$$= 1.371 \text{ in.}$$

$x_o = -(\bar{x}+m) = -(0.820+1.371)$

$= -2.191$ in.

$J = (t^3/3)[a+2b+2c+4u]$

$= [(0.105)^3/3][7.415+2x2.415+2x0.508+4x0.377]$

$= 0.005699$ in.^{4}

$C_w = (t^2/A)\{[\bar{x}A(\bar{a})^2/t][(\bar{b})^2/3+m^2-m\bar{b}+(A/3t)[(m)^2(\bar{a})^3$
$+(\bar{b})^2(\bar{c})^2(2\bar{c}+3\bar{a})]-(I_x m^2/t)(2\bar{a}+4\bar{c})+[m(\bar{c})^2/3][8(\bar{b})^2(\bar{c})$
$+2m(2\bar{c}(\bar{c}-\bar{a})+\bar{b}(2\bar{c}-3\bar{a}))]+[(\bar{b})^2(\bar{a})^2/6][(3\bar{c}+\bar{b})(4\bar{c}+\bar{a})-6(\bar{c})^2]$
$-[m^2(\bar{a})^4/4]\}$

$= [(0.105)^2/1.551]\{[0.820x1.551x(7.895)^2/0.105][(2.895)^2/3$
$+(1.371)^2-1.371x2.895]+1.551/(3x0.105)[(1.371)^2(7.895)^3$
$+(2.895)^2(0.748)^2(2x0.748+3x7.895)]$
$-[15.108x(1.371)^2/0.105](2x7.895+4x0.748)$
$+[1.371(0.748)^2/3][8(2.895)^2(0.748)$
$+2x1.371(2x0.748(0.748-7.895)+2.895(2x0.748-3x7.895))]$
$+[(2.895)^2(7.895)^2/6][(3x0.748+2.895)(4x0.748+7.895)$
$-6(0.748)^2]-[(1.371)^2(7.895)^4/4]\}$

$= 23.468$ in.^{6}

$\beta_w = -\{0.0833[t\bar{x}(\bar{a})^3]+t(\bar{x})^3\bar{a}\}$

$= -\{0.0833[0.105x0.820(7.895)^3]+0.105(0.820)^3x7.895\}$

$= -3.987$

$B_f = (t/2)[(\bar{b}-\bar{x})^4-(\bar{x})^4]+[t(\bar{a})^2/4][(\bar{b}-\bar{x})^2-(\bar{x})^2]$

$= (0.105/2)[(2.895-0.820)^4-(0.820)^4]$
$+[0.105(7.895)^2/4][(2.895-0.820)^2-(0.820)^2]$

$= 6.894$

$\beta_l = 2\bar{c}t(\bar{b}-\bar{x})\,3+(2/3)t(\bar{b}-\bar{x})[(\bar{a}/2)^3-(\bar{a}/2-\bar{c})^3]$

$= 2x0.748x0.105(2.895-0.820)^3+(2/3)x0.105(2.895$
$-0.820)\{(7.895/2)^3-[(7.895/2)-0.748]^3\}$

$= 5.581$

$J = (1/2I_y)(\beta_w+\beta_f+\beta_l)-xo$ (Eq. C3.1.2-11)

$= [1/(2x1.786)](-3.987+6.894+5.581)-(-2.191)$

$= 4.567$

$r_x = \sqrt{I_x/A} = \sqrt{15.108/1.551} = 3.121$ in.

$K_xL_x/r_x = [1(16x12)]/3.121 = 61.52$

$r_y = \sqrt{I_y/A} = \sqrt{1.786/1.551} = 1.073$ in.

$K_yL_y/r_y = [1(16x12)]/1.073 = 178.94 < 200$ (Section C4-(d))

$r_o = \sqrt{r_x^2 + r_y^2 + x_o^2}$ (Eq. C3.1.2-10)

$= \sqrt{(3.121)^2+(1.073)^2+(-2.191)^2} = 3.961$ in.

$\beta = 1-(x_o/r_o)^2$ (Eq. C4.2-3)

$= 1-(-2.191/3.961)^2 = 0.694$

2. Determination of $\phi_c P_n$ (Section C4):

Since the channel is singly symmetric, F_e shall be taken as the smaller of F_e calculated according to Section C4.1 or F_e calculated according to Section C4.2.

Section C4.1:

$(F_e)_1 = (\pi^2 E)/(K_y L_y/r_y)^2$ (Eq. C4.1-1)

$= (\pi^2 x29500)/(178.94)^2 = 9.093$ ksi

Section C4.2:

$\sigma_{ex} = (\pi^2 E)/(K_x L_x/r_x)^2$ (Eq. C3.1.2-7)

$= (\pi^2 x29500)/(1x16x12/3.121)^2 = 76.93$ ksi.

$\sigma_t = 1/(Ar_o^2)[\,GJ+(\pi^2 EC_w)/(K_t L_t)^2]$ (Eq. C3.1.2-9)

$= [1/1.551(3.961)^2][11300x0.005699]$

$+(\pi^2 x29500x23.468)/[(1x16x12)^2]$

$= 10.26$ ksi.

$(F_e)_2 = (1/2\beta)[(\sigma_{ex}+\sigma_t)-\sqrt{(\sigma_{ex}+\sigma_t)^2-4\beta\sigma_{ex}\sigma_t}\,]$ (Eq. C4.2-1)

$= 1/(2x0.694)[\,(76.93+10.26)$

$-\sqrt{76.93+10.26)^2-4x0.694x76.93x10.26}\,]$

$= 9.820$ ksi

Therefore

$F_e = 9.093$ ksi

$F_y/2 = 50/2 = 25.00$ ksi

Since $F_e < F_y/2$ it follows that

$F_n = F_e$ (Eq. C4-3)

$= 9.093$ ksi.

For element 1:

$w = 7.415$ in.

$w/t = 7.415/0.105 = 70.62 < 500$ OK (Section B1.1-(a)-(2))

$k = 4.0$ (Since connected to two stiffened elements)

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E},\ f = F_n$ (Eq. B2.1-4)

$= (1.052/\sqrt{4})(70.62)\sqrt{9.093/29500}$

$= 0.652 < 0.673$

$b = w$ (Eq. B2.1-1)

= 7.415 in. (Element 1 fully effective)

For element 2:

w = 2.415 in.

w/t = 2.415/0.105 = 23.00

S = $1.28\sqrt{E/f}$, $f = F_n$ (Eq. B4-1)

= $1.28\sqrt{29500/9.093}$ = 72.91

w/t = 23.00 < S/3=24.30

b = w (Eq. B4.2-3)

= 2.415 in. (Element 2 fully effective)

For element 3:

d = 0.508 in.

d/t = 0.508/0.105 = 4.84 < 14 OK (Section B4 of the Commentary)

k = 0.43 (unstiffened compression element)

λ = $(1.052/\sqrt{0.43})(4.84)\sqrt{9.093/29500}$

= 0.136 < 0.673

d'_s = d = 0.508 in.

d_s = d'_s (Eq. B4.2-4)

= 0.508 in. (Element 3 fully effective)

Thus the whole section is fully effective.

A_e = A = 1.551 in.^{2}

P_n = A_eF_n

= 1.551 x 9.093

= 14.10 kips

ϕ_c = 0.85

ϕ_cP_n = 0.85 x 14.10

= 11.985 kips

3. P_u = 1.2x0.4+1.6x2.0 = 3.68 kips

P_u/ϕ_cP_n = 3.68/11.985 = 0.307 > 0.15

Must check both interaction equations (Eq. C5-1), (Eq. C5-2)

4. Determination of ϕ_cP_{no} (Section C4 for $F_n = F_y$):

For element 1:

λ = $(1.052/\sqrt{4.00})(70.62)\sqrt{50/29500}$ = 1.529 > 0.673

ρ = $(1-0.22/\lambda)/\lambda$ (Eq. B2.1-3)

= (1-0.22/1.529)/1.529 = 0.560

b = ρw (Eq. B2.1-2)

= 0.560x7.415 = 4.152 in.

For element 2:

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

$S/3 = 10.36$

$S/3 = 10.36 < w/t = 23.00 < S = 31.09$

$I_a = 399t^4\{[(w/t)/S] - 0.33\}^3$ (Eq. B4.2-6)

$= 399(0.105)^4[(23/31.09)-0.33]^3 = 0.003337 \text{ in.}^4$

$I_s = d^3t/12 = (0.508)^3(0.105)/12$

$= 0.001147 \text{ in.}^4$

$I_s/I_a = 0.001147/0.003337 = 0.344$

$D/w = 0.8/2.415 = 0.331$

$n = 1/2$

$k = [4.82-5(D/w)](I_s/I_a)^n + 0.43 \leq 5.25-5(D/w)$ (Eq. B4.2-9)

$= [4.82-5(0.331)](0.344)^{1/2} + 0.43 = 2.286$

$5.25-5(0.331) = 3.595 > 2.286$

$k = 2.286$

$\lambda = (1.052/\sqrt{2.286})(23.00)\sqrt{50/29500} = 0.659 < 0.673$

$b = w = 2.415$ in. (Element 2 fully effective)

For element 3:

$\lambda = (1.052/\sqrt{0.43})(4.84)\sqrt{50/29500} = 0.320 < 0.673$

$d'_s = d = 0.508$ in.

$d_s = d'_s(I_s/I_a) \leq d'_s$ (Eq. B4.2-11)

Since $I_s/I_a = 0.344 < 1.0$

$d_s = 0.508(0.344) = 0.175$ in.

$A_e = 1.551-0.105(7.415-4.152)-0.105(0.508-0.175)\text{x}2$

$= 1.138 \text{ in.}^2$

$P_{no} = 1.138 \times 50 = 56.90$ kips

$\phi_c = 0.85$

$\phi_c P_{no} = 0.85 \times 56.90$

$= 48.37$ kips

5. Determination of M_{uy} (required flexural strength about y-axis): ($M_{ux} = 0$ since $e_y = 0$), M_{uy} will be with respect to the centroidal axes of the effective section determined for the required axial strength alone.

$A_e = 1.551 \text{ in.}^2$ under required axial strength alone

Since $A_e = A$, the centroidal axes for the effective section are the same as those for the full section. Therefore, e_x did not change.

$M_{uy} = 3.68(2.00) = 7.36$ kip-in. (Required Flexural Strength)

The interaction equations (Eq. C5-1) and (Eq. C5-2) reduce to the following:

$P_u/\phi_c P_n + C_{my} M_{uy}/\phi_b M_{ny} \alpha_{ny} \le 1.0$ (Eq. C5-1)

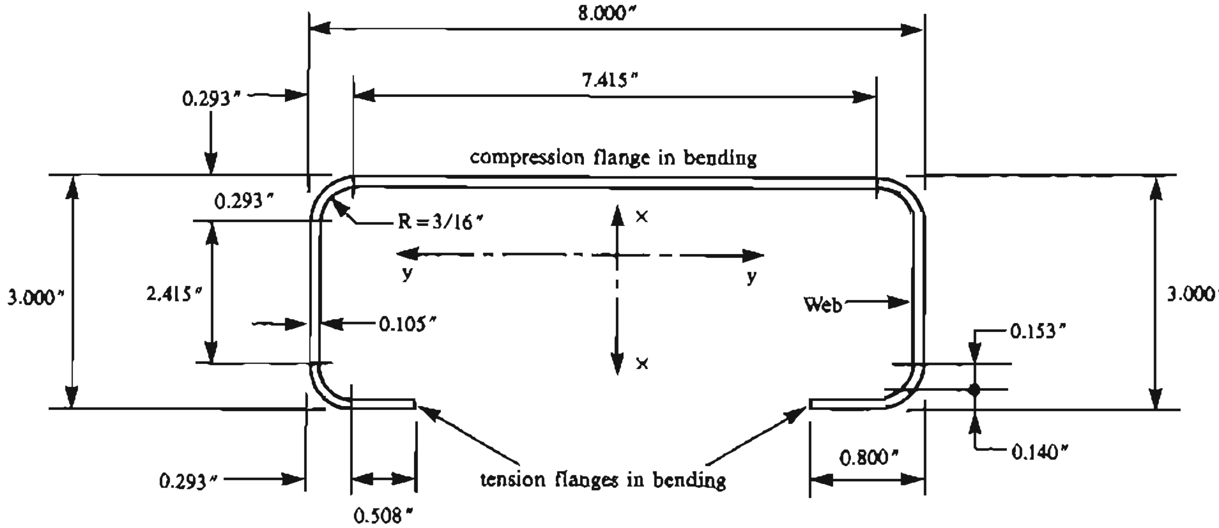
$P_u/\phi_c P_{no} + M_{uy}/\phi_b M_{ny} \le 1.0$ (Eq. C5-2)

6. Determination of $\phi_b M_{ny}$ (Section C3.1):

   $\phi_b M_{ny}$ shall be taken as the smaller of the design flexural strengths calculated according to sections C3.1.1 and C3.1.2:

   a. Section C3.1.1: M_{ny} will be calculated on the basis of initiation of yielding.

   Here it is evident that the initial yielding will not be in the compression flange, rather it will be in the tension flange.



The procedure is iterative: one assumes the actual compressive stress f under M_{ny}. Knowing f one proceeds as usual to obtain x_{cg} (measured from top fiber) to neutral axis. Then one obtains $f = F_y\,[\,x_{cg}/(3 - x_{cg})]$ and checks if it equals to the assumed value. If not, one reiterates by assuming another f until finally it checks. Then for this condition one obtains I_y and $M_{ny} = f(I_y/x_{cg}) = F_y\,[\,I_y/(3 - x_{cg})]$. For the first iteration assume a compressive stress f = 20 ksi in the top compression fibers and that the webs are fully effective.

Compression flange:

$k = 4.00$

$w/t = 7.415/0.105 = 70.62$

$\lambda = (1.052/\sqrt{4.00})(70.62)\sqrt{20/29500} = 0.967 > 0.673$

$\rho = [1-(0.22/0.967)]/0.967 = 0.799$

$b = 0.799 \times 7.415 = 5.925$ in.

To calculate effective section properties about y-axis:

Element	L Effective Length (in.)	x Distance from Top Fiber (in.)	Lx ($in.^2$)	Lx^2 ($in.^3$)	I'_1 About Own Axis ($in.^3$)
Webs	2x2.415 = 4.830	1.500	7.245	10.868	2.347
Upper Corners	2x0.377 = 0.754	0.140	0.106	0.015	—
Lower Corners	2x0.377 = 0.754	2.860	2.156	6.167	—
Compression Flange	5.925	0.053	0.314	0.017	—
Tension Flanges	2x0.508 = 1.016	2.948	2.995	8.830	—
Sum	13.279		12.816	25.897	2.347

Distance from top fiber to y-axis is

x_{cg} = 12.816/13.279 = 0.965 in.

f = $F_y[\, x_{cg} /(3\text{-}x_{cg})]$

= 50 [0.965/(3.00-0.965)] = 23.71 ksi > 20 ksi

need to do another iteration.

For the second iteration assume a compressive stress f = 24.95 ksi in the top compression fibers, and that the webs are fully effective.

Compression flange:

λ = $(1.052/\sqrt{4.00}\,)(70.62)\sqrt{24.95/29500}$ = 1.080 >0.673

ρ = [1-(0.22/1.080)]/1.080 = 0.737

b = 0.737 x 7.415 = 5.465 in.

Effective section properties about y-axis:

Element	L Effective Length (in.)	x Distance from Top Fiber (in.)	Lx ($in.^2$)	Lx^2 ($in.^3$)	I'_1 About Own Axis ($in.^3$)
Webs	2x2.415 = 4.830	1.500	7.245	10.868	2.347
Upper Corners	2x0.377 = 0.754	0.140	0.106	0.015	—
Lower Corners	2x0.377 = 0.754	2.860	2.156	6.167	—
Compression Flange	5.465	0.053	0.290	0.015	—
Tension Flanges	2x0.508 = 1.016	2.948	2.995	8.830	—
Sum	12.819		12.792	25.895	2.347

Distance from top fiber to y-axis is

x_{cg} = 12.792/12.819 = 0.998 in.

f = 50[0.998/(3.00-0.998)] = 24.93 ksi (close enough)

Thus actual compressive stress f = 24.95 ksi

To check if the webs are fully effective (Section B2.3):

f_1 = [(0.998-0.293)/2.002](50) = 17.61 ksi(compression)

f_2 = -[(2.002-0.293)/2.002](50) = -42.68 ksi(tension)

ψ = f_2/f_1 = -42.68/17.61 = -2.424

k = $4+2(1-\psi)^3+2(1-\psi)$ (Eq. B2.3-4)

= $4+2[1-(-2.424)]^3+2[1-(-2.424)]$

= 91.132

h = w = 2.415 in.

w/t = 2.415/0.105 = 23.00 < 200 OK (Section B1.2-(a))

λ = $(1.052/\sqrt{91.132})(23.00)\sqrt{17.61/29500}$ = 0.062 < 0.673

b_e = 2.415 in.

b_2 = $b_e/2$ (Eq. B2.3-2)

= 2.415/2 = 1.208 in.

b_1 = $b_e/(3-\psi)$ (Eq. B2.3-1)

= 2.415/[3-(-2.424)] = 0.445 in.

Compression portion of each web calculated on the basis of the effective section
= x_{cg}-0.293 = 0.998-0.293 = 0.705 in.

Since b_1+b_2 = 1.653 in. > 0.705 in., b_1+b_2 shall be taken as 0.705 in.. This verifies the assumption that the web is fully effective.

I'_y = $Lx^2 + I'_1 - Lx^2_{cg}$

= $25.895 + 2.347 - 12.819(0.998)^2$

= 15.474 in.^{3}

Actual I_y = $I'_y t$= 15.474(0.105) = 1.625 in.^{4}

S_e = $I_y/(3.000-x_{cg})$

= 1.625/(3.000-0.998)

= 0.812 in.^{3}

M_{ny} = S_eF_y (Eq. C3.1.1-1)

= 0.812(50)

= 40.60 kip-in.

ϕ_b = 0.95

$\phi_b M_{ny}$ = 0.95 x 40.65 = 38.57 kip-in.

b. Section C3.1.2: M_{ny} will be calculated on the basis of the lateral buckling strength. (y-axis is the axis of bending).

For the full section:

I_y = 1.786 in.^{4}

x_{cg} = $\bar{x}$+ t/2 = 0.820+0.105/2 = 0.873 in.

S_f = I_y/x_{cg} = 1.786/0.873 = 2.046 in.^{3}

M_y = S_fF_y (Eq. C3.1.2-4)

= 2.046(50) = 102.30 kip-in.

C_s = +1.00

A = 1.551 in.^{2}

σ_{ex} = 76.93 ksi

σ_t = 10.26 ksi

M_1/M_2 = -1.00 (single curvature)

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

= 0.6-0.4(-1.00) = 1.00

r_o = 3.961 in.

j = 4.567

M_e = $C_sA\sigma_{ex}[\,j+C_s\sqrt{j^2+r_o^2(\sigma_t/\sigma_{ex})}\,]/C_{TF}$ (Eq. C3.1.2-6)

= $1.0(1.551)(76.93)[\,4.567$

$+1.00\sqrt{(4.567)^2+(3.961)^2(10.26/76.93)}\,]/1.00$

= 1116.54 kip-in.

M_e = 1116.54 kip-in. > $0.5M_y$ = 51.15 kip-in.

M_c = $M_y[\,1-(M_y/4M_e)]$ (Eq. C3.1.2-2)

= 102.30{1-[102.30/(4x1116.54)]} = 99.96 kip-in.

M_c/S_f = 99.96/2.046 = 48.86 ksi

To calculate effective section properties to obtain S_c at stress 48.86 ksi , we assume that the webs are fully effective.

Compression flange:

λ = $(1.052/\sqrt{4.00})(70.62)\sqrt{48.86/29500}$ = 1.512 > 0.673

ρ = [1-(0.22/1.512)]/1.512 = 0.565

b = 0.565 x 7.415 = 4.189 in.

Effective section properties about y-axis:

Element	L Effective Length (in.)	x Distance from Top Fiber (in.)	Lx ($in.^2$)	Lx^2 ($in.^3$)	I'_1 About Own Axis ($in.^3$)
Webs	2x2.415 = 4.830	1.500	7.245	10.868	2.347
Upper Corners	2x0.377 = 0.754	0.140	0.106	0.015	—
Lower Corners	2x0.377 = 0.754	2.860	2.156	6.167	—
Compression Flange	4.189	0.053	0.222	0.012	—
Tension Flanges	2x0.508 = 1.016	2.948	2.995	8.830	—
Sum	11.543		12.724	25.892	2.347

Distance from top fiber to y-axis is

x_{cg} = 12.724/11.543 = 1.102 in.

To check if the webs are fully effective (Section B2.3):

f_1 = [(1.102-0.293)/1.102](48.86) = 35.87 ksi (compression)

f_2 = -[(1.898-0.293)/1.102](48.86) = -71.16 ksi (tension)

ψ = -71.16/35.87 = -1.984

k = $4+2[1-(-1.984)]^3+2[1-(-1.984)]$

= 63.109

λ = $(1.052/\sqrt{63.109})(23.00)\sqrt{35.87/29500}$ = 0.106 < 0.673

b_e = 2.415 in.

b_2 = 2.415/2 = 1.208 in.

b_1 = 2.415/[3-(-1.984)] = 0.485 in.

Compression portion of each web calculated on the basis of the effective section = 1.102-0.293 = 0.809 in.

Since b_1+b_2 = 1.693 in. > 0.809 in., b_1+b_2 shall be taken as 0.809 in.. This verifies the assumption that the web is fully effective.

I'_y = $25.892 + 2.347 - 11.543(1.102)^2$

= 14.221 $in.^3$

Actual I_y = 14.221(0.105) = 1.493 $in.^4$

S_c = I_y/x_{cg} = 1.493/1.102 = 1.355 $in.^3$

M_{ny} = M_cS_c/S_f (Eq. C3.1.2-1)

= 99.96(1.355)/2.046

= 66.20 kip-in.

ϕ_b = 0.90

ϕ_bM_{ny} = 0.90 x 66.20 = 59.58 kip-in.

ϕ_bM_{ny} shall be the smaller of 38.57 kip-in. and 59.58 kip-in.

Thus

$\phi_b M_{ny}$ = 38.57 kip-in.

7. C_{my} = $0.6-0.4(M_1/M_2)$

M_1/M_2 = -1.00 (single curvature)

$0.6-0.4(-1.00) = 1.00$

C_{my} = 1.00

8. Determination of $1/\alpha_{ny}$:

$\phi_c = 0.85$

$P_E = \pi^2 EI_y/(K_yL_y)^2$ (Eq. C5-5)

$I_y = 1.786$ in.^{4}

$K_yL_y = 1.0(16x12) = 192$ in.

$P_E = [\pi^2 (29500)(1.786)]/(192)^2 = 14.11$ kips

$1/\alpha_{ny} = 1/[1-P_u/(\phi_c P_E)]$ (Eq. C5-4)

$= 1/[1-3.68/(0.85x14.11)] = 1.443$

$\alpha_{ny} = 0.693$

9. Check interaction equations:

$P_u/\phi_cP_n + C_{my}M_{uy}/\phi_bM_{ny}\alpha_{ny} \leq 1.0$ (Eq. C5-1)

$3.68/11.985+1.00x7.36/(38.57x0.693) = 0.307+0.275$

$= 0.582 < 1.0$ OK

$P_u/\phi_cP_{no} + M_{uy}/\phi_bM_{ny} \leq 1.0$ (Eq. C5-2)

$3.68/48.37+7.36/38.57 = 0.076+0.191 = 0.267 < 1.0$ OK

Therefore the section is adequate for the applied loads.

Solution: Part (b)

1. Full section properties are the same as previously calculated in part (a.1).

2. $\phi_cP_n = 11.985$ kips (calculated in part (a)).

3. $P_u/\phi_cP_n = 3.68/11.985 = 0.307 > 0.15$

Therefore the following interaction equations must be satisfied.

$P_u/\phi_cP_n + C_{mx}M_{ux}/\phi_bM_{nx}\alpha_{nx} + C_{my}M_{uy}/\phi_bM_{ny}\alpha_{ny} \leq 1.0$ (Eq. C5-1)

$P_u/\phi_cP_{no} + M_{ux}/\phi_bM_{nx} + M_{uy}/\phi_bM_{ny} \leq 1.0$ (Eq. C5-2)

4. $\phi_cP_{no} = 48.37$ kips (calculated in part (a.4)).

5. Determination of M_{ux} (Section C5):

The centroidal x-axis is the same for both the full and effective sections.

e_y = 4.000 in.

M_{ux} = $P_u e_y$ = 3.68(4.000) = 14.72 kip-in.

6. Determination of $\phi_b M_{nx}$ (Section C3.1):

   $\phi_b M_{nx}$ shall be taken as the smaller of the design flexural strengths calculated according to Sections C3.1.1 and C3.1.2.

   a. Section C3.1.1: M_{nx} will be calculated based on the initiation of yielding.

      First approximation:

      * Assume a compressive stress of $f = F_y = 50$ ksi in the top fiber of the section.
      * Assume that the web is fully effective.

      Compression flange:

      w = 2.415 in.

      w/t = 2.415/0.105 = 23.00

      S = $1.28\sqrt{E/f}$ (Eq. B4-1)

      = $1.28\sqrt{29500/50}$ = 31.09

      For S/3 = 10.36 < w/t = 23.00 < S = 31.09

      I_a = $t^4\, 399\{[(w/t)/S] - 0.33\}^3$ (Eq. B4.2-6)

      = $(0.105)^4(399)[(23.00/31.09)-0.33]^3$

      = 0.003337 in.^{4}

      I_s = $d^3t/12$ (Eq. B4-2)

      = $(0.508)^3(0.105)/12$ = 0.001147 in.^{4}

      I_s/I_a = 0.001147/0.003337 = 0.344

      D = 0.800 in.

      D/w = 0.800/2.415 = 0.331

      w/t = 23.00 < 14 OK (Section B4 of the Commentary)

      For 0.25 < D/w = 0.331 < 0.8

      k = $[4.82-5(D/w)](I_s/I_a)^{1/2}+0.43 \leq 5.25-5(D/w)$ (Eq. B4.2-9)

      = $[4.82-5(0.331)](0.344)^{1/2}+0.43$ = 2.286

      5.25-5(0.331) = 3.595 > 2.286

      k = 2.286

      λ = $(1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

      = $(1.052/\sqrt{2.286})(23.00)\sqrt{50/29500}$ = 0.659 < 0.673

      b = w (Eq. B2.1-1)

      = 2.415 in. (fully effective)

      Compression stiffener:

      d = 0.508 in.

$d/t \quad = 0.508/0.105 = 4.84 < 14$ OK (Section B4 of the Commentary)

$k \quad = 0.43$

Assume max. stress in element, $f = F_y = 50$ ksi although it will be actually less.

$\lambda \quad = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$\quad = (1.052/\sqrt{0.43})(4.84)\sqrt{50/29500} = 0.320$

For $\lambda < 0.673$

$b \quad = w$ (Eq. B2.1-1)

$d'_s \quad = 0.508$ in.

$d_s \quad = d'_s(I_s/I_a) \leq d'_s$ (Eq. B4.2-11)

$\quad = 0.508(0.344)$

$\quad = 0.175$ in.

Effective section properties about x-axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Compression Flange	2.415	0.053	0.128	0.007	—
Compression Stiffener	0.175	0.381	0.067	0.025	—
Compression Corners	2x0.377 = 0.754	0.140	0.106	0.015	—
Web	7.415	4.000	29.660	118.640	33.974
Tension Flange	2.415	7.948	19.194	152.557	—
Tension Stiffener	0.508	7.453	3.786	28.218	0.011
Tension Corners	2x0.377 = 0.754	7.860	5.926	46.582	—
Sum	14.436		58.867	346.044	33.985

Distance from neutral axis to top fiber,

$y_{cg} = Ly/L = 58.867/14.436 = 4.078$ in.

Since the distance from the neutral axis to the top compression fiber is greater than half the depth of the section, a compressive stress of $F_y = 50$ ksi governs as assumed.

$I'_x \quad = Ly^2 + I'_1 - Ly^2_{cg}$

$\quad = 346.044 + 33.985 - 14.436(4.078)^2$

$\quad = 140.0$ in.^{3}

Actual $I_x \quad = tI'_x$

$\quad = (0.105)(140.0) = 14.70$ in.^{4}

Check Web

$w/t \quad = 7.415/0.105 = 70.62 < 200$ OK (Section B1.2-(a))

f_1 = [(4.078-0.293)/4.078](50) = 46.41 ksi(compression)

f_2 = -[(3.922-0.293)/4.078](50) = -44.50 ksi(tension)

ψ = f_2/f_1 = -44.50/46.41 = -0.959

k = 4+2[1-(-0.959)] ^{3} +2[1-(-0.959)]

= 22.95

λ = $(1.052/\sqrt{22.95})(70.62)\sqrt{46.41/29500}$ = 0.615

For λ <0.673

b = w (Eq. B2.1-1)

b_e = 7.415 in.

b_2 = 7.415/2 = 3.708 in.

b_1 = 7.415/[3-(-0.959)] =1.873 in.

b_1+b_2 = 1.873+ 3.708 = 5.581 in. > 3.785 in. (compression portion of web)

Therefore web is fully effective as assumed.

Check Compression Stiffener

Actual maximum stress in stiffener = 46.41 ksi

λ = $(1.052/\sqrt{0.43})(4.84)\sqrt{46.41/29500}$ = 0.308

For λ < 0.673

d'_s = 0.508 in.

Since I_s/I_a is unchanged

d_s = 0.175 in.

Conservative assumption OK

S_e = I_x/y_{cg} = 14.70/4.078 = 3.605 in.^{3}

M_{nx} = S_eF_y (Eq. C3.1.1-1)

= (3.605)(50) = 180.25 kip-in.

ϕ_b = 0.95

ϕ_bM_{nx} = 0.95 x 180.25 = 171.24 kip-in.

b. Section C3.1.2: M_{nx} will be calculated based on the lateral buckling strength.

For the full section:

I_x = 15.108 in.^{4}

y_{cg} = 4.000 in.

S_f = I_x/y_{cg} = 15.108/4.000 = 3.777 in.^{3}

M_y = S_fF_y (Eq. C3.1.2-4)

= 3.777(50) = 188.85 kip-in.

C_b = 1.00 (for members subject to combined axial load and bending moment)

r_o = 3.961 in.

$A = 1.551 \text{ in.}^2$

$\sigma_{ey} = \pi^2 E/(K_y L_y/r_y)^2$

$= \pi^2(29500)/(178.94)^2 = 9.093 \text{ ksi}$

$\sigma_t = 10.26 \text{ ksi}$

$M_e = C_b r_o A \sqrt{\sigma_{ey}\sigma_t}$ (Eq. C3.1.2-5)

$= (1.000)(3.961)(1.551)\sqrt{(9.093)(10.26)}$

$= 59.34 \text{ kip-in.}$

$0.5M_y = 0.5(188.85) = 94.43 \text{ kip-in.}$

For $M_e < 0.5M_y$

$M_c = M_e$ (Eq. C3.1.2-3)

$= 59.34 \text{ kip-in.}$

$M_c/S_f = 59.34/3.777 = 15.71 \text{ ksi}$

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

For compression flange:

$w = 2.415 \text{ in.}$

$w/t = 2.415/0.105 = 23.00$

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

$= 1.28\sqrt{29500/15.71} = 55.47$

$S/3 = 18.49 < w/t = 23.00 < S = 55.47$

$I_a = 399(0.105)^4[(23.00/55.47)-0.33]^3$

$= 0.000029 \text{ in.}^4$

$I_s = 0.001147 \text{ in.}^4$

$I_s/I_a = 0.001147/0.000029 = 39.55$

$k = [4.82-5(0.331)](39.55)^{1/2}+0.43 = 20.334 > 3.595$

$k = 3.595$

$\lambda = (1.052/\sqrt{3.595})(23.00)\sqrt{15.71/29500} = 0.294 < 0.673$

$b = w = 2.415$ in. (compression flange fully effective)

For compression stiffener:

f is taken conservatively 15.71 ksi as in the top compression fiber.

$d/t = 4.84$

$\lambda = (1.052/\sqrt{0.43})(4.84)\sqrt{15.71/29500} = 0.179 < 0.673$

$d'_s = d = 0.508 \text{ in.}$

And since $I_s/I_a = 39.55 > 1.0$

$d_s = d'_s = 0.508$ in. (compression stiffener fully effective)

And since the web was fully effective at the stress $f = F_y = 50$ ksi, it will be fully effective for $f =$ 15.71 ksi. Thus the whole section is fully effective at $M_c/S_f = 15.71$ ksi

Therefore

$S_c = S_e = 3.777$ in.^{3}

$M_{nx} = M_cS_c/S_f$

$= 59.34(3.777)/3.777$

$= 59.34$ kip-in.

$\phi_b = 0.90$

$\phi_bM_{nx} = 0.90 \times 59.34 = 53.41$ kip-in.

ϕ_bM_{nx} shall be the smaller of 171.24 kip-in. and 53.41 kip-in.

Therefore

$\phi_bM_{nx} = 53.41$ kip-in.

7. Determination of C_{mx} (Section C5):

$M_1/M_2 = -1.00$ (single curvature)

$C_{mx} = 0.6\text{-}0.4(-1.0) = 1.00$

8. Determination of α_{nx} (Section C5):

$P_u = 3.68$ kips

$P_E = \pi^2EI_x/(K_xL_x)^2$ (Eq. C5-5)

$= [\ \pi^2\,(29500)(15.108)]\,/[\ 1(16)\text{x}12]^2 = 119.32$ kips

$\phi_c = 0.85$

$1/\alpha_{nx} = 1/[\ 1\text{-Pu}/(\ \phi_cP_E\ )]$ (Eq. C5-4)

$= 1/[\ 1\text{-}3.68/(0.85\text{x}119.32)] = 1.038$

$\alpha_{nx} = 0.964$

9. $M_{uy} = 7.36$ kip-in. (calculated in part (a.5))

10. $\phi_bM_{ny} = 38.57$ kip-in. (calculated in part (a.6))

11. $C_{my} = 1.0$ (calculated in part (a.7))

12. $\alpha_{ny} = 0.693$ (calculated in part (a.8))

13. Interaction equations (Section C5):

$P_u/\phi_cP_n + C_{mx}M_{ux}/\phi_bM_{nx}\alpha_{nx} + C_{my}M_{uy}/\phi_bM_{ny}\alpha_{ny} \leq 1.0$ (Eq. C5-1)

3.68/11.985+1.0x14.72/(53.41x0.964)+1.0x7.36/(38.57x0.693)

$0.307+0.286+0.275 = 0.868 < 1.0$ OK

$P_u/\phi_cP_{no} + M_{ux}/\phi_bM_{nx} + M_{uy}/\phi_bM_{ny} \leq 1.0$ (Eq. C5-2)

3.68/48.37+14.72/53.41+7.36/38.57

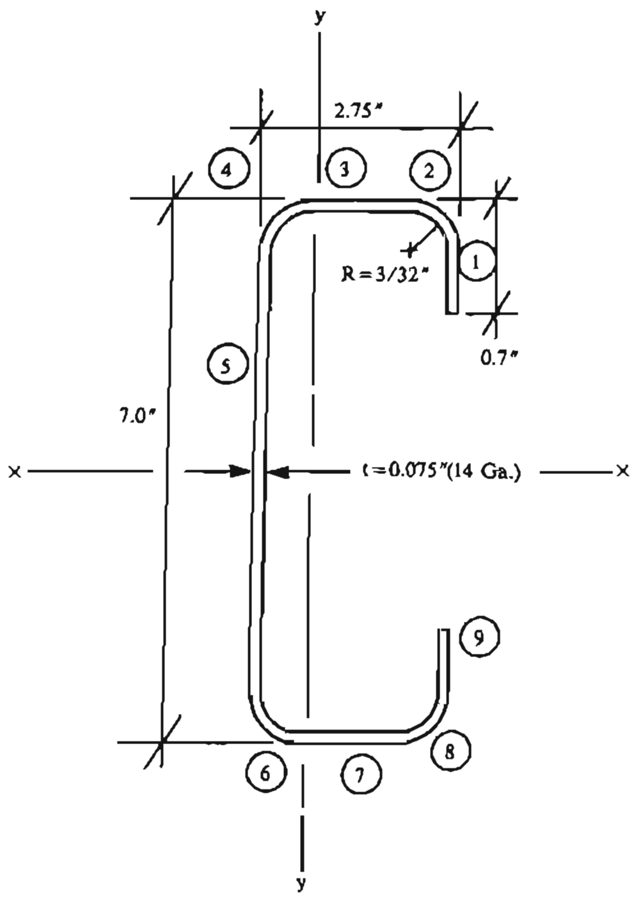
$0.076+0.276+0.191 = 0.543 < 1.0$ OK

Therefore the section is adequate for the applied loads.

EXAMPLE NO. 16

C-SECTION - WALL STUD

Stud Cross Section/Properties



S_x = 2.19 in.^{3}

A = 1.003 in.^{2}

I_x = 7.66 in.^{4}

Iy = 1.00 in.^{4}

r_x = 2.76 in.

r_y = 0.999 in.

J = 0.00188 in.^{4}
(St. Venant Torsion Constant)

C_w = 10.10 in.^{6}
(Warping constant of Torsion)

r_o = 3.57 in.
(Polar r About Shear Ctr.)

x_o = 2.03 in.
(Distance From Shear Ctr. to centroid) along the x-axis

G = 11,300 ksi
(Shear Modulus)

Note : y - y axis perpendicular to wall board

Given:

1. Steel: F_y = 50 ksi.
2. Section: As shown, spacing 24 in. O.C.
3. Length: 15 ft.-0 in.
4. Cladding: On both sides, 1/2 in. gypsum board with No. 6 Type S-12 self-drilling screws @ 12 in. O.C. vertically.
5. Dead load to live load ratio D/L = 1/5.

Required:

1. The design axial strength, $\phi_c P_n$.
2. The permitted service axial load in combination with 5 psf lateral load.

Solution:

1. No Lateral Load (Section D4.1)

   a. Check column buckling between fasteners

   $KL/r_y \quad = (2)(12)/0.999 = 24.0$

   For flexural buckling about the y-y axis

   $F_{e1} \quad = \pi^2 E/(KL/r_y)^2$ (Eq. C4.1-1)

   $= [\pi^2 (29500)]/(24.0)^2 = 505$ ksi

   For torsional-flexural buckling

   $F_{e2} \quad = (1/2\beta)[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\ ]$ (Eq. C4.2-1)

   $\sigma_{ex} \quad = (\pi^2 E)/(K_x L_x/r_x)^2$ (Eq. C3.1.2-7)

   $= \pi^2 (29500)/[(2)(12)/2.76]^2 = 3850$ ksi.

   $\sigma t \quad = (1/Ar_o^2)[GJ + (\pi^2 EC_w)/(K_t L_t)^2]$ (Eq. C3.1.2-9)

   $= [1/1.003(3.57)^2][11300 \text{x} 0.00188]$

   $+ (\pi^2 \text{x} 29500 \text{x} 10.10)/[(2 \text{x} 12)^2] = 401$ ksi

   $\beta \quad = 1 - (x_o/r_o)^2$ (Eq. C4.2-3)

   $= 1 - (2.03/3.57)^2 = 0.677$

   $F_{e2} \quad = [1/(2 \text{x} 0.677)][(3850+401)$

   $- \sqrt{(3850+401)^2 - 4(0.677)(3850)(401)}\ ]$

   $= 387$ ksi < 505 ksi

   Since $F_{e2} > F_y/2 = 25$ ksi

   $F_{n1} \quad = F_y(1 - F_y/4F_e)$ (Eq. C4-2)

   $= 50[1 - 50/(4 \text{x} 387)] = 48.4$ ksi

   b. Check flexural and/or torsional overall column buckling

   $\sigma_{cR} \quad = \sigma_{ey} + \overline{Q}_a$ (Eq. D4.1-2)

   OR

   $\sigma_{cR} \quad = (1/2\beta)[(\sigma_{ex} + \sigma_{tQ}) - \sqrt{(\sigma_{ex} + \sigma_{tQ})^2 - 4\beta\sigma_{ex}\sigma_{tQ}}\ ]$ (Eq. D4.1-3)

   $\sigma_{ex} \quad = \pi^2 E/(L/r_x)^2$ (Eq. D4.1-8)

   $= \pi^2 (29500)/[(15)(12)/2.76]^2 = 68.5$ ksi

   $\sigma_{tQ} \quad = \sigma_t + \overline{Q}_t$ (Eq. D4.1-12).

   $\sigma_t \quad = (1/Ar_o^2)[GJ + (\pi^2 EC_w/L^2)]$ (Eq. D4.1-11)

   $= [1/1.003(3.57)^2][11300 \text{x} 0.00188$

   $+ (\pi^2 \text{x} 29500 \text{x} 10.10)/[(15 \text{x} 12)^2]$

   $= 8.76$ ksi

   $\overline{Q}_t \quad = (\overline{Q} d^2)/(4Ar_o^2)$ (Eq. D4.1-14)

   $\overline{Q} \quad = \overline{q} B$

$B \quad = 24$ in. O.C.

$\bar{q} \quad = \bar{q}_o(2\text{-}s/12)$ (Eq. D4.1-26)

$\bar{q}_o \quad = 2.0$ kip/in.

$s \quad = 12$ in.

$\bar{q} \quad = 2.0(2\text{-}12/12) = 2.0$ kip/in.

$\bar{Q} \quad = 2.0(24) = 48$ kips

$\bar{Q}_t \quad = 48(7)^2/(4)(1.003)(3.57)^2 = 46.0$ ksi

$\sigma_{tQ} \quad = 8.76 + 46.0 = 54.8$ ksi

$\sigma_{ey} \quad = \pi^2E/(L/r_y)^2$ (Eq. D4.1-10)

$= \pi^2(29500)/[(15)(12)/0.999]^2 = 8.97$ ksi

$\bar{Q}_a \quad = \bar{Q}/A$ (Eq. D4.1-13)

$= 48/1.003 = 47.9$ ksi

$\beta \quad = 0.677$ (calculated previously)

$\sigma_{cR} \quad = 8.97 + 47.9 = 56.9$ ksi

OR

$\sigma_{cR} \quad = [1/(2x0.677)][(68.5+54.8)$

$-\sqrt{(68.5 + 54.8)^2 - 4(0.677)(68.5)(54.8)}\,]$

$= 38.6$ ksi

Use $\sigma_{cR} = 38.6$ ksi $= F_e$

$F_e > F_y/2 = 25$ ksi so,

$F_{n2} \quad = F_y(1\text{-}F_y/4F_e)$ (Eq. C4-2)

$= 50[1\text{-}50/(4x38.6)] = 33.8$ ksi

c. Check shear strain of wall material

$C_o \quad = L/350$ (Eq. D4.1-21)

$= (12)(15)/350 = 0.514$ in.

$D_o \quad = L/700$ (Eq. D4.1-22)

$= (12)(15)/700 = 0.257$ in.

$E_o \quad = L/(dx10{,}000)$ (Eq. D4.1-23)

$= (12)(15)/(7x10{,}000)$

$= 0.00257$ rad

Let initial trial value of F_{n3} be based on largest stress for elastic values of E and G

Assume $F_n = 0.5F_y = 0.5(50) = 25$ ksi

$E \quad = 29500$ ksi

$G \quad = 11300$ ksi

$C_1 \quad = (F_nC_o)/(\sigma_{ey}\text{-}F_n + \bar{Q}_a)$ (Eq. D4.1-16)

σ_{ey} = 8.97 ksi

$\overline{Q}_s$ = 47.9 ksi

C_1 = (25)(0.514)/(8.97-25+47.9) = 0.403

$E_1 = \{F_n[(\sigma_{ex} - F_n)(r_o^2E_o - x_oD_o) - F_nx_o(D_o - x_oE_o)]\}/[(\sigma_{ex} - F_n)r_o^2(\sigma_{tQ} - F_n) - (F_nx_o)^2]$ (Eq. D4.1-17)

σ_{ex} = 68.5 ksi

σ_tQ = 54.8 ksi

E_1 = (25){(68.5-25)[(3.57)^2(0.00257)-(2.03)(0.257)] -(25)(2.03)[0.257-(2.03)(0.00257)]}/{(68.5 -25)(3.57)^2(54.8-25)-[(25)(2.03)]^2}

E_1 = -0.0610, use absolute value of 0.0610

$\overline{\gamma}$ = 0.008 in./in. (Table D4)

γ = (π/L)[C_1 +(E_1 d/2)] (Eq D4.1-15)

= [π/(15)(12)][0.403 + (0.0610)(7)/2]

= 0.0108 >$\overline{\gamma}$ = 0.008

Now, try a new value of F_{n3} = 21 ksi

C_1 = (21)(0.514)/(8.97-21+47.9) = 0.301

E_1 = (21){(68.5-21)[(3.57)^2(0.00257)-(2.03)(0.257)] -(21)(2.03)[0.257-(2.03)(0.00257)]}/{(68.5 -21)(3.57)^2(54.8-21)-[(21)(2.03)]^2}

E_1 = -0.0382, use absolute value of 0.0382

γ = [π/(15)(12)][0.301+(0.0382)(7)/2]

= 0.0076 < $\overline{\gamma}$ = 0.008

Interpolating, and trying a value of F_{n3} = 21.5 ksi

C_1 = (21.5)(0.514)/(8.97-21.5+47.9) = 0.312

E_1 = (21.5){(68.5-21.5)[(3.57)^2(0.00257)-(2.03)(0.257)] -(21.5)(2.03)[0.257-(2.03)(0.00257)]}/{(68.5 -21.5)(3.57)^2(54.8-21.5)-[(21.5)(2.03)]^2}

E_1 = -0.0405, use absolute value of 0.0405

γ = [π/(15)(12)][0.312+(0.0405)(7)/2]

= 0.0079 < $\overline{\gamma}$ = 0.008

Trying one final value of F_{n3} = 21.6 ksi

C_1 = (21.6)(0.514)/(8.97-21.6+47.9) = 0.315

E_1 = (21.6){(68.5-21.6)[(3.57)^2(0.00257)-(2.03)(0.257)] -(21.6)(2.03)[0.257-(2.03)(0.00257)]}/{(68.5 -21.6)(3.57)^2(54.8-21.6)-[(21.6)(2.03)]^2}

E_1 = -0.0409, use absolute value of 0.0409

γ = $[\pi/(15)(12)][0.315+(0.0409)(7)/2]$

=0.008 OK

Calculate A_e at F_n = 21.6 ksi, the smallest value of F_{n1}, F_{n2}, and F_{n3}

Web:

w = 6.663 in.

λ = $(1.052/\sqrt{4})(6.663/0.075)\sqrt{21.6/29500}$ = 16.264 (Eq. B2.1-4)

For $\lambda > 0.673$,

ρ = (1-0.22/1.264)/1.264 = 0.653 (Eq. B2.1-3)

β = ρw = (0.653)(6.663) = 4.351 in. (Eq. B2.1-2)

Flange:

w = 2.413 in.

w/t = 32.17

S = $1.28\sqrt{29500/21.6}$ = 47.3 (Eq. B4-1)

S/3 < w/t < S so,

I_a = $399t^4\{[(w/t)/S]-0.33\}^3$ (Eq. B4.2-6)

= $399(0.075)^4[(32.17/47.3)-0.33]^3$

= 0.000542 in.^{4}

Stiffener:

w = 0.531 in.

I_s = $w^3t/12 = (0.531)^3(0.075)/12$

= 0.000936 in.^{4}

λ = $(1.052/\sqrt{0.43})(0.531/0.075)\sqrt{21.6/29500}$ (Eq. B2.1-4)

= 0.307 < 0.673

b = d'_s = 0.531 in.

d_s = d'_s = 0.531 in. (for $I_s \geq I_a$) (Eq. B4.2-11)

D/w = 0.7/2.413 = 0.290

k = $[4.82-5(0.290)](0.000936/0.000542)^{1/2}+0.43$ (Eq. B4.2-9)

= 4.86

5.25-5(0.290) = 3.80 < 4.86, so use k = 3.80

λ = $(1.052/\sqrt{3.80})(32.17)\sqrt{21.6/29500}$ = 0.470 < 0.673

b = w = 2.413 in. (Eq. B2.1-1)

A_e = 1.003-(6.663-4.351)(0.075) = 0.830 in.^{2}

P_n = A_eF_n (Eq. D4.1-1)

= 0.830 x 21.6 = 17.93 kips

ϕ_c $= 0.85$

$\phi_c P_n$ $= 0.85 \times 17.93 = 15.24$ kips

$\phi_c P_n$ $= P_u = 1.2P_{DL} + 1.6P_{LL}$

$= [\ 1.2(P_{DL}/P_{LL})+1.6]P_{LL}$

$= [\ 1.2(1/5)+1.6]P_{LL}$

$= 1.84P_{LL}$

P_{LL} $= \phi_c P_n/1.84 = 15.24/1.84 = 8.28$ kips

P_s $= P_{DL}+ P_{LL}$

$= (1/5+1)P_{LL}$

$= 1.2(8.28) = 9.94$ kips

Where

P_u = Required axial strength

P_s = Service axial load

P_{DL} = Axial load determined on the basis of nominal dead load

P_{LL} = Axial load determined on the basis of nominal live load

2. Permitted Service Axial Load with 5 psf Lateral Load (Section D4.3)

M_{ux} $= 1.3(0.9)(5\text{ psf})(2\text{ ft O.C.})(15\text{ ft})2(12\text{ in./ft})/8$

$= 3948.75$ in.-lbs

$P_u/\phi_c P_n+C_{mx}M_{ux}/\phi_b M_{nx}\alpha_{nx}+C_{my}M_{uy}/\phi_b M_{ny}\alpha_{ny} \le 1.0$ (Eq. C5-1)

M_{uy} $= 0$

Assume $C_{mx} = 1.0$ (braced against joint translation in the plane of loading, subject to transverse loading between supports with member ends unrestrained)

$1/\alpha_{nx} = 1/(1-Pu/\phi_c P_E)$ (Eq. C5-4)

ϕ_c $= 0.85$

P_E $= \pi^2 EI_b/(K_b L_b)^2$ (Eq. C5-5)

$= [\pi^2 (29500)(7.66)] / [\ (12)(15)]^2 = 68.8$ kips

$\phi_c P_E$ $= 0.85(68.8) = 58.48$ kips

$\phi_c P_n$ $= 15.24$ kips

M_{nx} $= M_{nxo}$ = Nominal moments about the centroidal axes determined in accordance with Section C3.1 except lateral buckling provisions

Following Procedure I - Based on Initiation of Yielding

ϕ_b = 0.95 for section with stiffened or partially stiffened compression flanges

M_{nx} $= S_{ex}F_y$ (Eq. C3.1.1-1)

F_y $= 50$ ksi

Calculation of S_{ex} with extreme compression fiber at F_y

Stiffener (compression): Assume maximum stress is F_y initially, although actually it will be less.

$w = 0.531$ in.

$\lambda = (1.052/\sqrt{0.43})(0.531/0.075)\sqrt{50/29500}$ (Eq. B2.1-4)

$= 0.468 < 0.673$

$I_s = 0.000936 \text{ in.}^4$

$b = d'_s = 0.531$ in. (max. stress assumption OK)

$d_s = d'_s(I_s/I_a)$ (Eq. B4.2-11)

$= 0.531(0.000936/0.00392) = 0.127$ in.

See below for calculation of I_a.

Flange (compression):

$w = 2.413$ in.

$w/t = 32.17$

$S = 1.28\sqrt{29500/50} = 31.1$ (Eq. B4-1)

$w/t \geq S$ so,

$I_a = t^4\{[115(w/t)/S] + 5\}$ (Eq. B4.2-13)

$= (0.075)^4\{[115(32.17)/31.1] + 5\} = 0.00392 \text{ in.}^4$

$D/w = 0.290$

$k = [4.82-5(0.290)](0.000936/0.00392)^{1/3} + 0.43 = 2.52$ (Eq. B4.2-9)

$5.25-5(0.290) = 3.80 > 2.52$ OK

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{2.52})(32.17)\sqrt{50/29500} = 0.878$

$\rho = (1-0.22/0.878)/0.878 = 0.854$ (Eq. B2.1-3)

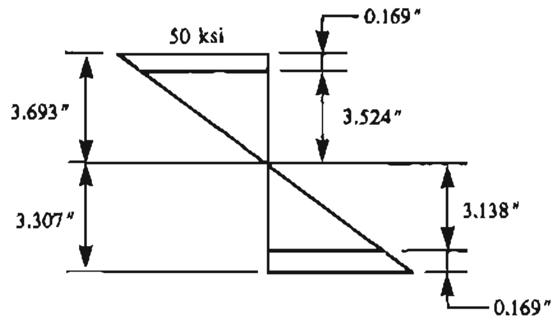
$b = (0.854)(2.413) = 2.061$ in. (Eq. B2.1-2)

Assume the web is fully effective initially.

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
1	0.127	0.232	0.029	0.007	—
2	0.206	0.085	0.018	0.001	—
3	2.061	0.038	0.078	0.003	—
4	0.206	0.085	0.018	0.001	—
5	6.663	3.500	23.321	81.622	24.651
6	0.206	6.915	1.424	9.850	—
7	2.413	6.963	16.802	116.990	—
8	0.206	6.915	1.424	9.850	—
9	0.531	6.566	3.487	22.893	0.012
Sum	12.619		46.601	241.217	24.663

y_{cg} = 46.601/12.619 = 3.693 in.

I_{eff} = 0.075(241.217+24.663-12.619x3.693^2) = 7.03 in.^{4}



Now, check to see if the web is fully effective.

f_1 = [(3.693-0.169)/3.693]x50 = 47.7 1 ksi(compression)

f_2 = -[(3.307-0.169)/3.693]x50 = -42.49 ksi (tension)

ψ = f_2/f_1 = -42.49/47.71 = -0.8906

k = 4+2(1-ψ)^3+2(1-ψ) (Eq. B2.3-4)

= 4+2[1-(-0.8906)] ^{3} +2[1-(-0.8906)] = 21.30

λ = $(1.052/\sqrt{21.30})(6.663/0.075)\sqrt{47.71/29500}$ (Eq. B2.1-4)

= 0.814 > 0.673

ρ = (1-0.22/0.814)/0.814 = 0.896 (Eq.B2.1-3)

b_e = 0.896 x 6.663 = 5.970 in. (Eq. B2.1-2)

b_2 = b_e/2 = 5.970/2 = 2.985 in. (Eq. B2.3-2)

b_1 = b_e/(3-ψ) = 5.970/(3+0.8906) = 1.534 in. (Eq. B2.3-1)

b_1+b_2 = 4.519 > 3.524 (compressed portion of the web)

So, the web is fully effective.

S_{ex} = 7.03/3.693 = 1.904 in.^{3}

M_{nx} = (1.904)(50) = 95.2 kip-in.

$\phi_b M_{nx}$ = 0.95(95.2) = 90.44 kip-in.

Using the interaction equation,

$P_u/15.24+(3948.75/1000)/[90.44(1-P_u/58.48)] \leq 1.0$

Iterating with different values of P_u on the left hand side yields a required axial strength,

P_u (with 5 psf wind load) = 14.36 kips

P_u = $1.2P_{DL} + 0.5P_{LL}$

= $[1.2(P_{DL}/P_{LL})+0.5]P_{LL}$

= $[1.2(1/5)+0.5]P_{LL}$

= $0.74P_{LL}$

$P_{LL} = P_u/0.74 = 14.36/0.74 = 19.41$ kips

$P_s = P_{DL}+P_{LL}$

$= (1/5+1)P_{LL}$

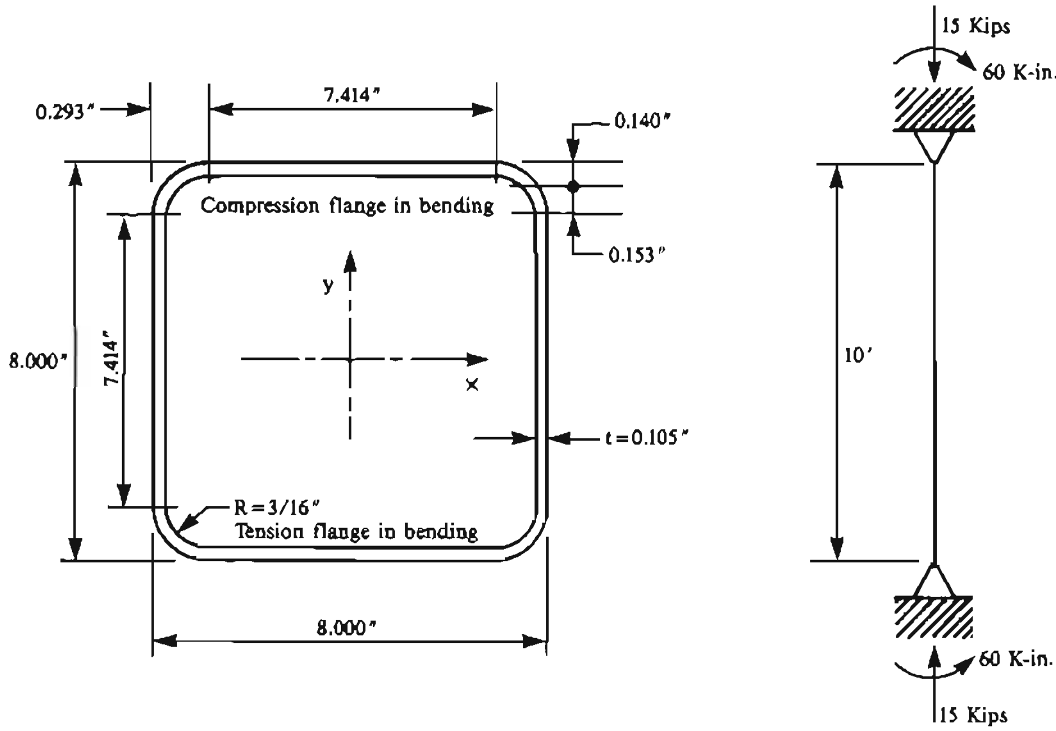
$= 1.2(19.41) = 23.29$ kips

The axial load alone controls, so $P_s = 9.94$ kips

Note that (Eq. C5-2) does not control in this case since the simply supported stud has zero end moments ($M_{ux} = 0$) and $\phi_c P_n < \phi_c P_{no}$ (design axial strength for zero length stud).

EXAMPLE NO. 17

TUBULAR SECTION



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: 8 x 8 x 0.105 Square Tube.
3. Unbraced length of column: 10 ft.
4. $K_x = K_y = 1.0$
5. Service axial load: P = 15 kips.
6. The eccentricity of axial load at each end of member, e_y, is 4 in. and member is bent in single curvature about x-axis.
7. $e_x = 0$.
8. Dead load to live load ratio D/L = 1/5 and 1 .2D + 1.6L governs the design.

Required:

Check the adequacy of the given section.

Solution:

1. Full section properties:
   $r = R + t/2 = 3/16+0.105/2 = 0.240$ in.

Length of arc, $u = 1.57r = 1.57 \times 0.240 = 0.377$ in.

Distance of c.g. from center of radius,

$c = 0.637r = 0.637 \times 0.240 = 0.153$ in.

$I_x = I_y$ (doubly symmetric section)

Element	L (in.)		y Distance to Center of Section (in.)	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Flanges	2 x 7.414 =	14.828	3.948	231.120	—
Corners	4 x 0.377 =	1.508	3.860	22.469	—
Webs	2 x 7.414 =	14.828	—	—	67.921
Sum		31.164		253.589	67.921

$A = Lt = 31.164 \times 0.105 = 3.272$ in.^{2}

$I' = Ly^2 + I'_1 = 253.589 + 67.921 = 321.510$ in.^{3}

$I_x = I_y = I't = 321.510 \times 0.105 = 33.759$ in.^{4}

$r_x = r_y = \sqrt{33.759/3.272} = 3.212$ in.

$S_x = I_x/4.000 = 33.759/4.000 = 8.440$ in.^{3}

$K_xL_x/r_x = 1.0(10 \times 12)/3.212 = 37.36 < 200$ OK (Section C4-(d))

2. Determination of $\phi_c P_n$ (Section C4):
Since the square tube is a doubly symmetric closed section, provisions of Section C4.1 apply, i.e., section is not subjected to torsional flexural buckling.

$F_e = \pi^2 E/(K_xL_x/r_x)^2$ (Eq. C4.1-1)

$= (\pi^2 \times 29500)/(37.36)^2 = 208.60$ ksi

$F_y/2 = 50/2 = 25.00$ ksi

For $F_e > F_y/2$:

$F_n = F_y(1 - F_y/4F_e)$ (Eq. C4-2)

$= 50[1 - 50.00/(4 \times 208.60)] = 47.00$ ksi

$w = 7.414$ in.

$w/t = 7.414/0.105 = 70.61 < 500$ OK (Section B1.1-(a)-(2))

$k = 4.00$ (Section B2.1-(a))

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$, $f = F_n$ (Eq. B2.1-4)

$= (1.052/\sqrt{4.00})(70.61)\sqrt{47/29500} = 1.482 > 0.673$

$\rho = (1 - 0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1 - 0.22/1.482)/1.482 = 0.575$

$b \quad = \rho w$ (Eq. B2.1-2)

$\quad = 0.575 \times 7.414 = 4.263$ in.

$A_e \quad = A\text{-}4(w\text{-}b)t$

$\quad = 3.272\text{-}4(7.414\text{-}4.263)(0.105) = 1.949 \text{ in.}^2$

$P_n \quad = A_e F_n$ (Eq. C4-1)

$\quad = 1.949 \times 47.00 = 91.60$ kips

$\phi_c \quad = 0.85$

$\phi_c P_n \quad = 0.85 \times 91.60 = 77.86$ kips

3. $P_{DL}+P_{LL}=(P_{DL}/P_{LL}+1)P_{LL}$

$\quad = (1/5+1)P_{LL} = 1.2P_{LL} = P$

$P_{LL} \quad = P/1.2 = 15/1.2 = 12.5$ kips

$P_u \quad = 1.2P_{DL}+1.6P_{LL} = (1.2P_{DL}/P_{LL}+1.6)P_{LL} = [1.2(1/5)+1.6](12.5) = 23$ kips

where

P_{DL} = Axial load determined on the basis of nominal dead load

P_{LL} = Axial load determined on the basis of nominal live load

$P_u/\phi_c P_n$ = $23/77.86 = 0.295 > 0.15$

Must check both interaction equations (Eq. C5-1), (Eq. C5-2).

4. Determination of $\phi_c P_{no}$ (Section C4 for $Fn = F_y$)

$\lambda \quad = (1.052/\sqrt{4.00})(70.61)\sqrt{50/29500} = 1.529 > 0.673$

$\rho \quad = (1\text{-}0.22/1.529)/1.529 = 0.560$

$b \quad = 0.560 \times 7.414 = 4.152$ in.

$A_e \quad = 3.272\text{-}4(7.414\text{-}4.152)(0.105) = 1.902 \text{ in.}^2$

$P_{no} \quad = 1.902 \times 50.00 = 95.10$ kips

$\phi_c P_{no} \quad = 0.85 \times 95.10 = 80.84$ kips

5. Determination of M_{ux}, M_{uy} (Section C5):

Since the section is doubly symmetric, the centroidal axes of the effective section at $\phi_c P_n$ are the same as those of the full section.

$M_{ux} \quad = P_u e_y = 23 \times 4 = 92$ kip-in.

$M_{uy} \quad = P_u e_x = 0$

Since $M_{uy} = 0$, the interaction equations (Eq. C5-1) and (Eq.C5-2)reduce to the following :

$P_u/\phi_c P_n + C_{mx} M_{ux}/\phi_b M_{nx} \alpha_{nx} \leq 1.0$ (Eq. C5-1)

$P_u/\phi_c P_{no} + M_{ux}/\phi_b M_{nx} \leq 1.0$ (Eq. C5-2)

6. Determination of $\phi_b M_{nx}$ (Section C3.1):

$\phi_b M_{nx}$ shall be taken as the smaller of the design flexural strengths calculated according to Sections C3.1.1 and C3.1.2:

a. Section C3.1.1: M_{nx} will be calculated on the basis of initiation of yielding.

Computation of I_x:

For the first approximation, assume a compression stress of $f = F_y = 50$ ksi in the compression flange, and that the web is fully effective.

Compression flange: k=4.00 (stiffened compression element supported by a web on each longitudinal edge)

w/t $= 7.414/0.105 = 70.61 < 500$ OK (Section B1.1-(a)-(2))

λ $= (1.052/\sqrt{4.00})(70.61)\sqrt{50/29500} = 1.529 > 0.673$

ρ $= (1\text{-}0.22/1.529)/1.529 = 0.560$

b $= 0.560 \times 7.414 = 4.152$ in.

Effective section properties about x-axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Webs	14.828	4.000	59.312	237.248	67.921
Upper Corners	0.754	0.140	0.106	0.015	—
Lower Corners	0.754	7.860	5.926	46.582	—
Compression Flange	4.152	0.053	0.220	0.012	—
Tension Flange	7.414	7.948	58.926	468.348	—
Sum	27.902		124.490	752.205	67.921

Distance from top fiber to x-axis is

$y_{cg} = Ly/L = 124.490/27.902 = 4.462$ in.

Since the distance of top compression fiber from neutral axis is greater than one half the section depth (i.e., 4.462 > 4.000), a compression stress of 50 ksi will govern as assumed (i.e., initial yielding is in compression).

To check if the web is fully effective (Section B2.3)

f_1 $= [(4.462\text{-}0.293)/4.462](50) = 46.72$ ksi(compression)

f_2 $= -[(3.538\text{-}0.293)/4.462](50) = -36.36$ ksi(tension)

ψ $= f_2/f_1 = -36.36/46.72 = -0.778$

k $= 4+2[1\text{-}(-0.778)]^3+2[1\text{-}(-0.778)]$

$= 18.798$

h $= w = 7.414$ in., $h/t = w/t = 7.414/0.105 = 70.61$

h/t $= 70.61 < 200$ OK (Section B1.2-(a))

λ $= (1.052/\sqrt{18.798})(70.61)\sqrt{46.72/29500} = 0.682 > 0.673$

ρ $= (1\text{-}0.22/0.682)/0.682 = 0.993$

b_e $= 0.993 \times 7.414 = 7.362$ in.

$b_2 = b_e/2$ (Eq. B2.3-2)

$= 7.362/2 = 3.681$ in.

$b_1 = b_e/(3-\psi)$ (Eq. B2.3-1)

$= 7.362/[3-(-0.778)] = 1.949$ in.

Compression portion of the web calculated on the basis of the effective section = y_{cg}-0.293 = 4.462-0.293 = 4.169 in.

Since $b_1+b_2 = 5.630$ in. > 4.169 in., b_1+b_2 shall be taken as 4.169 in.

This verifies the assumption that the web is fully effective.

$I'_x = Ly^2 + I'_1 - Ly^2_{cg}$

$= 752.205 + 67.921 - 27.902(4.462)^2$

$= 264.613$ in.^{3}

Actual $I_x = tI'_x$

$= (0.105)(264.613) = 27.784$ in.^{4}

$S_e = I_x/y_{cg} = 27.784/4.462 = 6.227$ in.^{3}

$M_{nx} = S_eF_y$ (Eq. C3.1.1-1)

$= (6.227)(50) = 311.35$ kip-in.

$\phi_b = 0.95$

$\phi_bM_{nx} = 0.95 \times 311.35 = 295.78$ kip-in.

b. Section C3.1.2: M_{nx} will be calculated on the basis of lateral buckling strength. However for this square tube (closed box-type member) the provisions of Section C3.1.2 do not apply.

Therefore

$\phi_bM_{nx}= 295.78$ kip-in.

7. $C_{mx} = 0.6-0.4(M_1/M_2)$

$M_1/M_2 = -(92/92) = -1.0$ (single curvature)

$0.6-0.4(M_1/M_2) = 0.6-0.4(-1.0) = 1.0$

8. Determination of $1/\alpha_{nx}$:

$\phi_c = 0.85$

$P_E = \pi^2EI_x/(K_xL_x)^2$ (Eq. C5-5)

$I_x = 33.759$ in.^{4}

$K_xL_x = 1.0(10\text{x}12) = 120$ in.

$P_E = [\pi^2 (29500)(33.759)]/(120)2 = 682.57$ kips

$1/\alpha_{nx} = 1/(1-Pu/\phi_cP_E)$ (Eq. C5-4)

$= 1/[1-23/(0.85\text{x}682.57)] = 1.041$

$\alpha_{nx} = 0.960$

9. Check interaction equations:

$P_u/\phi_cP_n+C_{mx}M_{ux}/\phi_bM_{nx}\alpha_{nx} \le 1.0$ (Eq. C5-1)

$23/77.86+1x92/(295.78x0.960) = 0.295+0.324 = 0.619 < 1.0$ OK

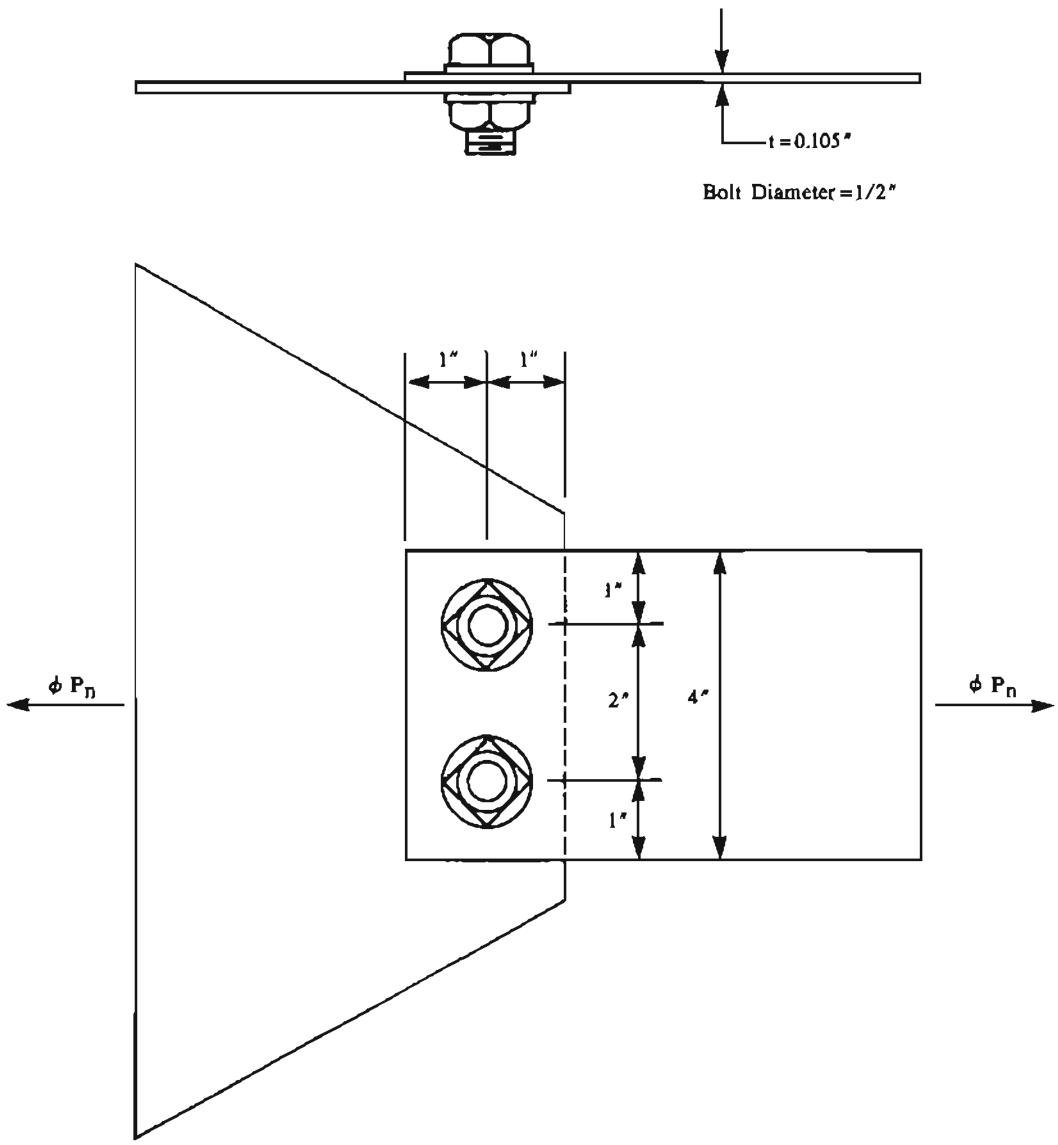
$P_u/\phi_c P_{no}+M_{ux}/\phi_b M_{nx} \leq 1.0$ (Eq. C5-2)

$23/80.84+92/295.78 = 0.285+0.311 = 0.596 < 1.0$ OK

Therefore the section is adequate for the applied loads.

EXAMPLE NO. 18

FLAT SECTION WITH BOLTED CONNECTION



Given:

1. Steel: F_y = 33 ksi, F_u = 45 ksi.
2. Bolts conforming to ASTM A307 with washers under bolt head and nut.

3. Detail of connection shown in sketch.

Required:

Determine the maximum design strength, ϕP_n.

Solution:

Thickness of thinnest part connected, t

$t = 0.105$ in. < 3/16 = 0.188 in.

Therefore, Section E3 applies.

1. Design strength based on spacing and edge distance (Section E3.1)

a. $F_u/F_y = 45/33 = 1.36 > 1.15$

For $F_u/F_y > 1.15$, $\phi = 0.70$

$P_n = teF_u$ (Eq. E3.1-1)

$= 0.105(1)(45) = 4.73$ kips/bolt

$\phi P_n = 0.7(2 \text{ bolts})(4.73 \text{ kips/bolt}) = 6.62$ kips

b. Distance between bolt hole centers must be greater than 3d.

$3d = 3(0.5) = 1.5$ in. <2 in. OK

c. Distance between bolt hole center and edge of connecting member must be greater than 1.5d.

$1.5d = 1.5(0.5) = 0.75$ in. <1 in. OK

2. Design strength based on tension on net section.

Required tension strength on net section of bolted connection shall not exceed $\phi_t T_n$ from Section C2:

A_n - based on Table E3

$A_n = 0.105\,[\,4-2(1/2+1/16)\,] = 0.302 \text{ in.}^2$

$T_n = A_n F_y$ (Eq. C2-1)

$= (0.302)(33) = 9.97$ kips

$\phi_t = 0.95$

$\phi_t T_n = 0.95(9.97) = 9.47$ kips

or ϕP_n from Section E3.2:

Since t = 0.105 in. < 3/16 in. and washers are provided under both bolt head and nut

$P_n = (1.0-0.9r+3rd/s)F_u A_n \le F_u A_n$ (Eq. E3.2-1)

where in this case:

$r = 2(\phi P_n/2)/\phi P_n = 1$

$d = 0.5$ in.

$s \quad = 2$ in.

$P_n \quad = [\,1.0\text{-}0.9(1)+3(1)(0.5)/2]\,(45)(0.302)$

$\quad\quad = 11.55$ kips $< 45(0.302) = 13.59$ kips OK

$\phi \quad = 0.55$ for single shear connection

$\phi P_n \quad = 0.55(11.55) = 6.35$ kips

Therefore, design strength based on tension on net section is 6.35 kips.

3. Design strength based on bearing (Section E3.3)

   For single shear with washers under bolt head and nut; 0.024 in.$\leq t = 0.105$ in. $< 3/16$ in.

   From Table E3.3-1

   $P_n \quad = 3.00F_u dt = 3.00(45)(0.5)(0.105) = 7.09$ kips/bolt

   $\phi \quad = 0.60$

   $\phi P_n \quad = 0.60$(2 bolts)(7.09 kips/bolt) $= 8.51$ kips

4. Design strength based on bolt shear (Section E3.4)

   $P_n \quad = A_b F_n$ (Eq. E3.4-1)

   $A_b \quad = (\pi/4)(0.5)^2 = 0.196$ in.^{2}

   $F_n \quad = F_{nv} = 27$ ksi (Table E3.4-1, $d \geq 1/2$ in.)

   $P_n \quad = (27)(0.196) = 5.29$ kips/bolt

   $\phi \quad = 0.65$

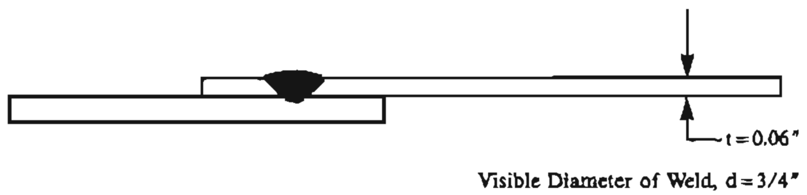
   $\phi P_n \quad = 0.65$(2 bolts)(5.29 kips/bolt) $= 6.88$ kips

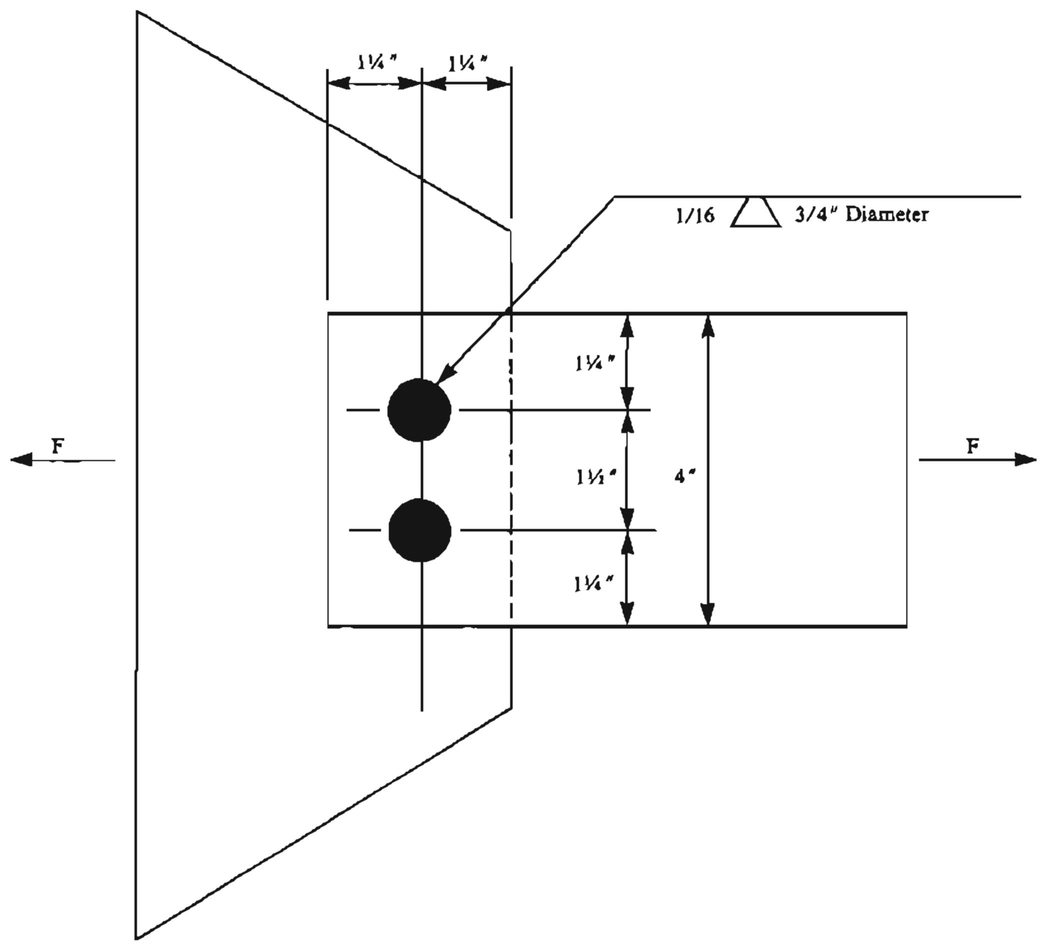
5. Comparing the values from 1, 2, 3, and 4 above, the design tensile strength on the net section of the connected part controls and

   $\phi P_n \quad = 6.35$ kips

EXAMPLE NO. 19

FLAT SECTION WITH ARC SPOT WELDED CONNECTION





Given:

1. Steel: $F_y = 50$ ksi, $F_u = 65$ ksi.
2. Total Required Strength, F = 6.8 kips

3. Detail of connection shown in sketch.

Required:

Design the connection to transmit F = 6.8 kips using arc spot welds having 3/4 in. visible diameter.

Solution:

1. Weld Dimensions

$d = 0.75$ in.

$d_a = d\text{-}t = 0.75\text{-}0.06 = 0.69$ in.

$d_e = 0.7d\text{-}1.5t$ but $\leq 0.55d$ (Eq. E2.2-5)

$= 0.7(0.75)\text{-}1.5(0.06) = 0.44$

$0.55d = 0.55(0.75) = 0.41$ in.

0.44 in. > 0.41 in., use $d_e = 0.41$ in. > 3/8 in. OK

2. Determine number of arc spot welds required.

a. $P_n = 0.589d_e2F_{xx}$ (Eq. E2.2-1)

Using E60 electrode, $F_{xx} = 60$ ksi

$P_n = 0.589(0.41)^2(60) = 5.94$ kips/weld

$\phi = 0.60$

$\phi P_n = 0.60(5.94) = 3.56$ kips/weld

b. Compute $d_a/t = 0.69/0.06 = 11.5$

Compute $\sqrt{E/F_u} = \sqrt{29500/65} = 21.3$

For $d_a/t = 11.5 < 0.815\sqrt{E/F_u} = 17.4$

$P_n = 2.20td_aF_u$ (Eq. E2.2-2)

$= 2.20(0.06)(0.69)(65) = 5.92$ kips/weld

$\phi = 0.60$

$\phi P_n = 0.60(5.92) = 3.55$ kips/weld (control)

Number of welds = 6.8 kips/(3.55 kips/weld) = 1.92 welds, use 2.

3. Check the edge distance and spacing requirements

a. $F_u/F_y = 65/50 = 1.3 > 1.15$

For $F_u/F_y > 1.15$, $\phi = 0.70$

$e_{min} = P_u/\phi F_u t$ (Eq. E2.2-6)

$= (6.8/2)/[(0.70)(65)(0.06)] = 1.25$ in.

1.25 in. edge distance = e_{min} = 1.25 in. OK

b. Edge distance shall not be less than 1.5d.

$1.5d = 1.5(0.75) = 1.13$ in. < 1.25 in. OK

c. Clear distance between weld and end of member shall not be less than 1.0d.

$1.0d = 1.0(0.75) = 0.75$ in.

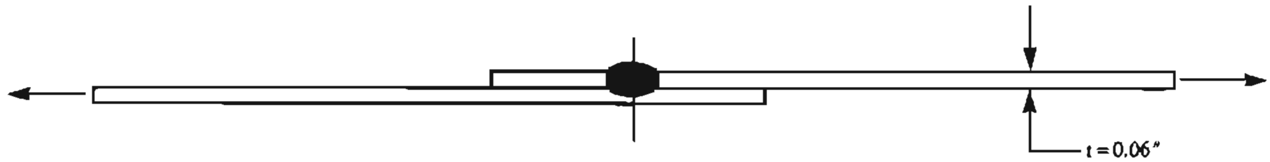
Clear distance = 1.25-0.375 = 0.875 in. > 0.75 in. OK

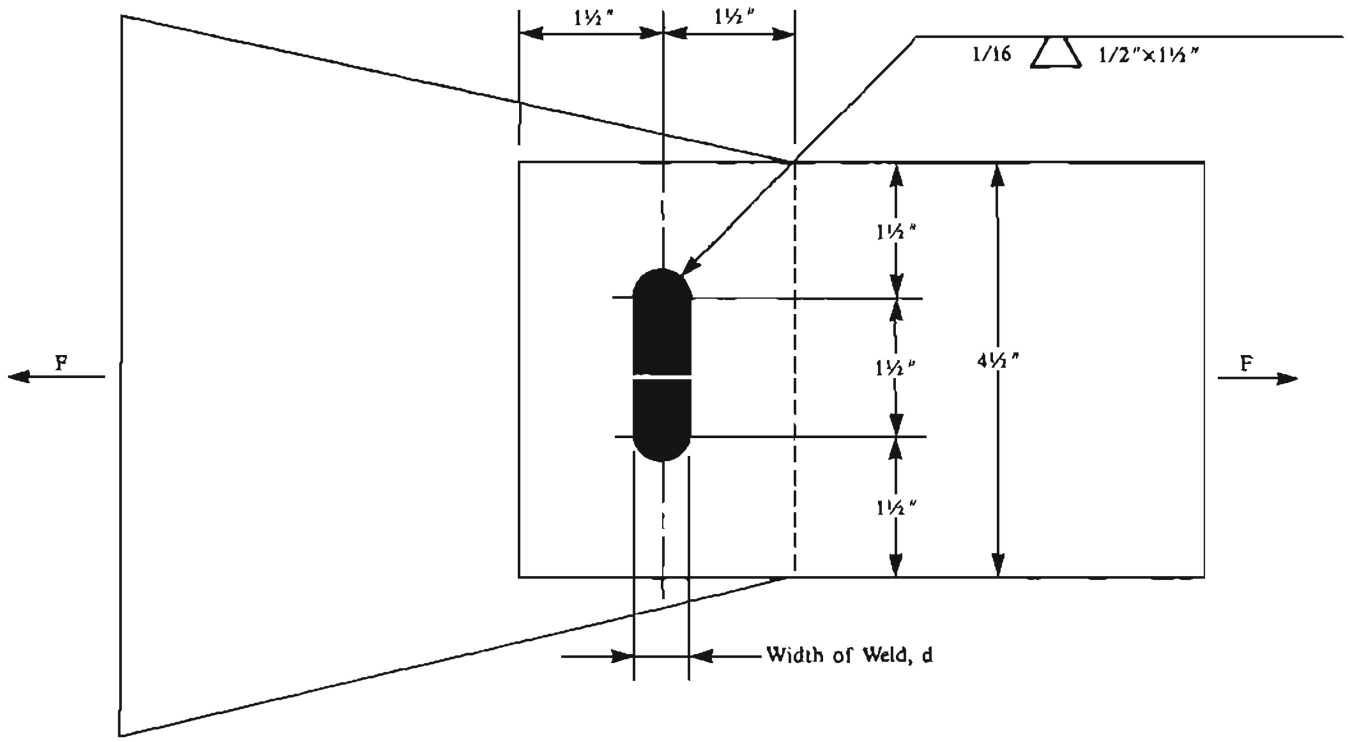
d. Thinnest connected part, $t = 0.06$ in. < 0.15 in. OK

4. Use 2-3/4 in. diameter spot welds in the configuration shown. No weld washers required because $t = 0.06$ in. > 0.028 in.

EXAMPLE NO. 20.

FLAT SECTION WITH ARC SEAM WELDED CONNECTION





Given:

1. Steel: $F_y = 50$ ksi, $F_u = 65$ ksi.
2. Total Required Strength, $F = 4.1$ kips

3. Detail of connection shown in sketch.

Required:

Design the connection to transmit F = 4.1 kips using arc seam welds.

Try d = 1/2 in..

Solution:

1. Required strength shall not exceed either

ϕ = 0.60

$P_n = (\pi d_e 2/4+Ld_e)(0.75F_{xx})$ (Eq. E2.3-1)

Try E60 electrode, F_{xx} = 60 ksi

L = 1.5 in., or maximum 3d, 3(0.5) = 1.5 in. OK

d_a = 0.5-0.06 = 0.44 in. (Eq. E2.3-3)

d_e = 0.7d-1.5t (Eq. E2.3-5)

= 0.7(0.5)-1.5(0.06) = 0.260 in.

$P_n = [\,\pi(0.26)^2/4+(1.5)(0.26)\,]\,[\,0.75(60)\,]$

= 19.94 kips

ϕP_n = 0.60(19.94) = 11.96 kips.

OR

ϕ = 0.60

$P_n = 2.5tF_u(0.25L+0.96d_a)$ (Eq. E2.3-2)

= 2.5(0.06)(65) [0.25(1.5)+0.96(0.44)] = 7.77 kips

ϕP_n = 0.60(7.77) = 4.66 kips (control)

ϕP_n = 4.66 kips > F = 4.10 kips OK

2. Determine minimum edge distance in line of force.

a. F_u/F_y = 65/50 = 1.3 > 1.15

For F_u/F_y > 1.15, ϕ = 0.70

$e_{min} = P_u/\phi F_u t$ (Eq. E2.2-6)

= 4.1/ [(0.70)(65)(0.06)] = 1.50 in.

1.50 in. edge distance = e_{min}. = 1.50 in. OK

b. Edge distance shall not be less than 1.5d.

1.5d = 1.5(0.50) = 0.75 in. < 1.50 in. OK

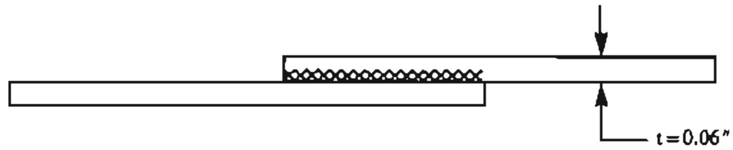
c. Clear distance between weld and end of member shall not be less than 1.0d.

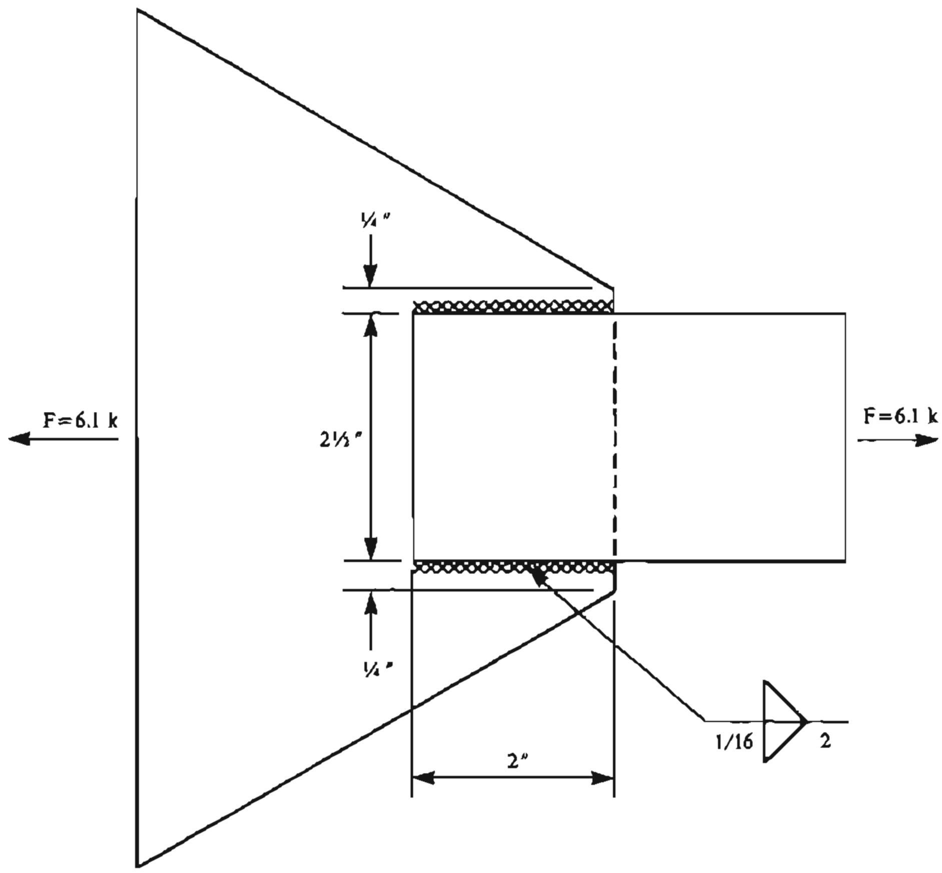
1.0d = 1.0(0.50) = 0.50 in. Clear distance = 1.50-0.25 = 1.25 in. > 0.50 in. OK

3. Use arc seam welded connection per sketch with E60 electrode and d = 1/2 in.

EXAMPLE NO. 21

FLAT SECTION WITH LAP FILLET WELDED CONNECTION





Given:

1. Steel: $F_y = 50$ ksi, $F_u = 65$ ksi.
2. Total Required Strength, F = 6.1 kips
3. Detail of connection shown in sketch.

Required:

Check to see if longitudinal fillet welded connection is adequate to transmit F = 6.1 kips.

Solution:

1. $L/t = 2/0.06 = 33.33 > 25$

   For $L/t \geq 25$,

   $\phi \quad = 0.55$

   $P_n \quad = 0.75tLF_u$ (Eq. E2.4-2)

$= 0.75(0.06)(2)(65) = 5.85$ kips

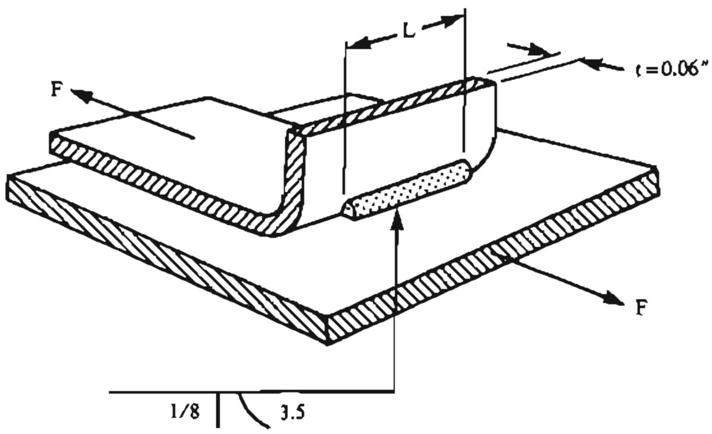
$\phi P_n = 0.55(5.85) = 3.22$ kips/weld

2. Note: t = 0.06 in. < 0.150 in.. Therefore, (Eq. E2.4-4)does not apply.

3. (3.22 kips/weld)(2 welds) = 6.44 kips < 6.1 kips OK

EXAMPLE NO. 22

FLAT SECTION WITH SINGLE FLARE BEVEL GROOVE WELDED CONNECTION



Given:

1. Steel: $F_y = 50$ ksi, $F_u = 65$ ksi.
2. Total Required Strength, F = 6.1 kips
3. Detail of connection shown in sketch.
4. Transverse loading.

Required:

Design the welded connection to transfer F = 6.1 kips.

Solution:

1. For flare-bevel groove welds, transverse loading, the required strength F shall not exceed ϕP_n

$\phi = 0.55$

$P_n = 0.833tLF_u$ (Eq. E2.5-1)

Solve for L

$L = F/0.833$

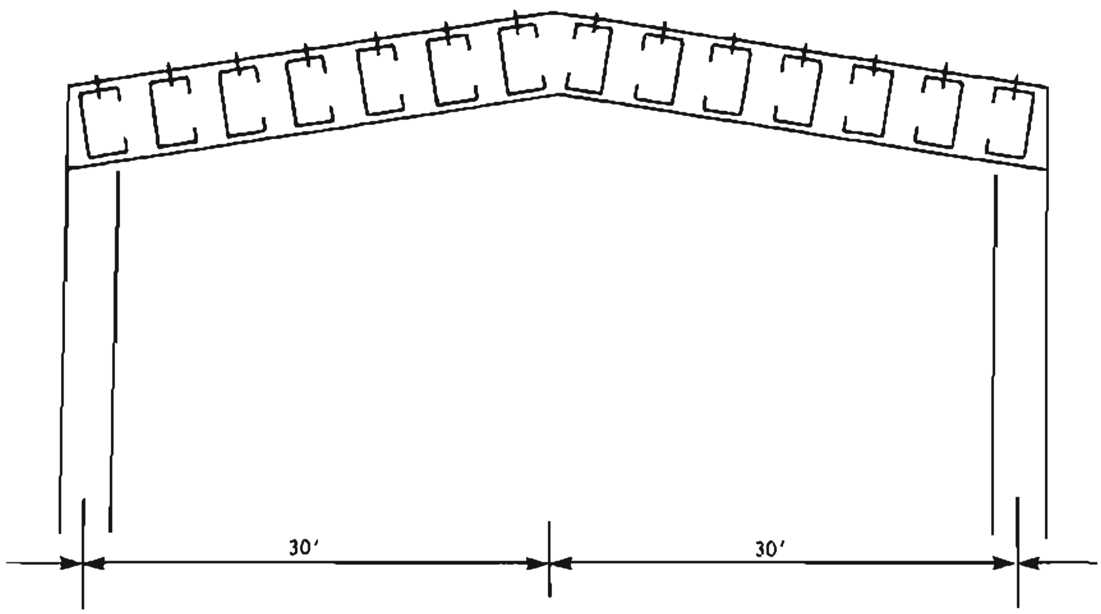
$\phi_t F_u = 6.1/[0.833(0.55)(0.06)(65)] = 3.41$ in.

2. Use 3.5 in. long flare bevel groove weld per sketch.

3. Size of weld 1/8 in. (1/16 in. min)

EXAMPLE NO. 23

C-SECTION BRACING UNDER GRAVITY LOADING



Given:

1. Steel: $F_y = 50$ ksi.
2. 60 ft. wide building, 20 ft. bays, simply supported purlins on 5-foot centers.
3. Roof slope 1:12.
4. Same channel as in ex. # 15.
5. Dead load = 3 psf; live load = 15 psf

Required:

Design of bracings of the roof system under gravity loads, using Section D3.2.1 of Specification.

Solution:

$$P = 0.05W$$

$$W = (1.2\text{x}3 + 1.6\text{x}15)\text{x}30\text{x}20 = 16560 \text{ lbs.}$$

$P = 0.05(16560) = 828$ lbs

A total restraint force of 828 lbs. must be supplied in each bay. It is up the designer to decide what devices can be used to supply this force.

EXAMPLE NO. 24

Z-SECTION BRACING UNDER GRAVITY LOADING

Given:

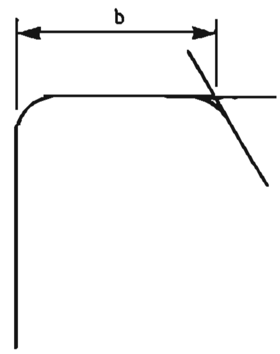
Same roof system as Example 23, with Z-section instead of channel.

Z-section is the one used in Example 4.

Required:

Design of bracings of the roof system under gravity loads, using Section D3.2.1 of Specification.

Solution:



$b = (3/32+0.06)(\tan 22.5°)+1.625 = 1.689$ in.

$d = 6.000$ in.

$t = 0.060$ in.

$n_p = 7$

$\theta = 4.76°$, $\sin\theta = 0.083$

$W = (1.2x3+1.6x15)x30x20 = 16560$ lbs.

1. Single span system with restraints at supports:

$P_L = 0.5\ [0.220b^{1.50}/(n_p^{0.72}d^{0.90}t^{0.60}) - \sin\theta]W$ (Eq. D3. 2.1-1)

$P_L = 0.5[0.22(1.689)^{1.5}/(7^{0.72}x6.000^{0.9}x0.060^{0.6}) - 0.083]16560$

$P_L = 373$ lbs.

2. Single span system with third point restraints:

$P_L = 0.5[0.474b^{1.22} / (n_p^{0.57}d^{0.89}t^{0.33}) - \sin\theta]W$ (Eq. D3.2.1-2)

$P_L = 0.5[0.474(1.689)^{1.22}/(7^{0.57}x6.000^{0.89}x0.060^{0.33}) - 0.083]16560$

$P_L = 573$ lbs.

3. Single span system with midspan restraint:

$P_L = [0.224b^{1.32}/(n_p^{0.65}d^{0.83}t^{0.50}) - \sin\theta]W$ (Eq. D3.2.1-3)

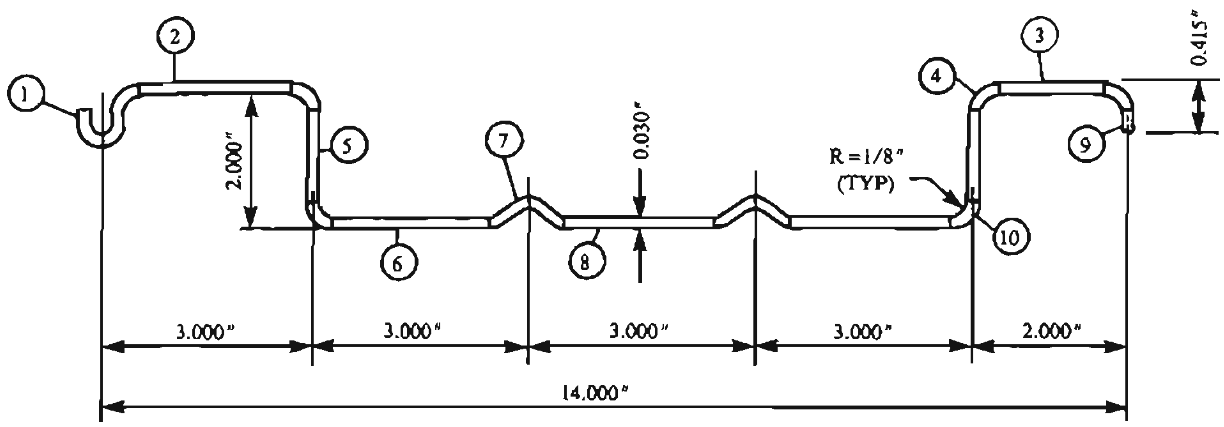
$P_L = [0.224(1.689)^{1.32}/(7^{0.65}x6.000^{0.83}x0.060^{0.50}) - 0.083]16560$

$P_L = 553$ lbs.

Each brace will be designed to resist one of the P_L forces, determined above, depending on the restraint condition of the span.

EXAMPLE NO. 25

WALL PANEL



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: Shown in sketch above.
3. Dead load to live load ratio D/L = 1/5 and 1.2D + 1.6L governs the design.

Required:

Section properties for positive and negative bending.

Solution:

1. Linear Properties.

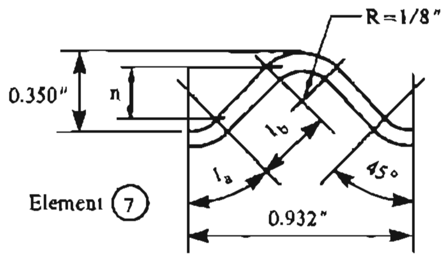
   Elements 4 and 10

   90° corners, $r = R + t/2 = 0.125 + 0.030/2 = 0.140$ in.

   Length of arc, $u = 1.57r = 1.57 \text{x} 0.140 = 0.220$ in.

   Distance of c.g. from center of radius,

$c_1 = 0.637r = 0.637x0.140 = 0.089$ in.



Element 7:

$r = 0.140$ in., $\theta = 45° = 0.785$ rad.

$c_1 = r\sin\theta/\theta = 0.140x0.707/0.785 = 0.126$ in.

$n = 0.350\text{-}2x0.140(1\text{-}\cos45°) = 0.350\text{-}0.082 = 0.268$ in.

$l_b = 0.268/\sin45° = 0.379$ in.

$l_a = \theta r = 0.785x0.140 = 0.110$ in.

$I'(\text{straight portions}) = 2x1/12xl_bxn^2$

$= 2x1/12x0.379x0.268^2 = 0.0045$ in.^{3}

$I'(\text{arcs}) = 4x0.110x(0.350/2\text{-}0.140\text{+}0.126)^2 = 0.0114$ in.^{3}

$I' \geq I'(\text{straight portions}) + I'(\text{arcs}) = 0.0045\text{+}0.0114 = 0.0159$ in.^{3}

$I \geq I't = 0.0159x0.030 = 0.000477$ in.^{4}

Check adequacy of intermediate stiffener according to Section B5.

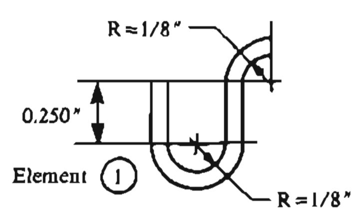
For $w/t = (3.000\text{-}0.140\text{-}0.932/2)/0.030 = 79.8$ (Element 6)

$I_{min} = [\,3.66\sqrt{(w/t)^2-(0.136E/F_y)}\,]\,t^4$ (Eq. B5-1)

$= [\,3.66\sqrt{(79.8)^2-(0.136x29500/50)}\,]\,(0.030)^4$

$= 0.000235$ in.$^4 < 0.000477$ in.^{4} satisfactory

$L_{sf} = 4l_a\text{+}2l_b = 0.440\text{+}0.758 = 1.198$ in.



Element 1

	L Length (in.)	y Distance from Center of Top Flange (in.)	Ly ($in.^2$)	Ly^2 ($in.^3$)	I'_1 About Own Axis ($in.^3$)
90° Corner	0.220	0.140-0.089 = 0.051	0.011	—	—
Straight Segments	0.500	0.265	0.133	0.035	0.003
Semi-Circled	0.440	0.390+0.089 = 0.479	0.211	0.101	—
Sum	1.160		0.355	0.136	0.003

$y_{cg} = 0.355/1.160 = 0.306$ in.

$I'_x = I'_1+Ly^2-Ly^2_{cg} = 0.003+0.136-1.160\text{x}0.306^2 = 0.139-0.109 = 0.030$ in.^{3}

$I_x = I'_x t = 0.030\text{x}0.030 = 0.00090$ in.^{4}

2. Section Modulus for Load Determination - Positive Bending
Since the neutral axis will be below the center of the cross section, the compression stress will govern.

Element 1 from Section B3.1(a)

$w = 0.25$ in.

$k = 0.43$

$f = F_y$

$\lambda = (1.052/\sqrt{0.43})(0.25/0.030)\sqrt{50/29500} = 0.550$ (Eq. B2.1-4)

$\rho = 1$ for $\lambda \leq 0.673$

$b = w = 0.25$ in. (Eq. B2.1-1)

Element 2 from Section B4.2(a)

$w/t = [\,3\text{-}3(0.140)\,]/0.030 = 86 \quad f = F_y$

$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/50} = 31.091$ (Eq. B4-1)

$D/w = [\,0.25+2(0.125)+1.5(0.030)\,]/[\,3\text{-}3(0.140)\,] = 0.211$

$n = 1/3$ for $w/t > S$

$I_a = (0.030)^4\,[\,115(86/31.091)+5\,] = 0.000262$ in.^{4} (Eq. B4.2-13)

$I_s = I_1 = 0.00090$ in.^{4}

$k = 3.57(0.00090/0.000262)^{1/3}+0.43 = 5.814 > 4$ (Eq. B4.2-10)

$\lambda = (1.052/\sqrt{4})(86)\sqrt{50/29500} = 1.862$ (Eq. B2.1-4)

ρ $= (1-0.22/1.862)/1.862 = 0.474$ (Eq. B2.1-3)

b $= \rho w = 0.474\ [\ 3-3(0.140)\ ] = 1.223$ in. (Eq. B2.1-2)

A_s $= A'_s = 1.160 \text{x} 0.030 = 0.0348$ in.^{2} for $I_s \geq I_a$ (Eq. B4.2-12)

Element 9 from Section B3.2(a)

w $= 0.415-0.030-0.125 = 0.26$ in.

$k = 0.43$

$f < F_y$, use F_y as conservative value

λ $= (1.052/\sqrt{0.43})(0.26/0.030)\sqrt{50/29500} = 0.572$ (Eq. B2.1-4)

ρ $= 1$ for $\lambda \leq 0.673$

b $= w = 0.26$ in. (Eq. B2.1-1)

Element 3 from Section B4.2(a)

w/t $= [2-2(0.140)\ ]/0.030 = 57.333,\ f = F_y$

S $= 1.28\sqrt{E/f} = 1.28\sqrt{29500/50} = 31.091$ (Eq. B4-1)

D/w $= [0.415-0.5(0.030)\ ]\ /\ [\ 2-2(0.140)\ ] = 0.233$

n $= 1/3$ for $w/t > S$

I_a $= (0.030)^4\ [\ 115(57.333/31.091)+5\ ] = 0.000176$ in.^{4} (Eq. B4.2-13)

I_s $= bh^3/12 = (0.030)(0.415-0.125-0.030)^3/12 = 0.000044$ in.^{4}

k $= 3.57(0.000044/0.000176)^{1/3}+0.43 = 2.679 < 4$ (Eq. B4.2-10)

λ $= (1.052/\sqrt{2.679})(57.333)\sqrt{50/29500} = 1.517$ (Eq . B2.1-4)

ρ $= (1-0.22/1.517)/1.517 = 0.564$ (Eq. B2.1-3)

b $= \rho w = 0.564\ [\ 2-2(0.140)\ ] = 0.970$ in. (Eq. B2.1-2)

d_s $= (I_s/I_a)d'_s$ (Eq. B4.2-11)

$= (0.000044/0.000176)(0.26) = 0.065$ in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly ($in.^2$)	Ly^2 ($in.^3$)	I'_1 About Own Axis ($in.^3$)
1	1.160	0.321	0.372	0.120	0.030
2	1.223	0.015	0.018	—	—
3	0.970	0.015	0.015	—	—
4	0.660	0.066	0.044	0.003	—
5	3.440	1.015	3.492	3.544	0.848
6	4.788	2.015	9.648	19.440	—
7	2.396	1.840	4.409	8.112	—
8	2.068	2.015	4.167	8.397	—
9	0.065	0.188	0.012	0.002	—
10	0.440	1.964	0.864	1.697	—
Sum	17.210		23.041	41.315	0.878

y_{cg} = Ly/L = 23.041/17.210 = 1.339 in.

I'_x = $Ly^2 + I'_1 - Ly^2_{cg}$

= $41.315 + 0.878 - 17.210(1.339)^2 = 11.337$ $in.^3$

I_x = $I'_x t$

= $11.337(0.030) = 0.340$ $in.^4$

$S_x = I_x/y_{cg} = 0.340/1.339 = 0.254$ $in.^3$

M_n = $S_e F_y = 0.254(50) = 12.70$ kip-in. (Eq. C3.1.1-1)

ϕ_b = 0.95

$\phi_b M_n = 0.95 \times 12.70 = 12.07$ kip-in.

Element 5 from Section B2.3(a)

y_{cg} = 1.339 in.

f_1 = [(1.339-0.125-0.030)/1.339] (50) = 44.212 ksi

f_2 = - [(2.030-0.125-0.030-1.339)/1.339] (50) = -20.015 ksi

ψ = f_2/f_1 = -20.015/44.212 = -0.453

k = $4+2(1-\psi)^3+2(1-\psi)$ (Eq. B2.3-4)

= $4+2\,[\,1-(-0.453)\,]^3 +2\,[\,1-(-0.453)\,] = 13.041$

λ = $(1.052/\sqrt{k})(w/t)\sqrt{f/E}$, $f = f_1$ (Eq. B2.1-4)

= $(1.052/\sqrt{13.041})\{$ [2.030

$-2(0.155)\,]/0.030\}\sqrt{44.212/29500} = 0.647$

For $\lambda = 0.647 < 0.673$

$b_e = w = 1.720$ in. (Eq. B2.1-1)

$b_2 = b_e/2$ (Eq. B2.3-2)

$= 1.720/2 = 0.860$ in.

$b_1 = b_e/(3-\psi)$ (Eq. B2.3-1)

$= 1.720/[\,3-(-0.453)\,] = 0.498$ in.

$w_c = 1.339-0.030-0.125 = 1.184$ in.

$b_1+b_2 = 0.498+0.860 = 1.358$ in. $> w_c = 1.184$ in.

Thus element 5 is fully effective so properties above are correct.

3. Moment of Inertia for Deflection Determination - Positive Bending

$\phi_b M_n = 1.2M_{DL} + 1.6M_{LL}$

$= [1.2(M_{DL}/M_{LL})+1.6\,]\,M_{LL}$

$= [1.2(1/5)+1.6\,]\,M_{LL}$

$= 1.84M_{LL}$

$M_{LL} = \phi_b M_n/1.84 = 12.07/1.84 = 6.56$ kip-in.

$M_s = M_{DL} + M_{LL}$

$= (1/5+1)M_{LL}$

$= 1.2(6.56) = 7.87$ kip-in.

where

M_{DL} = Moment determined on the basis of nominal dead load

M_{LL} = Moment determined on the basis of nominal live load

Computation of I_{eff}, first approximation:

* Assume a compressive stress of f = 30 ksi in the top fibers of the section.
* Since the web was fully effective at a higher stress gradient, it will be fully effective at this stress level.

Element 2 from Section B4.2(b)

$w = 3-3(0.140) = 2.580$ in.

$\lambda_c = 0.256+0.328(2.580/0.030)\sqrt{50/29500} = 1.417$ (Eq. B2.1-10)

$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/30} = 40.138$ (Eq. B4-1)

$I_a = (0.030)^4\,[115(86/40.138)+5\,]\;0.000204$ in.^{4} (Eq. B4.2-13)

$k = 3.57(0.00090/0.000204)^{1/3}+0.43 = 6.282 > 4$ (Eq. B4.2-10)

$k \quad = 4$

$\lambda \quad = (1.052/\sqrt{4})(86)\sqrt{30/29500} = 1.443$ (Eq. B2.1-4)

$\rho \quad = (0.41+0.59\sqrt{50/30} - 0.22/1.443)/1.443 = 0.706$ (Eq. B2.1-9)

$b \quad = \rho w = 0.706(2.580) = 1.821$ in. (Eq. B2.1-2)

$A_s \quad = A'_s = 0.0348$ in.^{2} (Eq. B4.2-12)

Element 3 from Section B4.2(b)

$w \quad = 2-2(0.140) = 1.720$ in.

$\lambda_c \quad = 0.256+0.328(1.720/0.030)\sqrt{50/29500} = 1.030$ (Eq. B2.1-10)

$S \quad = 1.28\sqrt{E/f} = 1.28\sqrt{29500/30} = 40.138$ (Eq. B4-1)

$I_a \quad = (0.030)^4\,[115(57.333/40.138)+5] = 0.000137$ in.^{4} (Eq. B4.2-13)

$k \quad = 3.57(0.000044/0.000137)^{1/3}+0.43 = 2.875 < 4$ (Eq. B4.2-10)

$k \quad = 2.875$

$\lambda \quad = (1.052/\sqrt{2.875})(57.333)\sqrt{30/29500} = 1.134$ (Eq. B2.1-4)

$\rho \quad = (0.41+0.59\sqrt{50/30} - 0.22/1.134)/1.134 = 0.862$ (Eq. B2.1-9)

$b \quad = \rho w = 0.862(1.720) = 1.483$ in. (Eq. B2.1-2)

$d_s \quad = (I_s/I_a)d'_s$ (Eq. B4.2-11)

$\quad = (0.000044/0.000137)(0.26) = 0.084$ in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
1	1.160	0.321	0.372	0.120	0.030
2	1.821	0.015	0.027	—	—
3	1.483	0.015	0.022	—	—
4	0.660	0.066	0.044	0.003	—
5	3.440	1.015	3.492	3.544	0.848
6	4.788	2.015	9.648	19.440	—
7	2.396	1.840	4.409	8.112	—
8	2.068	2.015	4.167	8.397	—
9	0.084	0.197	0.017	0.003	—
10	0.440	1.964	0.864	1.697	—
Sum	18.340		23.062	41.316	0.878

y_{cg} = Ly/L = 23.062/18.340 = 1.257 in.

I'_x = $Ly^2 + I'_1 - Ly^2_{cg}$

= $41.316 + 0.878 - 18.340(1.257)^2 = 13.216$ in.^{3}

I_x = $I'_x t$ = 13.216(0.030) = 0.396 in.^{4}

S_x = I_x/y_{cg} = 0.396/1.257 = 0.315 in.^{3}

M = $S_e f$ = 0.315(30) = 9.45 kip-in. > M_s = 7.87 kip-in.

To determine compression stress f in the top fibers of the section at M = 7.87 kip-in., extrapolate using

M = 12.70 kip-in., f = 50 ksi

M = 9.45 kip-in., f = 30 ksi

M = 7.87 kip-in., f = ?

(12.70-9.45)/(50-30) = (9.45-7.87)/(30-f)

0.1625(30-f) = 1.58

f = (0.1625x30-1.58)/0.1625 = 20.28 ksi

For the second approximation, assume a compression stress of f = 20.28 ksi in the top fibers of the section.

Element 2 from Section B4.2(b)

w = 2.580 in.

λ_c = 1.417

$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/20.28} = 48.819$ (Eq. B4-1)

$I_a = (0.030)^4\,[\,115(86/48.819)+5\,] = 0.000168$ in.^{4} (Eq. B4.2-13)

$k = 3.57(0.00090/0.000168)^{1/3}+0.43 = 6.677 > 4$ (Eq. B4.2-10)

$k = 4$

$\lambda = (1.052/\sqrt{4})(86)\sqrt{20.28/29500} = 1.186 < \lambda_c$ (Eq. B2.1-4)

$\rho = (1.358\text{-}0.461/1.186)/1.186 = 0.817$ (Eq. B2.1-8)

$b = \rho w = 0.817(2.580) = 2.108$ in. (Eq. B2 .1-2)

$A_s = A'_s = 0.0348$ in.^{2} (Eq. B4.2-12)

Element 3 from Section B4.2(b)

$w = 1.720$ in.

$\lambda_c = 1.030$

$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/20.28} = 48.819$ (Eq. B4-1)

$I_a = (0.030)^4\,[115(57.333/48.819)+5\,] = 0.000113$ in.^{4} (Eq. B4.2-13)

$k = 3.57(0.000044/0.000113)^{1/3}+0.43 = 3.037 < 4$ (Eq. B4.2-10)

$k = 3.037$

$\lambda = (1.052/\sqrt{3.037})(57.333)\sqrt{20.28/29500} = 0.907 < \lambda_c$ (Eq. B2.1-4)

$\rho = (1.358\text{-}0.461/0.907)/0.907 = 0.937$ (Eq. B2.1-8)

$b = \rho w = 0.937(1.720) = 1.612$ in. (Eq. B2.1-2)

$d_s = (I_s/I_a)d'_s$ (Eq. B4.2-11)

$d = (0.000044/0.000113)(0.26) = 0.101$ in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
1	1.160	0.321	0.372	0.120	0.030
2	2.108	0.015	0.032	—	—
3	1.612	0.015	0.024	—	—
4	0.660	0.066	0.044	0.003	—
5	3.440	1.015	3.492	3.544	0.848
6	4.788	2.015	9.648	19.440	—
7	2.396	1.840	4.409	8.112	—
8	2.068	2.015	4.167	8.397	—
9	0.101	0.206	0.021	0.004	—
10	0.440	1.964	0.864	1.697	—
Sum	18.773		23.073	41.317	70.878

y_{cg} $= Ly/L = 23.073/18.773 = 1.229$ in.

I'_x $= Ly^2 + I'_1 - Ly^2_{cg}$

$= 41.317 + 0.878 - 18.773(1.229)^2 = 13.839$ in.^{3}

I_x $= I'_x t$

$= 13.839(0.030) = 0.415$ in.^{4}

S_x $= I_x/y_{cg} = 0.415/1.229 = 0.338$ in.^{3}

M $= S_e f = 0.338(20.28) = 6.85$ kip-in. $< M_s = 7.87$ kip-in.

To determine compression stress f in the top fibers of the section at M = 7.87 kip-in., interpolate using

M $= 9.45$ kip-in., $f = 30$ ksi

M $= 6.85$ kip-in., $f = 20.28$ ksi

M $= 7.87$ kip-in., $f = ?$

$(9.45\text{-}6.85)/(30\text{-}20.28) = (7.87\text{-}6.85)/(f\text{-}20.28)$

$0.2675(f\text{-}20.28) = 1.02$

$f = (1.02+0.2675 \times 20.28)/0.2675 = 24.09$ ksi

For the third approximation, assume a compression stress of f = 24.09 ksi in the top fibers of the section.

Element 2 from Section B4.2(b)

w $= 2.580$ in.

λ_c = 1.417

S = $1.28\sqrt{E/f} = 1.28\sqrt{29500/24.09} = 44.792$ (Eq. B4-1)

I_a = $(0.030)^4\,[115(86/44.792)+5] = 0.000183$ in.^{4} (Eq. B4.2-13)

k = $3.57(0.00090/0.000183)^{1/3}+0.43 = 6.501 > 4$ (Eq. B4.2-10)

k = 4

λ = $(1.052/\sqrt{4})(86)\sqrt{24.09/29500} = 1.293 < \lambda_c$ (Eq. B2.1-4)

ρ = $(1.358-0.461/1.293)/1.293 = 0.775$ (Eq. B2.1-8)

b = $\rho w = 0.775(2.580) = 2.000$ in. (Eq. B2.1-2)

A_s = $A'_s = 0.0348$ in.^{2} (Eq. B4.2-12)

Element 3 from Section B4.2(b)

w = 1.720 in.

λ_c = 1.030

S = $1.28\sqrt{E/f} \sim= 1.28\sqrt{29500/24.09} = 44.792$ (Eq. B4-1)

I_a = $(0.030)^4\,[115(57.333/44.792)+5] = 0.000123$ in.^{4} (Eq. B4.2-13)

k = $3.57(0.000044/0.000123)^{1/3}+0.43 = 2.964 < 4$ (Eq. B4.2-10)

k = 2.964

λ = $(1.052/\sqrt{2.964})(57.333)\,s\sqrt{24.09/29500} = 1.001 < \lambda_c$ (Eq. B2.1-4)

ρ = $(1.358-0.461/1.001)/1.001 = 0.897$ (Eq. B2.1-8)

b = $\rho w = 0.897(1.720) = 1.543$ in. (Eq. B2.1-2)

d_s = $(I_s/I_a)d'_s$ (Eq. B4.2-11)

d = $(0.000044/0.000123)(0.26) = 0.093$ in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly ($in.^2$)	Ly^2 ($in.^3$)	I'_1 About Own Axis ($in.^3$)
1	1.160	0.321	0.372	0.120	0.030
2	2.000	0.015	0.030	—	—
3	1.543	0.015	0.023	—	—
4	0.660	0.066	0.044	0.003	—
5	3.440	1.015	3.492	3.544	0.848
6	4.788	2.015	9.648	19.440	—
7	2.396	1.840	4.409	8.112	—
8	2.068	2.015	4.167	8.397	—
9	0.093	0.202	0.019	0.004	—
10	0.440	1.964	0.864	1.697	—
Sum	18.588		23.068	41.317	0.878

y_{cg} = $Ly/L = 23.068/18.588 = 1.241$ in.

$I'_x = Ly^2 + I'_1 - Ly^2_{cg}$

$= 41.317 + 0.878 - 18.588(1.241)^2 = 13.568\ in.^3$

$I_x = I'_x t$

$= 13.568\ (0.030) = 0.407\ in.^4$

$S_x = I_x/y_{cg} = 0.407/1.241 = 0.328\ in.^3$

$M = S_e f = 0.328(24.09) = 7.90$ kip-in. $\approx M_s = 7.87$ kip-in.

Therefore

$I_x = 0.407\ in.^4$

4. Section Modulus for Load Determination - Negative Bending
Since the N.A. may be closer to the compression flange than to the tension flange, the compression stress is unknown, and therefore the effective width of the compression flange and section properties must be determined by an iterative method.

Elements 1, 2, 3, 4, 5, 9, and 10 do not vary with stress level. Assume compression stress will govern, i.e., $f = F_y = 50$ ksi in the bottom compression fibers of the section.

Element 6 from Section B5(d)

$w/t = [\,3\text{-}0.140\text{-}0.5(0.932)\,]/0.030 = 79.8 > 60$

$k = 4$

$\lambda = (1.052/\sqrt{4})(79.8)\sqrt{50/29500} = 1.728$ (Eq. B2.1-4)

$\rho = (1\text{-}0.22/1.728)/1.728 = 0.505$ (Eq. B2.1-3)

$b = \rho w = 0.505(2.394) = 1.209$ in. Since $w/t > 60$, (Eq. B2.1-2)

$b_e = 0.030\,[(1.209/0.030)-0.10(79.8-60)\,] = 1.150$ in. (Eq. B5-3)

Element 7 from Section B5(d)

$60 < w/t = 79.8 < 90$

$\alpha = 3-[2(1.150)/2.394\,]$

$-\{(1/30)\,[\,1-(1.150/2.394)\,]\,(79.8)\} = 0.657$ (Eq. B5-5)

$L_{ef} = \alpha L_{st} = 0.657(2.396) = 1.574$ in. (Eq. B5-4)

Element 8 from Section B5(d)

$w/t = (3-0.932)/0.030 = 68.933 > 60$

$k = 4$

$\lambda = (1.052/\sqrt{4})(68.933)\sqrt{50/29500} = 1.493$ (Eq. B2.1-4)

$\rho = (1-0.22/1.493)/1.493 = 0.571$ (Eq. B2.1-3)

$b = \rho w = 0.571(2.068) = 1.181$ in. (Eq. B2.1-2)

$b_e = 0.030\,[(1.181/0.030)-0.10(68.933-60)] = 1.154$ in. (Eq. B5-3)

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
1	1.160	0.321	0.372	0.120	0.030
2	2.580	0.015	0.039	0.001	—
3	1.720	0.015	0.026	—	—
4	0.660	0.066	0.044	0.003	—
5	3.440	1.015	3.492	3.544	0.848
6	2.300	2.015	4.635	9.339	—
7	1.574	1.840	2.896	5.329	—
8	1.154	2.015	2.325	4.685	—
9	0.260	0.285	0.074	0.021	—
10	0.440	1.964	0.864	1.697	—
Sum	15.288		14.767	24.739	0.878

$y_{cg} = Ly/L = 14.767/15.288 = 0.966$ in.

$y_c = 2.030-0.966 = 1.064$ in. $> 2.030/2 = 1.015$ in.

Therefore, compression stress controls as assumed.

$I'_x = Ly^2 + I'_1 - Ly^2_{cg} = 24.739 + 0.878 - 15.288(0.966)^2 = 11.351\ \text{in.}^3$

$I_x = I'_x t$

$= 11.351(0.030) = 0.341\ \text{in.}^4$

$S_x = I_x/y_{cg} = 0.341/1.064 = 0.320\ \text{in.}^3$

$M_n = S_e F_y = 0.320(50) = 16.00$ kip-in. (Eq. C3.1.1-1)

$\phi_b = 0.95$

$\phi_b M_n = 0.95 \times 16.00 = 15.20$ kip-in.

Element 5 from Section B2.3(a)

$f_1 = [(1.064-0.125-0.030)/1.064\ ]\ (50) = 42.716$ ksi

$f_2 = -\ [(0.966-0.125-0.030)/1.064\ ]\ (50) = -38.111$ ksi

$\psi = f_2/f_1 = -38.111/42.716 = -0.892 < -0.236$

$k = 4+2(1-\psi)^3+2(1-\psi)$ (Eq. B2.3-4)

$= 4+2\ [\ 1-(\ -0.892)\ ]^3 + 2\ [\ 1-(-0.892)\ ] = 21.329$

$\lambda = (1.052/\sqrt{k}\ )(w/t)\sqrt{f/E}\ , f = f_1$ (Eq. B2.1-4)

$= (1.052/\sqrt{21.329}\ )\{\ [\ 2.030$

$-2(0.155)\ ]\ /0.030\}\ \sqrt{42.716/29500} = 0.497$

For $\lambda = 0.497 < 0.673$

$b_e = w = 1.720$ in. (Eq. B2.1-1)

$b_2 = b_e/2$ (Eq. B2.3-2)

$= 1.720/2 = 0.860$ in.

$b_1 = b_e/(3-\psi)$ (Eq. B2.3-1)

$= 1.720/\ [3-(-0.892)\ ] = 0.442$ in.

$w_c = 1.064-0.030-0.125 = 0.909$ in.

$b_1+b_2 = 0.442+0.860 = 1.302$ in. $> w_c = 0.909$ in.

Thus element 5 is fully effective so properties above are correct.

5. Moment of Inertia for Deflection Determination - Negative Bending

$\phi_b M_n = 1.2M_{DL} + 1.6M_{LL}$

$= [\ 1.2(M_{DL}/M_{LL}) + 1.6\ ]\ M_{LL}$

$= [\ 1.2(1/5) + 1.6\ ]\ M_{LL}$

$= 1.84M_{LL}$

$M_{LL} = \phi_b M_n / 1.84 = 15.20/1.84 = 8.26$ kip-in.

$M_s = M_{DL} + M_{LL}$

$= (1/5+1)M_{LL}$

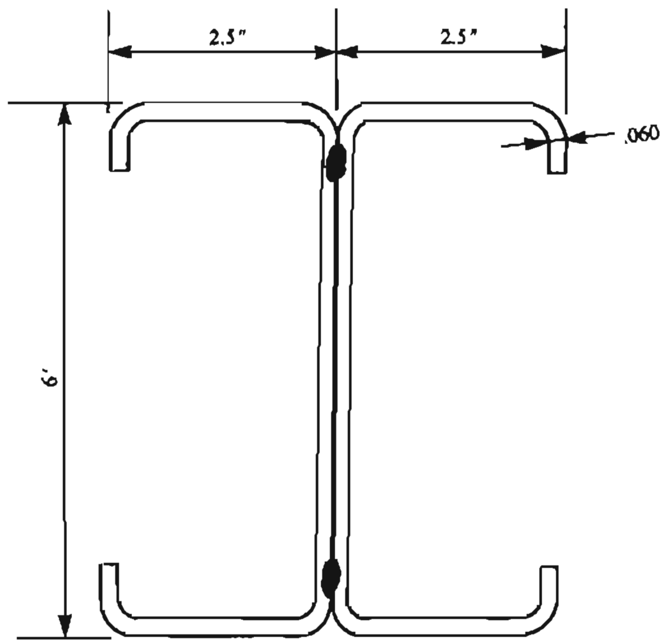
$= 1.2(8.26) = 9.91$ kip-in.

For deflection determination, the procedure is iterative: one assume the actual compressive stress f under this service moment M_s. Knowing f, one proceeds as usual to obtain S_e and checks to see if (f x S_e) is equal to M_s as it should. If not, reiterate until one obtains the desired level of accuracy.

(Refer to Example No. 8 for example of procedure to follow.)

EXAMPLE NO. 26

BUILT-UP SECTION - CONNECTING TWO CHANNELS



Given:

1. Steel: $F_y = 50$ ksi.
2. Sections: 2 - 6 x 2.5 x 0.060 channels with stiffened flanges.

Required:

1. Determine the maximum permissible longitudinal spacing of connectors joining two channels to form an I-section used as a compression member with unbraced length of 12 ft.
2. Design resistance welds connecting the two channels to form an I-section used as a beam with the following load, span, and support conditions:

   a. Span: 10′-0"

   b. Total uniformly distributed factored load including factored dead load: 0.520 kips per lin. ft.

   c. Length of bearing at end support: 3 in.

Solution:

1. Maximum longitudinal spacing of connectors for compression member [Section D1.1(a)]

   For compression members, the maximum permissible longitudinal spacing of connectors is

   $s_{max} = Lr_{cy}/(2r_I)$ (Eq. D1.1-1)

$= (12x12)x0.909/(2x1.18) = 55.46$ in.

r_{cy} is from Table 1 and r_1 is from Table 5 of Part V of the Manual.

Therefore, the maximum spacing of connectors used for connecting these two channels as a compression member is 55 in.

2. Design resistance welds connecting the two channels to form an I-section used as a beam [Section D1.1(b)]

a. Spacing of welds between end supports:

The maximum permissible longitudinal spacing of welds for a flexural member is

$s_{max} = L/6$ (Eq. D1.1-2)

$= 12x10/6 = 20$ in.

Maximum spacing is also limited by

$s_{max} = 2gT_s/(mq)$ (Eq. D1.1-3)

in which

$g = 5.0$ in. (assumed for 6 in. deep section)

$T_s = 0.65x2.28x0.25 \approx 0.371$ kips (Section E2.6)

$m = 1.148$ in. (from Table 1 of Part V of the Manual)

$q = 3x0.520/12 = 0.130$ kips per lin. in.

Therefore

$s_{max} = 2x5.0x0.371/(1.148x0.130) = 24.86$ in.

$s_{max} = L/6$ controls. Use a spacing of 20 in. throughout the span.

b. Strength of welds at end supports:Since the weld spacing is larger than the bearing length of 3.0 in., the required design strength of the welds directly at the reaction is

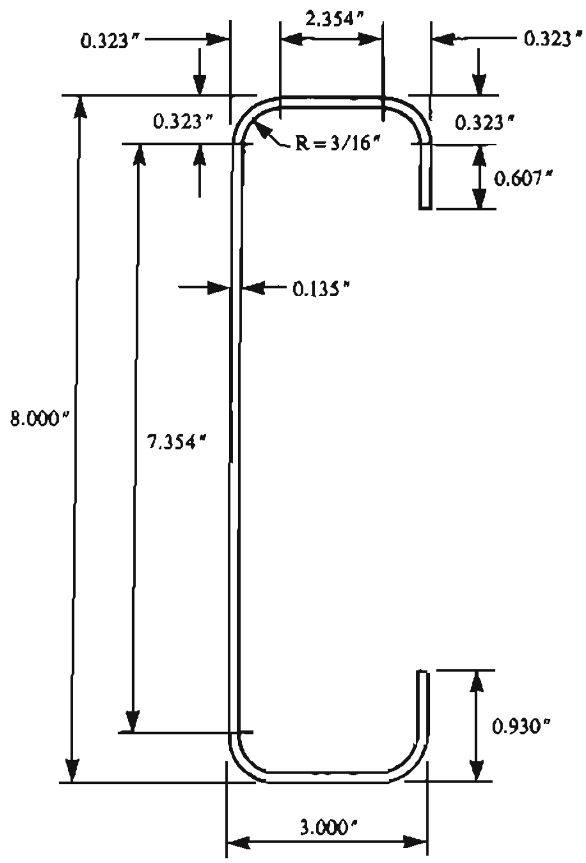
$T_s = Pm/(2g)$ (Eq. D1.1-6)

$= 0.520x5x1.148/(2x5) = 0.298$ kips

which is less than 0.370 kips as provided.

EXAMPLE NO. 27

STRENGTH INCREASE FROM COLD WORK OF FORMING



Given:

1. Steel: $F_y = 50$ ksi and $F_u = 65$ ksi.
2. Section: 8 x 3 x 0.135 channel with stiffened flanges.
3. Section to be used as a beam.

Required:

Determine the average tensile yield point of steel, F_{ya}, for the flange considering the increase in strength resulting from the cold work of forming.

Solution:

1. Check the limitations.

   a. In order to use Eq. A5.2.2-1 for computing the average tensile yield point for the beam flange, the channel section must be that the quantity ρ is unity as determined according to Section B2 for each of the component element of the section, i.e., the channel section must be fully effective (Section-A5.2.2).

Whether the channel section is fully effective or not can be determined either by computations which follow the flow charts for Section B2 provided in Part VI of the Manual (See Examples 2 and 3), or by comparision of S_x from Table 1 and S_e from Table 10 (based on $F_y = 50$ ksi) of Part V the Manual, if $S_e = S_x$ then the channel section is fully effective. The second method is used in this example.

S_e = 4.843 in.^{3} (from Table 10)

S_x = 4.843 in.^{3} (from Table 1)

Therefore, section is fully effective. Eq. A5.2.2-1 can be used to determine F_{ya}.

b. Eq. A5.2.2-2 is applicable only when $F_{uv}/F_{yv} \geq 1.2$, $R/t \leq 7$,.in and minimum included angle $\leq 120°$. Since

F_{uv}/F_{yv} = 65/50 = 1.3 > 1.2

R/t = 0.1875/0.135 = 1.389 < 7

θ = 90° < 120°

Therefore, Eq. A5.2.2-2 can be used to determine F_{yc}.

2. Determination of F_{yc}.

$B_c = 3.69(F_{uv}/F_{yv})-0.819(F_{uv}/F_{yv})^2-1.79$ (Eq. A5.2.2-3)

$= 3.69(1.3)-0.819(1.3)^2-1.79 = 1.623$

$m = 0.192(F_{uv}/F_{yv})-0.068$ (Eq. A5.2.2-4)

$= 0.192(1.3)-0.068 = 0.182$

$F_{yc} = B_cF_{yv}/(R/t)^m$ (Eq. A5.2.2-2)

$= 1.623(50)/(0.1875/0.135)^{\circ 182} = 76.44$ ksi.in

3. Determination of F_{ya}.

$r = R+t/2 = 0.1875+0.135/2 = 0.255$ in.

Cross-sectional area of corner = 1.57x0.255x0.135 = 0.054 in.^{2}

Total corner cross-sectional area of the controlling flange

= 0.054x2 = 0.108 in.^{2}

Full cross-sectional area of the controlling flange

= 0.108+(2.354x0.135) = 0.426 in.^{2}

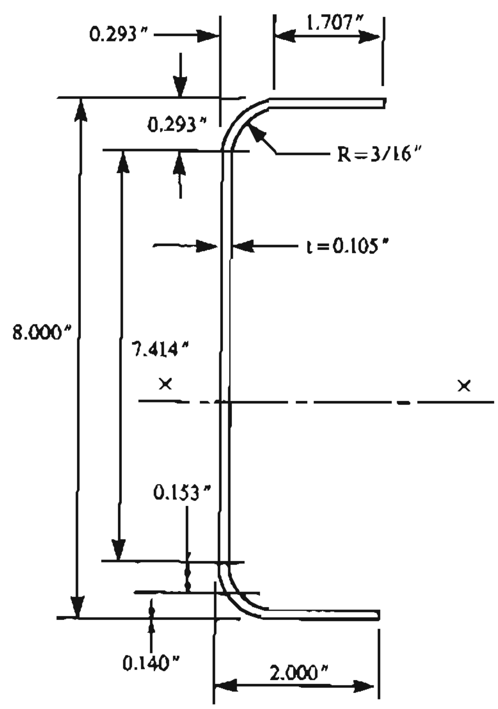
$C = 0.108/0.426 = 0.254$

$F_{ya} = F_{yc}+(1-C)F_{yf}$ (Eq. A5.2.2-1)

$= 0.254(76.44)+(1-0.254)(50) = 56.72$ ksi

EXAMPLE NO. 28

FLANGE CURLING



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: 8 x 2 x 0.105 channel with unstiffened flanges.
3. Compression flange braced against lateral buckling.
4. Dead load to live load ratio D/L = 1/5 and 1.2D + 1.6L governs the design.

Required:

Determine the amount of curling for the compression flange.

Solution:

1. Determination of the design flexural strength, $\phi_b M_n$:

   The elastic section modulus of the effective section, S_e, calculated with the extreme compression or tension fiber at F_y can be determined by either computations which follow the flow charts for Section B2 in Part VI of the Manual (see Example 1), or by taking the S_e value provided in Table 11 in Part V of the Manual. The latter method is used in this example.

$S_e = 2.304$ in.^{3}

$M_n = S_eF_y$ (Eq. C3.1.1-1)

$= 2.304 \times 50 = 115.20$ kip-in.

$\phi_b = 0.90$

$\phi_bM_n = 0.90 \times 115.20 = 103.68$ kip-in.

2. Determination of the average stress in compression flange, f_{av}, at the service moment M_s:

$\phi_bM_n = 1.2M_{DL} + 1.6M_{LL}$

$= [\ 1.2(M_{DL}/M_{LL})+1.6\ ]\ M_{LL}$

$= [\ 1.2(1/5)+1.6\ ]\ M_{LL}$

$= 1.84M_{LL}$

$M_{LL} = \phi_bM_n/1.84 = 103.68/1.84 = 56.35$ kip-in.

$M_s = M_{DL} + M_{LL}$

$= (1/5+1)M_{LL}$

$= 1.2(56.35) = 67.62$ kip-in.

where

M_{DL} = Moment determined on the basis of nominal dead load

M_{LL} = Moment determined on the basis of nominal live load

The procedure is iterative: one assumes the actual compressive stress f under this service moment M_s. Knowing f, proceeds as usual to obtain S_e and checks to see if ($f \times S_e$) is equal to M_s as it should. If not, reiterate until one obtains the desired level of accuracy.

Properties of 90° corners:

$r = R + t/2 = 3/16 + 0.105/2 = 0.240$ in.

Length of arc, $u = 1.57r = 1.57 \times 0.240 = 0.377$ in.

Distance of c.g. from center of radius,

$c = 0.637r = 0.637 \times 0.240 = 0.153$ in.

a. For the first iteration, assume a compression stress of $f = F_y/2 = 25$ ksi in the top fibers of the section and that the web is fully effective.

Compression flange: $k = 0.43$ (unstiffened compression element)

$w/t = 1.707/0.105 = 16.26 < 60$ OK (Section B1.1-(a)-(3))

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{0.43})(16.26)\sqrt{25/29500} = 0.759 > 0.673$

$\rho = w = [1-(0.22/\lambda)]/\lambda$ (Eq. B2.1-3)

$= [1-(0.22/0.759)]/0.759 = 0.936$

$b = \rho w$ (Eq. B2.1-2)

$= 0.936 \times 1.707$

$= 1.598$ in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 About Own Axis (in.^{3})
Web	7.414	4.000	29.656	118.624	33.961
Upper Corner	0.377	0.140	0.053	0.007	—
Lower Corner	0.377	7.860	2.963	23.291	—
Compression Flange	1.598	0.053	0.085	0.004	—
Tension Flange	1.707	7.948	13.567	107.832	—
Sum	11.473		46.324	249.758	33.961

Distance from top fiber to x-axis is

$y_{cg} = 46.324/11.473 = 4.038$ in.

Since distance of top compression fiber from neutral axis is greater than one half the beam depth, a compression stress of 25 ksi will control as assumed.

To check if web is fully effective (Section B2.3):

$f_1 = [(4.038-0.293)/4.038] \times 25 = 23.19$ ksi (compression)

$f_2 = -[(3.962-0.293)/4.038] \times 25 = -22.72$ ksi (tension)

$\psi = f_2/f_1 = -22.72/23.19 = -0.980$

$k = 4+2(1-\psi)^3+2(1-\psi)$ (Eq. B2.3-4)

$= 4+2[1-(-0.980)]^3+2[1-(-0.980)]$

$= 23.48$

$h = w = 7.414$ in., $h/t = w/t = 7.414/0.105 = 70.61$

$h/t = 70.61 < 200$ OK (Section B1.2-(a))

$\lambda = (1.052/\sqrt{23.48})(70.61)\sqrt{23.19/29500} = 0.430 < 0.673$

$b_e = w$ (Eq. B2.1-1)

$= 7.414$ in.

$b_2 \quad = b_e/2$ (Eq. B2.3-2)

$= 7.414/2 = 3.707$ in.

$b_1 \quad = b_e/(3-\psi)$ (Eq. B2.3-1)

$= 7.414/[\,3-(-0.980)\,] = 1.863$ in.

$b_1+b_2 = 1.863 + 3.707 = 5.570$ in.

Compression portion of the web calculated on the basis of the effective section = y_{cg} - 0.293 = 4.038 - 0.293 = 3.745 in.

Since b_1+b_2 = 5.570 in. > 3.745 in., b_1+b_2 shall be taken as 3.745 in.. This verifies the assumption that the web is fully effective.

$I'_x \quad = Ly^2+I'_1-Ly^2_{cg}$

$= 249.758 + 33.961 - 11.473(4.038)^2$

$= 96.647$ in.^{3}

Actual $I_x \quad = I'_x t$

$= 96.647 \times 0.105 = 10.148$ in.^{4}

$S_e \quad = I_x/y_{cg}$

$= 10.148/4.038$

$= 2.513$ in.^{3}

$M \quad = f \times S_e = 25 \times 2.513$

$= 62.83$ kip-in. $< M_s = 67.62$ kip-in.

Need to do another iteration and also to increase f.

b. For the second iteration, assume f = 27.18 ksi in the top fibers of the section and that the web is fully effective.

Compression flange:

$\lambda \quad = (1.052/\sqrt{0.43})(16.26)\sqrt{27.18/29500} = 0.792 > 0.673$

$\rho \quad = [1-(0.22/0.792)]/0.792 = 0.912$

$b \quad = 0.912 \times 1.707 = 1.557$ in.

Effective section properties about x-axis:

$L \quad = 11.473 - 1.598 + 1.569 = 11.444$ in.

$Ly \quad = 46.324 - 0.085 + 1.569 \times 0.053 = 46.322$ in.^{2}

$Ly^2 \quad = 249.755 - 0.004 + 1.569(0.053)^2 = 249.758$ in.^{3}

$I'_1 \quad = 33.961$ in.^{3}

y_{cg} = 46.322/11.444 = 4.047 in. greater than one half beam depth. Thus top compression fiber controls in determination of S_e.

To check if web is fully effective:

f_1 = [(4.047-0.293)/4.047] x27.18 = 25.21 ksi

f_2 = - [(3.953-0.293)/4.047] x27.18 = -24.51 ksi

ψ = -24.51/25.21 = -0.972

k = 4+2 [1-(-0.972)]^3 +2 [1-(-0.972)] = 23.28

λ = $(1.052/\sqrt{23.28})(70.61)\sqrt{25.21/29500}$ = 0.450 < 0.673

b_e = 7.414 in.

b_2 = 7.414/2 = 3.707 in.

b_1 = 7.414/ [3-(-0.972)] = 1.866 in.

Compression portion of the web calculated on the basis of the effective section = 4.047 - 0.293 = 3.754 in..

Since b_1+b_2 = 5.573 in. > 3.759 in., b_1+b_2 shall be taken as 3.759 in.. This verifies the assumption that the web is fully effective.

I'_x = 249.755 + 33.961 - 11.432(4.047)^2= 96.48 in.^{3}

Actual I_x = 96.48 x 0.105

= 10.130 in.^{4}

S_e = 10.130/4.047 = 2.503 in.^{3}

M = f x S_e = 27.18 x 2.503

= 68.034 kip-in. = M_sOK

f_{av} = f(b/w) = 27.18(1.569/1.707) = 24.98 ksi

3. Determination of the curling of the compression flange, c_f.

w_f = 2.000 - 0.105 = 1.895 in.

w_f = $\sqrt{0.061tdE/f_{av}}\ \sqrt[4]{(100c_f/d)}$ (Eq. B1.1b-1)

1.895= $\sqrt{0.061(0.105)(8)(29500)/24.98}\ \sqrt[4]{100c_f/8}$

= 7.779 $\sqrt[4]{12.5c_f}$

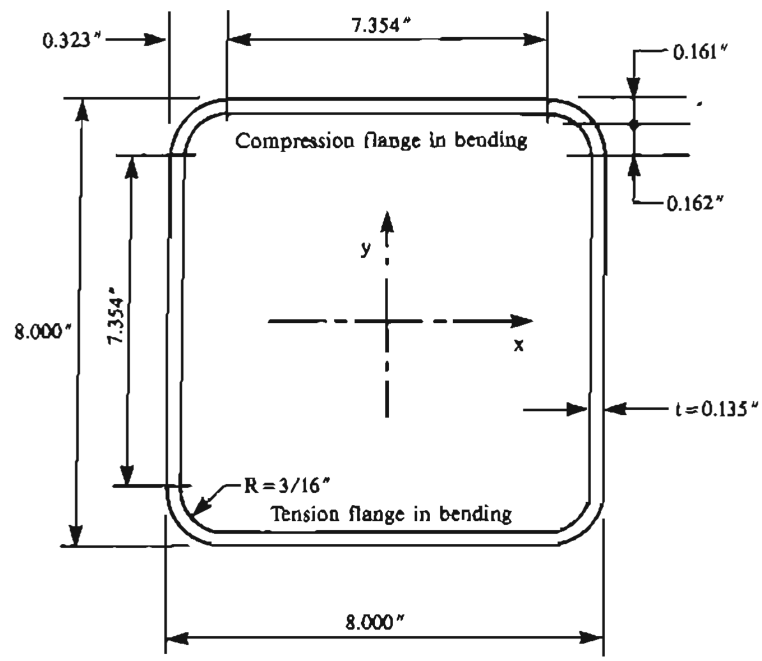
$\sqrt[4]{12.5c_f}$ = 1.895/7.779

$12.5c_f$ = (1.895/7.779)^4

c_f = (1.895/7.779)^4/12.5 = 0.00078 in.

EXAMPLE NO. 29

SHEAR LAG



Given:

1. Steel: $F_y = 50$ ksi.
2. Section: 8 x 8 x 0.135 square tube.
3. Span: L = 3 ft., with simple supports.
4. Loading: Concentrated load at midspan.

Required:

Determine the design flexural strength, $\phi_b M_n$.

Solution:

1. Determination of the nominal moment, M_n, based on initiation of yielding (Section C3.1.1).

   Properties of 90° corners:

   $r = R + t/2 = 3/16 + 0.135/2 = 0.255$ in.

Length of arc, u = 1.57r = 1.57 x 0.255 = 0.400 in.

Distance of c.g. from center of radius,

c = 0.637r = 0.637 x 0.255 = 0.162 in.

Computation of I_x:

For the first approximation, assume a compression stress of $f = F_y = 50$ ksi in the compression flange, and that the webs are fully effective.

Compression flange: k = 4.00 (stiffened compression element supported by a web on each longitudinal edge)

w/t =7.354/0.135 = 54.47 < 500 OK (Section B1.1-(a)-(2))

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$=(1.052/\sqrt{4.00})(54.47)\sqrt{50/29500} = 1.180 > 0.673$

$\rho = (1-0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1-0.22/1.180)/1.180 = 0.689$

$b = \rho w$ (Eq. B2.1-2)

= 0.689 x 7.354

= 5.067 in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly ($in.^2$)	Ly^2 ($in.^3$)	I'_1 About Own Axis ($in.^3$)
Webs	14.708	4.000	58.832	235.328	66.286
Upper Corners	0.800	0.161	0.129	0.021	—
Lower Corners	0.800	7.839	6.271	49.160	—
Compression Flange	5.067	0.068	0.345	0.023	—
Tension Flange	7.354	7.933	58.339	462.806	—
Sum	28.729		123.916	747.338	66.286

Distance from top fiber to x-axis is

$y_{cg} = 123.916/28.729 = 4.313$ in.

Since distance of top compression fiber from neutral axis is greater than one half the beam depth, a compression stress of 50 ksi will govern as assumed (i.e., initial yielding is in compression).

To check if webs are fully effective (Section B2.3):

f_1 = [(4.313-0.323)/4.313] x50 = 46.26 ksi (compression)

f_2 = - [(3.687-0.323)/4.313] x50 = -39.00 ksi(tension)

ψ = f_2/f_1 = -39.00/46.26 = -0.843

k = $4+2(1-\psi)^3+2(1-\psi)$ (Eq. B2.3-4)

= $4+2\,[\,1-(-0.843)\,]^3+2\,[\,1-(-0.843)\,]$ = 20.206

h = w = 7.354 in., h/t = w/t = 7.354/0.135 = 54.47

h/t = 54.47 < 200 OK (Section B1.2-(a))

λ = $(1.052/\sqrt{20.206})(54.47)\sqrt{46.26/29500}$ = 0.505 < 0.673

b_e = w (Eq. B2.1 -1)

= 7.354 in.

b_2 = $b_e/2$ (Eq. B2.3-2)

= 7.354/2 = 3.677 in.

b_1 = $b_e/(3-\psi)$ (Eq. B2.3-1)

=7.354/ [3-(-0.843)] = 1.914 in.

Compression portion of the web calculated on the basis of the effective section = y_{cg} - 0.323 = 4.313 - 0.323 = 3.990 in.

Since b_1+b_2 = 5.591 in. > 3.990 in., b_1+b_2 shall be taken as 3.990 in.. This verifies the assumption that the webs are fully effective.

I'_x = $Ly^2+I'_1-Ly^2_{cg}$

= $747.338 + 66.286 - 28.729(4.313)^2$

= 279.208 $in.^3$

Actual I_x = $I'_x t$

= 279.208x0.135

= 37.693 $in.^4$

S_e = I_x/y_{cg}

= 37.693/4.313

= 8.739 $in.^3$

M_n = S_eF_y = 8.739 x 50

= 436.95 kip-in.

2. Determination of the nominal moment, M_n, based on shear lag consideration (Section B1.1(c)).

w_f = (8-2x0.135)/2 = 3.865 in.

L/w_f = 3x12/3.865 = 9.314 < 30

Because the L/w_f ratio is less than 30, and the member carries a concentrated load, consideration for shear lag is needed.

From Table B1.1(c):

L/w_f = 10, effective design width/actual width = 0.73

L/w_f = 8, effective design width/actual width = 0.67

L/w_f = 9.314, effective design width/actual width = ?

$$(10-9.314)/(9.314-8) = (0.73-x)/(x-0.67)$$
$$0.686(x-0.67) = 1.314(0.73-x)$$
$$x = 0.709$$

Therefore, the effective design widths of compression and tension flanges between webs are

$0.709(8-2x0.135) = 5.481$ in.

$b = 5.481-2R = 5.481-2(3/16) = 5.106$ in.

Because of symmetry and assume webs are fully effective,

$y_{cg} = 4.000$ in.

Effective section properties about x-axis:

$L = 28.729-5.067-7.354+5.106x2 = 26.520$ in.

$Ly^2 = 747.338-0.023-462.806+5.106(0.068)^2+5.106(7.933)^2$

$= 605.866$ in.^{3}

$I'_1 = 66.286$ in.^{3}

To check if webs are fully effective:

$f_1 = [(4.000-0.323)/4.000]x50 = 45.96$ ksi

$f_2 = -45.96$ ksi

$\psi = -45.96/45.96 = -1.000$

$k = 4+2[1-(-1.000)]^3+2[1-(-1.000)] = 24.000$

$\lambda = (1.052/\sqrt{24.000})(54.47)\sqrt{45.96/29500} = 0.462 < 0.673$

$b_e = 7.354$ in.

$b_2 = 7.354/2 = 3.677$ in.

$b_1 = 7.354/[3-(-1.000)] = 1.839$ in.

Compression portion of the web calculated on the basis of the effective section = 4.000 - 0.323 = 3.677 in..

Since $b_1+b_2 = 5.516$ in. > 3.677 in., b_1+b_2 shall be taken as 3.677 in.. This verifies the assumption that the webs are fully effective.

$I'_x = 605.866 + 66.286 - 26.520(4.000)^2$

$= 247.832 \text{ in.}^3$

Actual $I_x = 247.832 \times 0.135$

$= 33.457 \text{ in.}^4$

$S_e = 33.457/4.000 = 8.364 \text{ in.}^3$

$M_n = 8.364 \times 50 = 418.20$ kip-in. < 436.95 kip-in. (initial yielding)

3. Determination of the design flexural strength, $\phi_b M_n$.

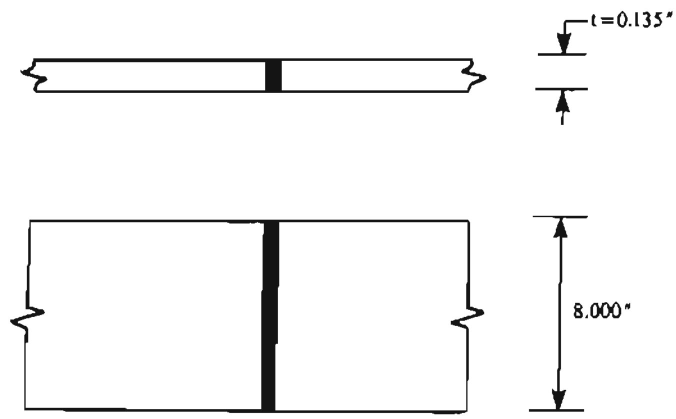
   $M_n = 418.20$ kip-in.

   $\phi_b = 0.95$

   $\phi_b M_n = 0.95 \times 418.20 = 397.29$ kip-in.

EXAMPLE NO. 30

FLAT SECTION WITH GROOVE WELDED CONNECTION IN BUTT JOINT



Given:

1. Steel: $F_y = 50$ ksi.
2. Electrode: $F_{xx} = 60$ ksi.
3. Detail of connection shown in sketch.

Required:

1. Determine the design tensile strength, ϕP_n, normal to the effective area.
2. Determine the design shear strength, ϕP_n, on the effective area.

Solution:

1. Determination of the design tensile strength, ϕP_n, normal to the effective area (Section E2.1(a)).

$$P_n = Lt_eF_y \qquad \text{(Eq. E2.1-1)}$$

$$= (8.000)(0.135)(50)$$

$$= 54.00 \text{ kips}$$

$$\phi = 0.90$$

$$\phi P_n = 0.90 \times 54.00$$

= 48.60 kips

2. Determination of the design shear strength, ϕP_n, on the effective area (Section E2.1(b))

$(P_n)_1 = Lt_e(0.6F_{xx})$ (Eq. E2.1-2)

$= (8.000)(0.135)(0.6x60)$

$= 38.88$ kips

$\phi = 0.80$

$\phi(P_n)_1 = 0.80 \times 38.88$

$= 31.10$ kips

$(P_n)_2 = Lt_e(F_y/\sqrt{3})$ (Eq. E2.1-3)

$= (8.000)(0.135)(50/\sqrt{3})$

$= 31.18$ kips

$\phi = 0.90$

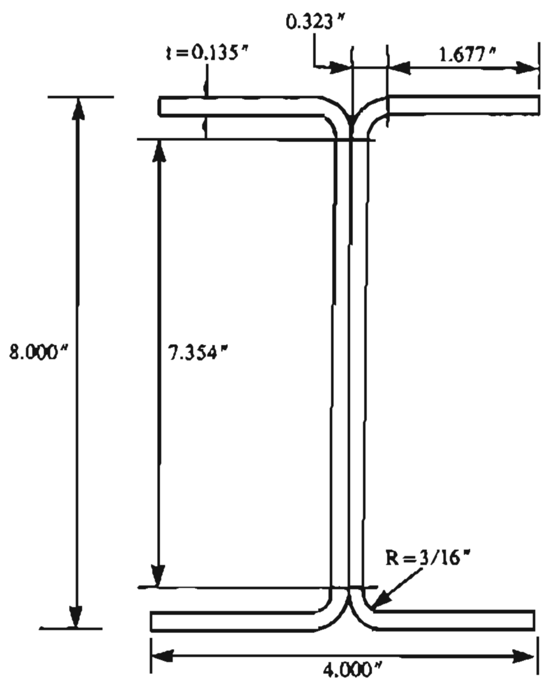
$\phi(P_n)_2 = 0.90 \times 31.18$

$= 28.06$ kips.

Since $\phi(P_n)_2 < \phi(P_n)_1$, therefore $\phi P_n = 28.06$ kips.in

EXAMPLE NO. 31

I-SECTION



Given:

1. Steel: F_y = 50 ksi.
2. Section: 8 x 4 x 0.135 I-section with unstiffened flanges.
3. Span: L = 12 ft., with simple supports, and carries uniform load.
4. Beam is laterally braced at both ends and midspan.

Required:

Determine the design flexural strength, $\phi_b M_n$.

Solution:

1. Nominal section strength (Section C3.1.1).

   a. Procedure I - based on initiation of yielding.

The elastic section modulus of the effective section, S_e, based on initiation of yielding can be determined by either computations which follow the flow charts provided in Part VI of the Manual (see Example 1), or by taking the S_e value provided in Table 15 of Part V of the Manual. The latter method is used in this example.

$S_e = 6.246$ in.^{3}

$M_n = S_eF_y$ (Eq. C3.1.1-1)

$= 6.246 \times 50 = 312.30$ kip-in.

b. Procedure II - based on inelastic reserve capacity

Since the member is subjected to lateral buckling, therefore this provision does not apply in this example. Then,

$(M_n)_l = 312.30$ kip-in.

$\phi_b = 0.90$

$\phi_b(M_n)_l = 0.90 \times 312.30 = 281.07$ kip-in.

2. Lateral buckling strength (Section C3.1.2).

From Table 6 of Part V of the Manual, $S_f = 6.538$ in.^{3}, $I_y = 1.4499$ in.^{4}

$M_y = S_fF_y$ (Eq. C3.1.2-4)

$= 6.538 \times 50 = 326.90$ kip-in.

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

$= 1.75+1.05(0/M_{max})+0.3(0/M_{max})^2 = 1.75 < 2.3$

$I_{yc} = I_y/2 = 1.4499/2 = 0.725$ in.^{4}

$M_e = \pi^2EC_bdI_{yc}/L^2$ (Eq. C3.1.2-15)

$= \pi^2(29500)(1.75)(8)(0.725)/(6\times12)^2$

$= 570.06$ kip-in.

$0.56M_y = 0.56 \times 326.90 = 183.06$ kip-in.

$2.78M_y = 2.78 \times 326.90 = 908.78$ kip-in.

Since $2.78M_y > M_e > 0.56M_y$, therefore.

$M_c = (10/9)M_y(1-10M_y/36M_e)$ (Eq. C3.1.2-13)

$= (10/9)(326.90)[1-10(326.90)/(36\times570.06)]$

$= 305.36$ kip-in.

$M_n = S_c(M_c/S_f)$ (Eq. C3.1.2-1)

$= S_cf$

where

f $= M_c/S_f = 305.36/6.538 = 46.71$ ksi

Properties of 90° corners:

$r = R + t/2 = 3/16 + 0.135/2 = 0.255$ in.

Length of arc, $u = 1.57r = 1.57 \times 0.255 = 0.400$ in.

Distance of c.g. from center of radius,

$c = 0.637r = 0.637 \times 0.255 = 0.162$ in.

Determination of elastic section modulus of the effective section calculated at a stress of f = 46.71 ksi in the extreme compression fiber (assume the webs are fully effective):

Compression flange: k = 0.43 (unstiffened compression element)

$w/t = 1.677/0.135 = 12.42 < 60$ OK (Section B1.1-(a)-(3))

$\lambda = (1.052/\sqrt{k})(w/t)\sqrt{f/E}$ (Eq. B2.1-4)

$= (1.052/\sqrt{0.43})(12.42)\sqrt{46.71/29500} = 0.793 > 0.673$

$\rho = (1-0.22/\lambda)/\lambda$ (Eq. B2.1-3)

$= (1-0.22/0.793)/0.793 = 0.911$

$b = \rho w$ (Eq. B2.1-2)

$= 0.911 \times 1.677 = 1.528$ in.

Effective section properties about x axis:

Element	L Effective Length (in.)	y Distance from Top Fiber (in.)	Ly ($in.^2$)	Ly^2 ($in.^3$)	I'_1 About Own Axis ($in.^3$)
Webs	14.708	4.000	58.832	235.328	66.286
Upper Corners	0.800	0.161	0.129	0.021	—
Lower Corners	0.800	7.839	6.271	49.158	—
Compression Flanges	3.056	0.068	0.208	0.014	—
Tension Flanges	3.354	7.933	26.607	211.076	—
Sum	22.718		92.047	495.597	66.286

Distance from top fiber to x-axis is

$y_{cg} = 92.047/22.718 = 4.052$ in.

Since distance of top compression fiber from neutral axis is greater than one half the beam depth, a compression stress of f = 46.71 ksi will govern.

To check if webs are fully effective (Section B2.3):

$f_1 = [(4.052-0.323)/4.052] \times 46.71 = 42.99$ ksi(compression)

$f_2 = -[(3.948-0.323)/4.052] \times 46.71 = -41.79$ ksi(tension)

$\psi = f_2/f_1 = -41.79/42.99 = -0.972$

$k = 4+2(1-\psi)^3+2(1-\psi)$ (Eq. B2.3-4)

$= 4+2[1-(-0.972)]^3+2[1-(-0.972)]$

$= 23.281$

$h = w = 7.354$ in., $h/t = w/t = 7.354/0.135 = 54.47$

$h/t = 54.47 < 200$ OK (Section B1.2-(a))

$\lambda = (1.052/\sqrt{23.281})(54.47)\sqrt{42.99/29500} = 0.453 < 0.673$

$b_e = w$ (Eq. B2.1-1)

$= 7.354$ in.

$b_2 = b_e/2$ (Eq. B2.3-2)

$= 7.354/2 = 3.677$ in.

$b_1 = b_e/(3-\psi)$ (Eq. B2.3-1)

$= 7.354/[3-(-0.972)] = 1.851$ in.

Compression portion of the web calculated on the basis of the effective section = $y_{cg} - 0.323 = 4.052 - 0.323 = 3.729$ in.

Since $b_1+b_2 = 5.528$ in. > 3.729 in., b_1+b_2 shall be taken as 3.729 in.. This verifies the assumption that the webs are fully effective.

$I'_x = Ly^2+I'_1-Ly^2_{cg}$

$= 495.599 + 66.286 - 22.718(4.052)^2$

$= 188.885$ in.^{3}

Actual $I_x = I'_x t$

$= 188.885 \times 0.135$

$= 25.499$ in.^{4}

$S_c = I_x/y_{cg}$

$= 25.499/4.052$

$= 6.293$ in.^{3}

$(M_n)_2 = S_c f = 6.293 \times 46.71$

$= 293.95$ kip-in.

$\phi_b = 0.90$

$\phi_b(M_n)_2 = 0.90 \times 293.95$

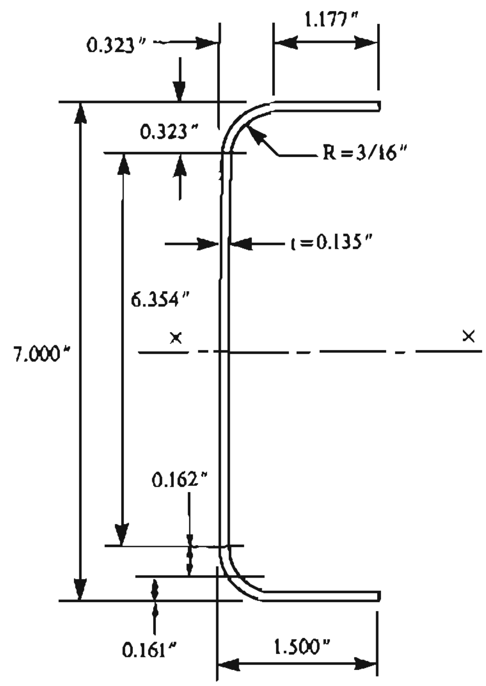
$= 264.56$ kip-in. $< \phi_b(M_n)_1 = 281.07$ kip-in.

Therefore, $\phi_b M_n$ = 264.56 kip-in. (i.e., lateral buckling strength controls).

EXAMPLE NO. 32

CHANNEL SECTION BRACED

Complete Flexural Design,
Unstiffened Compression Flange



Given:

1. Steel: F_y = 50 ksi.
2. Section: 7 x 1.5 x 0.135 channel with unstiffened flanges.
3. Span: Section is continuous over three 10 ft. spans with 6 in support bearing lengths.
4. Loading: Live load = 360 lb/ft. Dead load = 40 lb/ft..
5. Deflection due to service live load is to be limited to 1/240 of the span.

Required:

Check adequacy of the section.

Solution:

1. Nominal section strength, M_n (Section C3.1.1).

   a. Procedure I - based on initiation of yielding. The elastic section modulus of the effective section, S_e, calculated with the extreme compression or tension fiber at F_y can be determined by either computations which follow the flow charts for Section B2 in Part VI of the Manual (see Example 1), or by taking the S_e value provided in Table 11 of Part V of the Manual. The latter method is used in this example.

   $S_e = 2.240 \text{ in.}^3$

   $M_n = S_eF_y$ (Eq. C3.1.1-1)

   $= 2.240 \times 50 = 112.00$ kip-in.

   b. Procedure II - based on inelastic reserve capacity

   For unstiffened compression element, $C_y = 1$.Maximum compressive strain $= C_ye_y = e_y$. Therefore, the nominal ultimate moment, M_n, is the same as the M_n determined by procedure I because the compression flange will yield first.

2. Lateral buckling strength, M_n (Section C3.1.2). Since the compression flange is braced against lateral buckling, this provision does not apply.

3. Design flexural strength, ϕ_bM_n (based on nominal section strength).

   $\phi_b = 0.90$

   $\phi_bM_n = 0.90 \times 112.00 = 100.80$ kip-in.

   This value can be used for both positive and negative bending.

   $w_u = 1.2w_D + 1.6w_{LL} = 1.2(0.04)+1.6(0.36) = 0.624$ kip/ft.

   For a continuous beam over three equal spans, the maximum bending moment is negative and occurs over the interior supports. It is given by

   $M_u = 0.100w_uL^2 = 0.100(0.624)(10)^2(12)$

   $= 74.88 \text{ kip-in.} < \phi_bM_n = 100.80$ kip-in. OK

4. Strength for shear only (Section C3.2).The required shear strength at any section shall not exceed the design shear strength, ϕ_vV_n:

   $K_v = 5.34$ (unreinforced web)

   $\sqrt{EK_v/F_y} = \sqrt{29500(5.34)/50} = 56.13$

   $h/t = 6.354/0.135 = 47.07$

   For $h/t < \sqrt{EK_v/F_y}$

   $\phi_v = 1.00$

   $V_n = 0.577F_yht$ (Eq. C3.2-1)

   $= 0.577(50)(6.354)(0.135) = 24.75$ kips

$\phi_v V_n = 1.00(24.75) = 24.75$ kips

The maximum required shear strength is given by

$V_u = 0.600 w_u L$

$= (0.600)(0.624)(10) = 3.74 \text{ kips} < \phi_v V_n = 24.75$ kips OK

5. Strength for combined bending and shear (Section C3.3). At the interior supports there is a combination of web bending and web shear:

$\phi_b M_{nxo} = 100.80$ kip-in. $\quad M_u = 74.88$ kip-in.

$\phi_v V_n = 24.75$ kips $\quad V_u = 3.74$ kips

For unreinforced webs.

$(M_u/\phi_b M_{nxo})^2 + (V_u/\phi_v V_n)^2 \leq 1.0$ (Eq. C3.3-1)

$(74.88/100.80)^2 + (3.74/24.75)^2 = 0.575 < 1.0$ OK

6. Web crippling strength (Section C3.4)

$R/t = (3/16)/0.135 = 1.389 < 6$ OK

$h/t = 6.354/0.135 = 47.07 < 200$

Table C3.4-1 applies:

For end reactions: Eq. C3.4-2

For interior reactions: Eq. C3.4-4

$k = F_y/33 = 50/33 = 1.515$ (Eq. C3.4-21)

$C_1 = (1.22-0.22k)$ (Eq. C3.4-10)

$= 1.22-0.22(1.515) = 0.887$

$C_2 = (1.06-0.06R/t) \leq 1.0$ (Eq. C3.4-11)

$= 1.06-0.06(1.389) = 0.977 < 1.0$ OK

$C_3 = (1.33-0.33k)$ (Eq. C3.4-12)

$= 1.33-0.33(1.515) = 0.830$

$C_4 = (1.15-0.15R/t) \leq 1.0$ but not less than 0.50 (Eq. C3.4-13)

$1.15-0.15R/t = 1.15-0.15(1.389) = 0.942 \leq 1.0$ OK

> 0.50 OK

$C_4 = 0.942$

$C_\theta = 0.7+0.3(\theta/90)^2$ (Eq. C3.4-20)

$= 0.7+0.3(90/90)^2 = 1.0$

For end reaction:

$P_n \quad = t^2kC_3C_4C_\theta\,[217-0.28(h/t)\,]\,[\,1+0.01(N/t)\,]$ (Eq. C3.4-2)

$= (0.135)^2(1.515)(0.830)(0.942)(1.0)\,[\,217-0.28(47.07)\,]$

$\quad x\,[\,1+0.01(6/0.135)\,] = 6.36$ kips

$\phi_w \quad = 0.75$

$\phi_w P_n \quad = 0.75(6.36) = 4.77$ kips

End reaction is given by

$R \quad = 0.400 w_u L$

$= (0.400)(0.624)(10) = 2.50 \text{ kips} < \phi_w P_n = 4.77$ kips OK

For interior reaction:

$P_n \quad = t^2kC_1C_2C_\theta\,[538-0.74(h/t)\,]\,[\,1+0.007(N/t)\,]$ (Eq. C3.4-4)

$= (0.135)^2(1.515)(0.887)(0.977)(1.0)\,[\,538-0.74(47.07)\,]$

$\quad x[\,1+0.007(6/0.135)\,] = 15.77$ kips

$\phi_w \quad = 0.75$

$\phi_w P_n \quad = 0.75(15.77) = 11.83$ kips

Interior reaction is given by

$R = \quad 1.10 w_u L$

$= \quad (1.10)(0.624)(10) = 6.86 \text{ kips} < \phi_w P_n = 11.84$ kips OK

7. Combined bending and web crippling strength (Section C3.5). At the interior supports there is a combination of web bending and web crippling:

$\phi_b M_{nxo} = 100.80$ kip-in. $\qquad M_u = 74.88$ kip-in.

$\phi_w P_n = 11.84$ kips $\qquad R = 6.86$ kips

For shapes having single unreinforced webs:

$1.07(P_u/\phi_w P_n)+(M_u/\phi_b M_{nxo}) \leq 1.42$ (Eq. C3.5-1)

$1.07(6.86/11.83)+(74.88/100.80) = 1.36 < 1.42$ OK

8. Deflection due to service live load.

$S_x \quad = 2.240 \text{ in.}^3$ (Table 2)

$S_e \quad = 2.240 \text{ in.}^3$ (Table 11)

Since $S_e = S_x$, therefore section is fully effective at $F_y = 50$ ksi. For any stress f which is less than $F_y = 50$ ksi, the section will be fully effective, i.e.,

$I_x \quad = 7.840 \text{ in.}^4$ (Table 2)

This value can be used for deflection determination.

The maximum deflection occurs at a distance of 0.446L from the exterior supports. It is given by

Δ = $0.0069wL^4/EI$

= $0.0069(0.36)(10)^4(12)^3/(29500x7.84)$

= 0.186 in.

This deflection is limited to 1/240 of the span, i.e.,

L/240 = 10x12/240 = 0.5 in. > 0.186in. OK

From the above calculations, it can be concluded that the section is adequate.

CHARTS & TABLES

CHARTS AND TABLES

FOR USE WITH THE
MARCH 16, 1991 EDITION OF THE

LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

LRFD Cold-Formed Steel Design Manual - Part V

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PREFACE

This document, *Part V of the LRFD Cold-Formed Steel Design Manual*, contains two groups of design aids: (A) charts and tables prepared to assist in the use of particular design provisions of the *Specification*, and (B) tables of section properties. Included in Group A is an extensive series of graphs related to the compression member design procedures contained in *Part III, Supplementary Information.*

These *Charts and Tables* should be used in conjunction with the other parts of the *Design Manual*, which include *Commentary (Part II), Supplementary Information (Part III)*, and *Illustrative Examples (Part IV)*, in addition to the *Specification (Part I).*

American Iron and Steel Institute
December 1991

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GROUP A

CHARTS RELATED TO PARTICULAR SPECIFICATION PROVISIONS

GENERAL NOTES

(a) The appropriate equations from the Specification are generally shown on each design aid.

(b) The definitions of the terms used in these charts and tables can be found in the Specification.

(c) The torsional-flexural buckling charts are grouped together by cross-section type for convenience; that is, singly-symmetric angle sections, singly-symmetric channel sections, and singly-symmetric hat sections.

(d) The torsional-flexural buckling design charts are based on a square corner approximation for all section properties.

(e) In the titles and labels for the torsional-flexural buckling design charts, "**a**" and "σ" are used interchangeably.

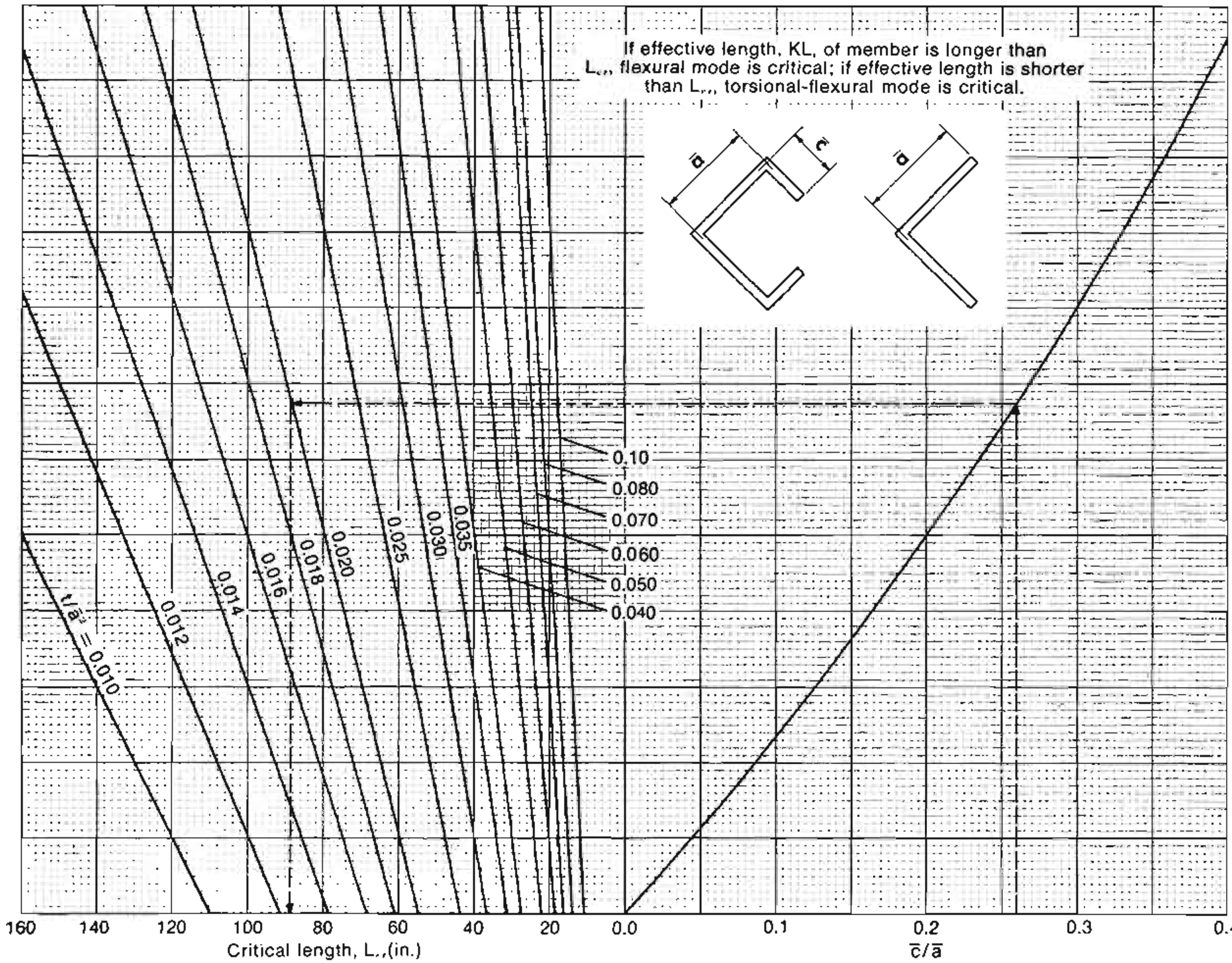


CHART V-1.1

Torsional-Flexural Buckling

(See Part III, Section 2 for application)

Buckling Mode for Equal Angles (Singly-Symmetric), With and Without Lips

CHART V-1.2
$C_x/\bar{a}^2$ for Equal Angles (Singly-Symmetric), With and Without Lips

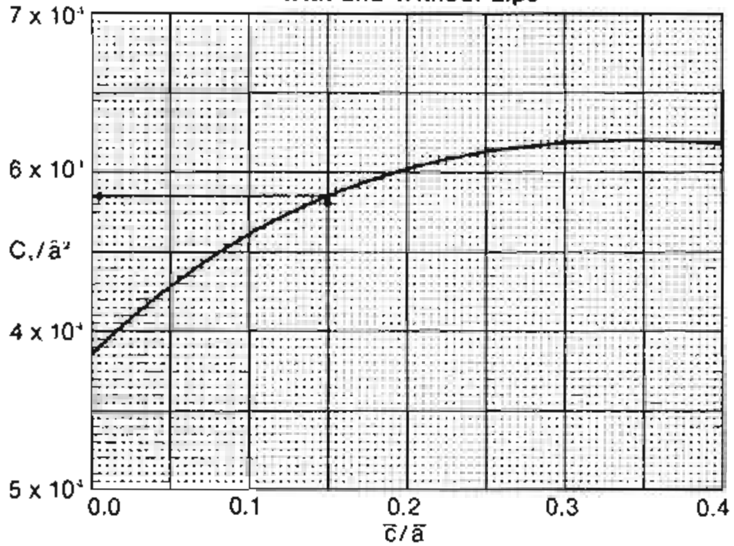


CHART V-1.3
$\sigma_{t0}\,(\bar{a}/t)^2$ for Equal Angles (Singly-Symmetric), With and Without Lips

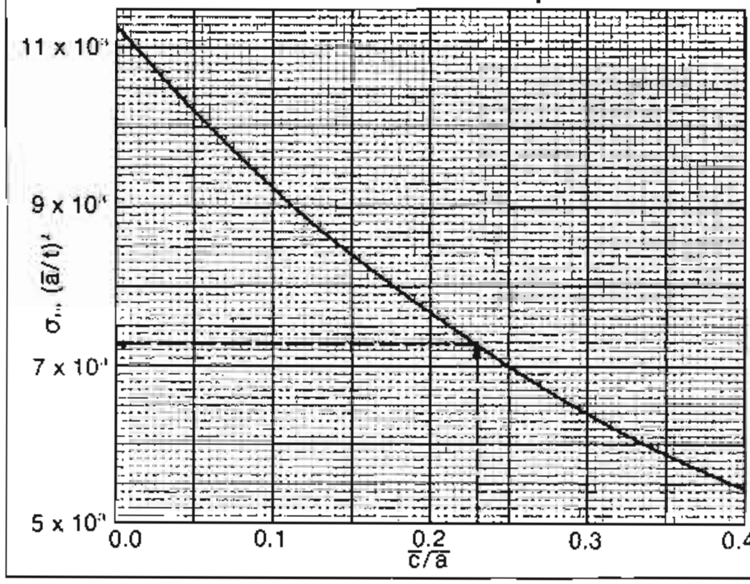
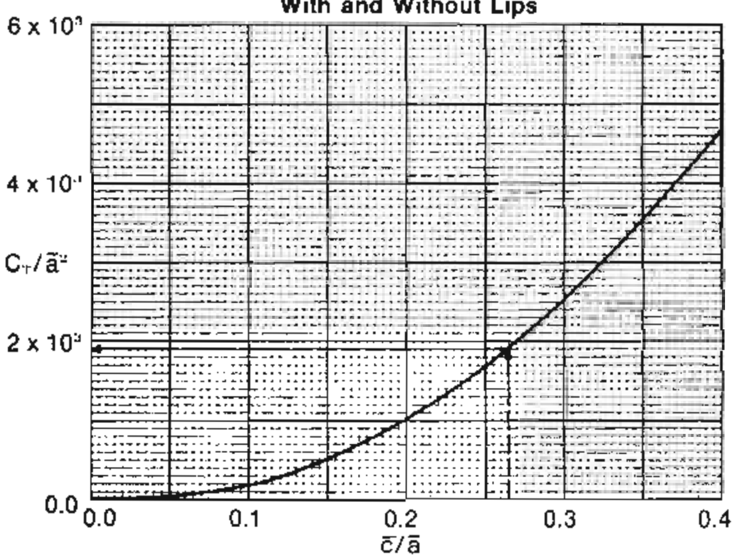


CHART V-1.4
$C_T/\bar{a}^2$ for Equal Angles (Singly-Symmetric), With and Without Lips



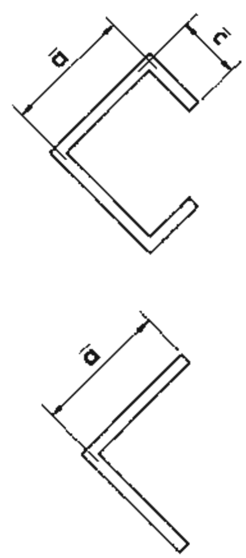
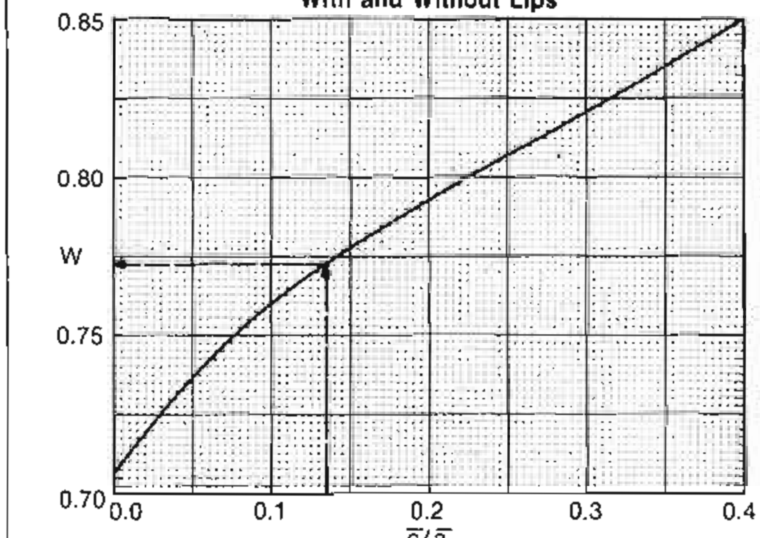


CHART V-1.6
W-Factor for Equal Angles (Singly-Symmetric), With and Without Lips



TORSIONAL-FLEXURAL BUCKLING
(See Part III, Section 2 for application)

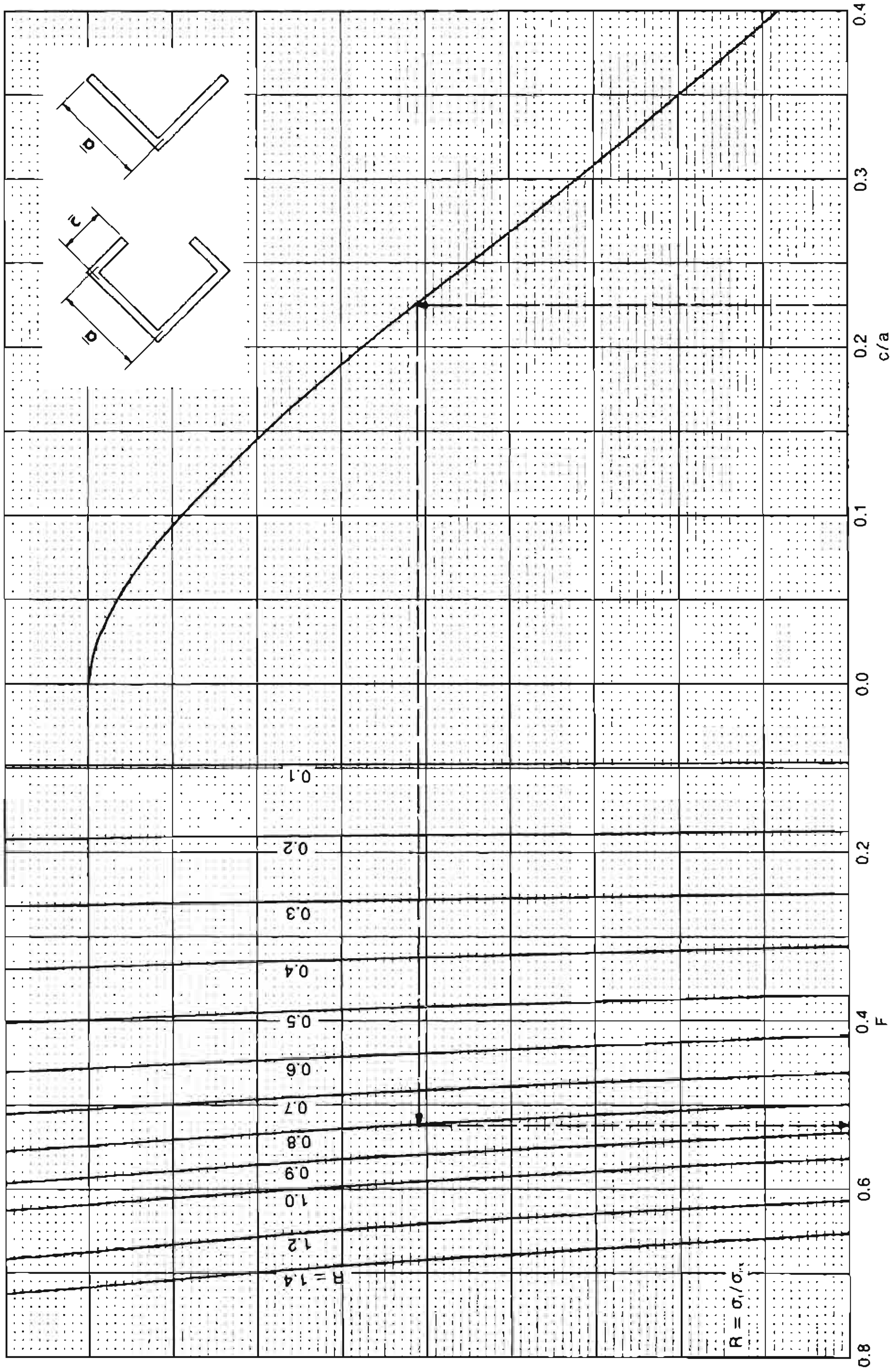


CHART V-1.5

Torsional-Flexural Buckling

(See Part III, Section 2 for application)

F-Factor for Equal Angles (Singly-Symmetric), With and Without Lips

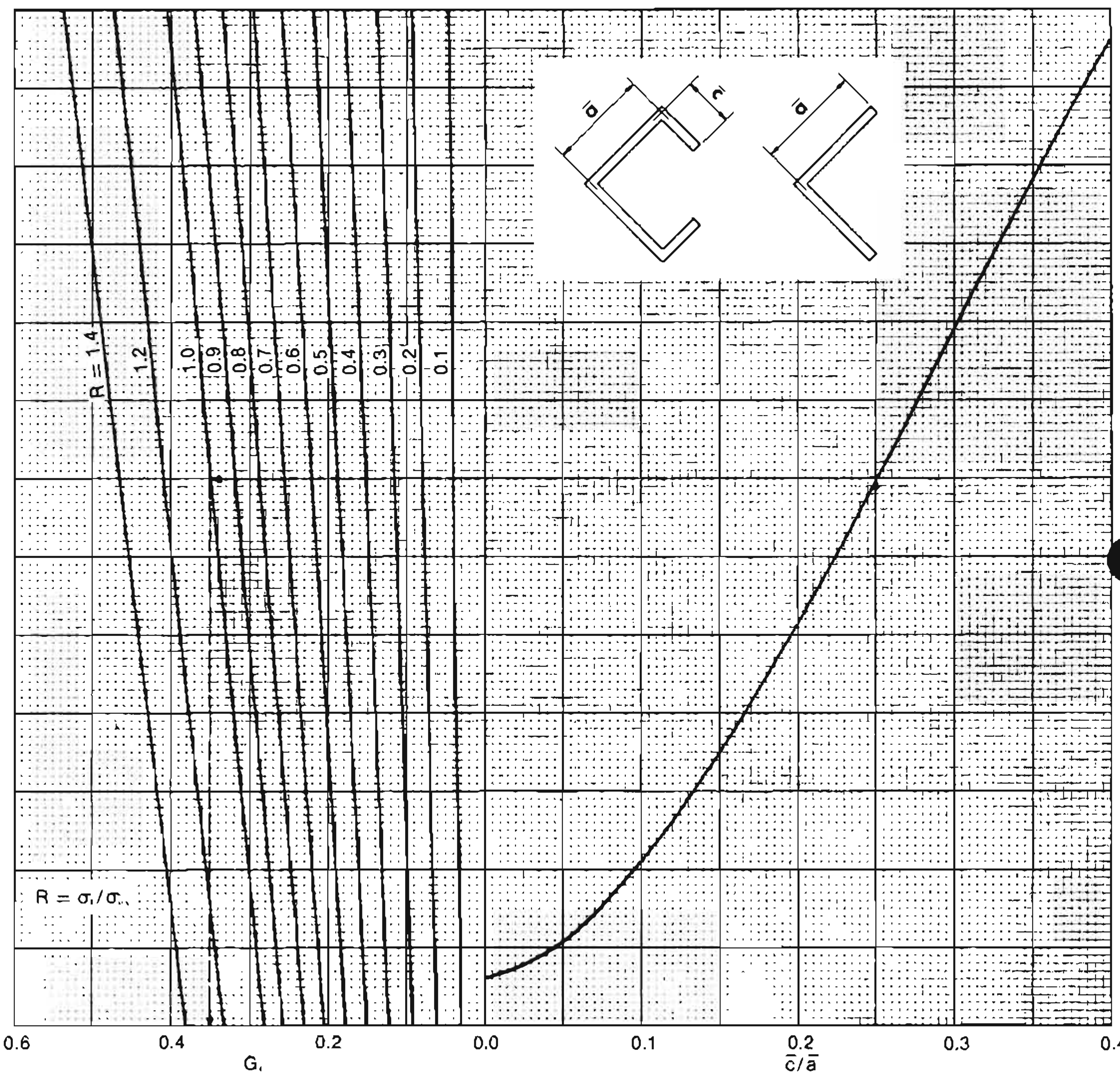


CHART V-1.7

Torsional-Flexural Buckling

(See Part III, Section 2 for application)

G-Factor for Equal Angles (Single-Symmetric), With and Without Lips

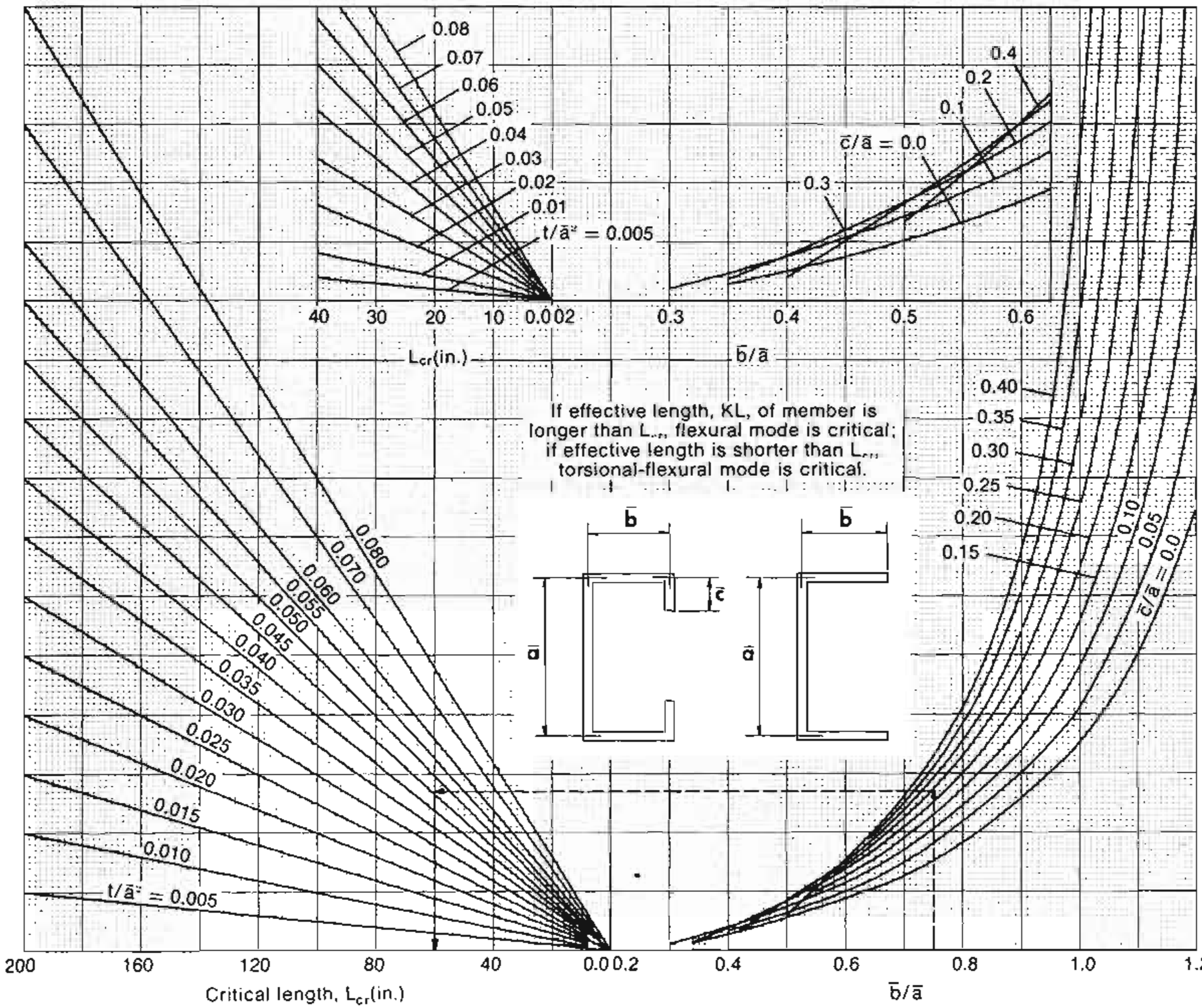


CHART V-2.1
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
Buckling Mode for Channels (Singly-Symmetric), With and Without Lips

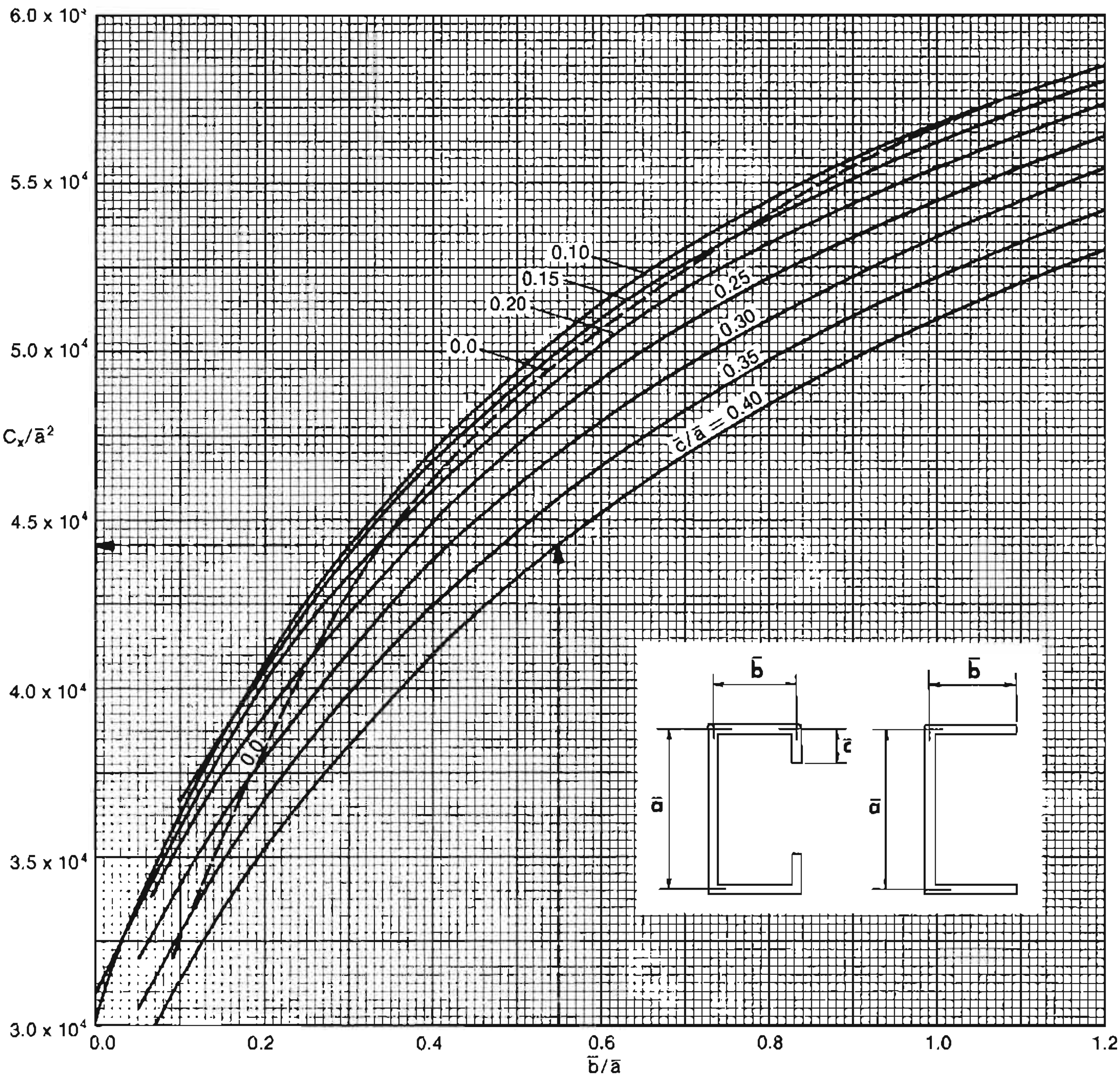


CHART V-2.2
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
$C_x/\bar{a}^2$ for Channels (Singly-Symmetric), With and Without Lips

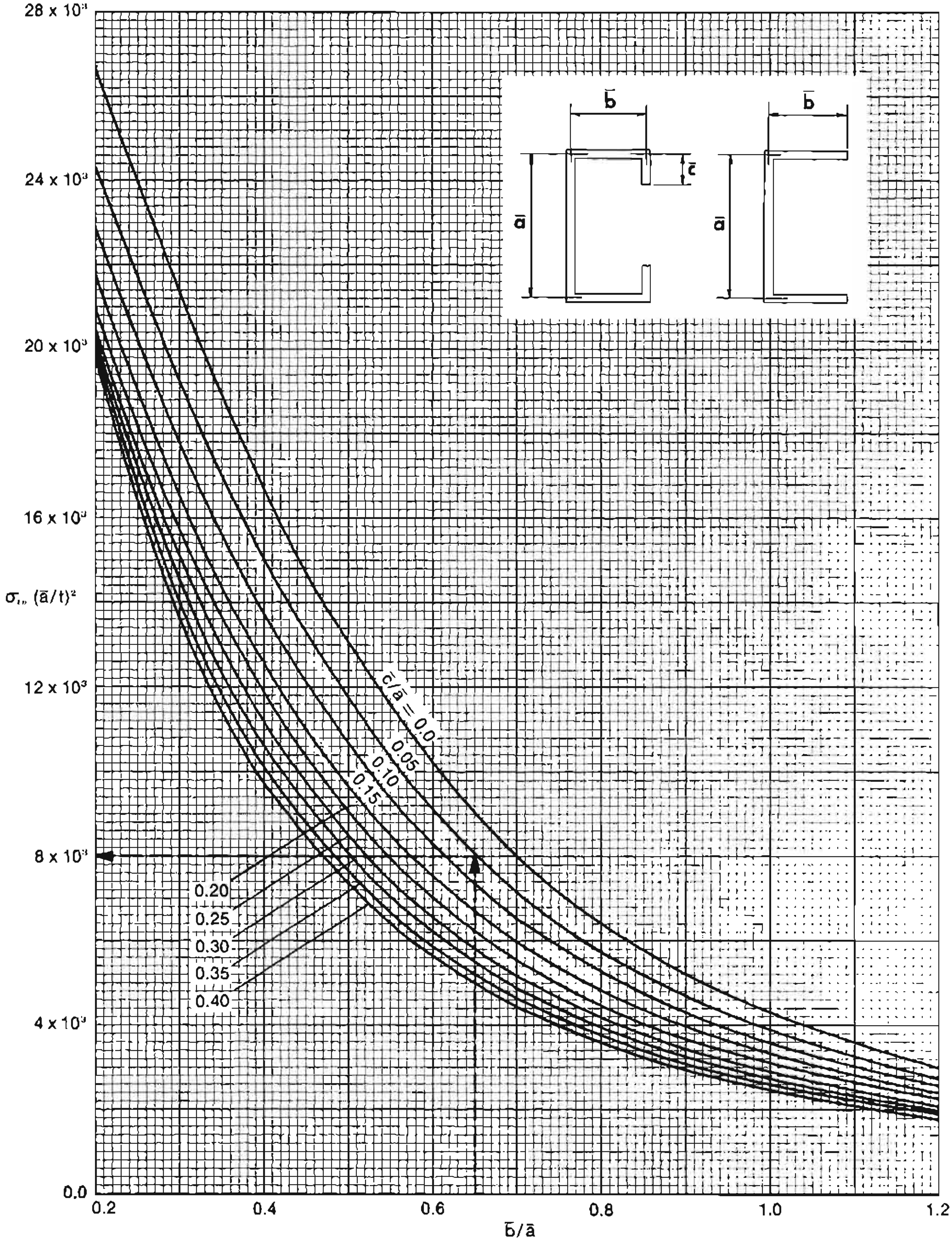


CHART V-2.3
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
σ_{t_o} (ā/t)² for Channels (Singly-Symmetric), With and Without Lips

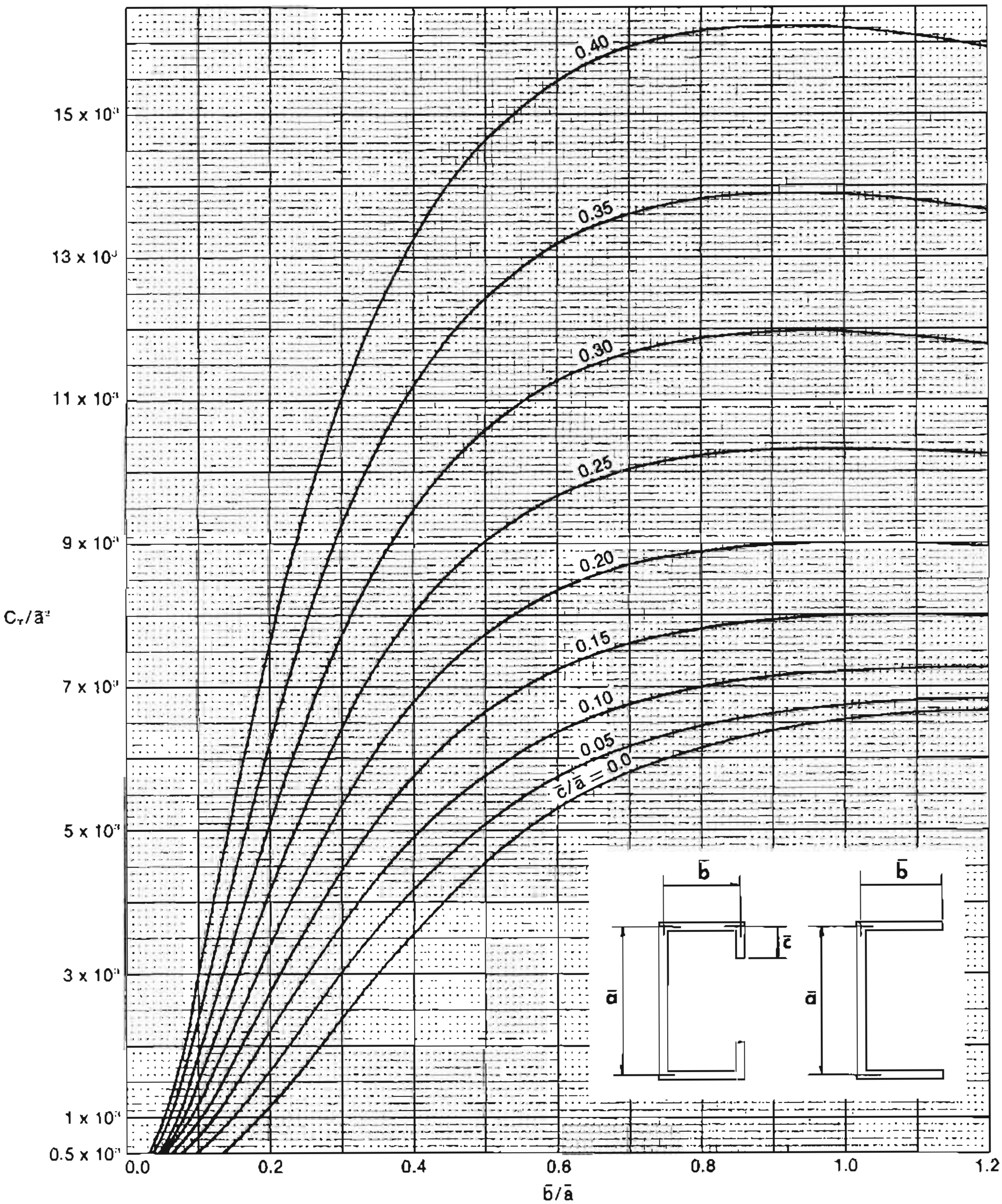


CHART V-2.4
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
$C_w/\bar{a}^2$ for Channels (Singly-Symmetric), With and Without Lips

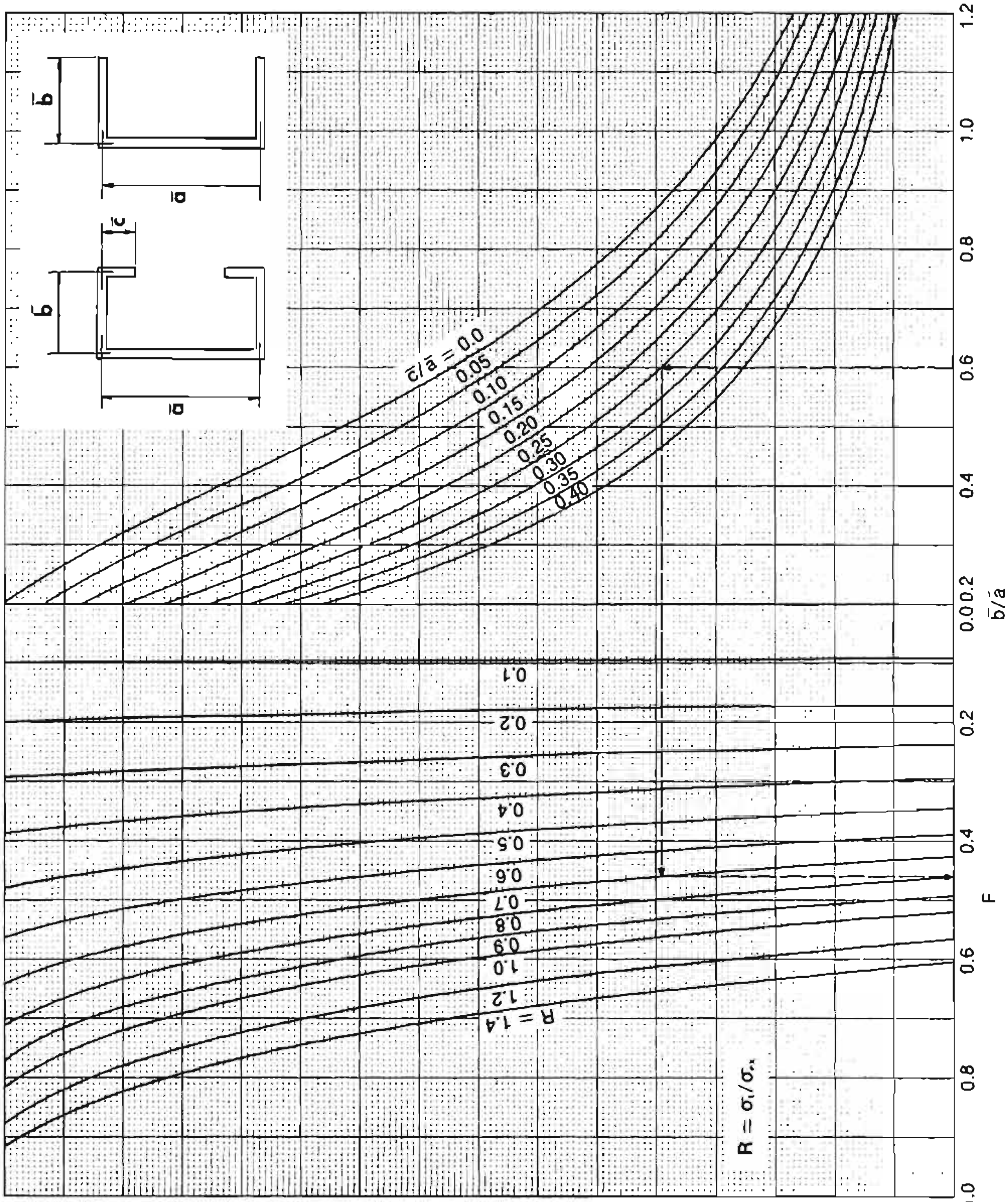


CHART V-2.5
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
F-Factor for Channels (Singly-Symmetric), With and Without Lips

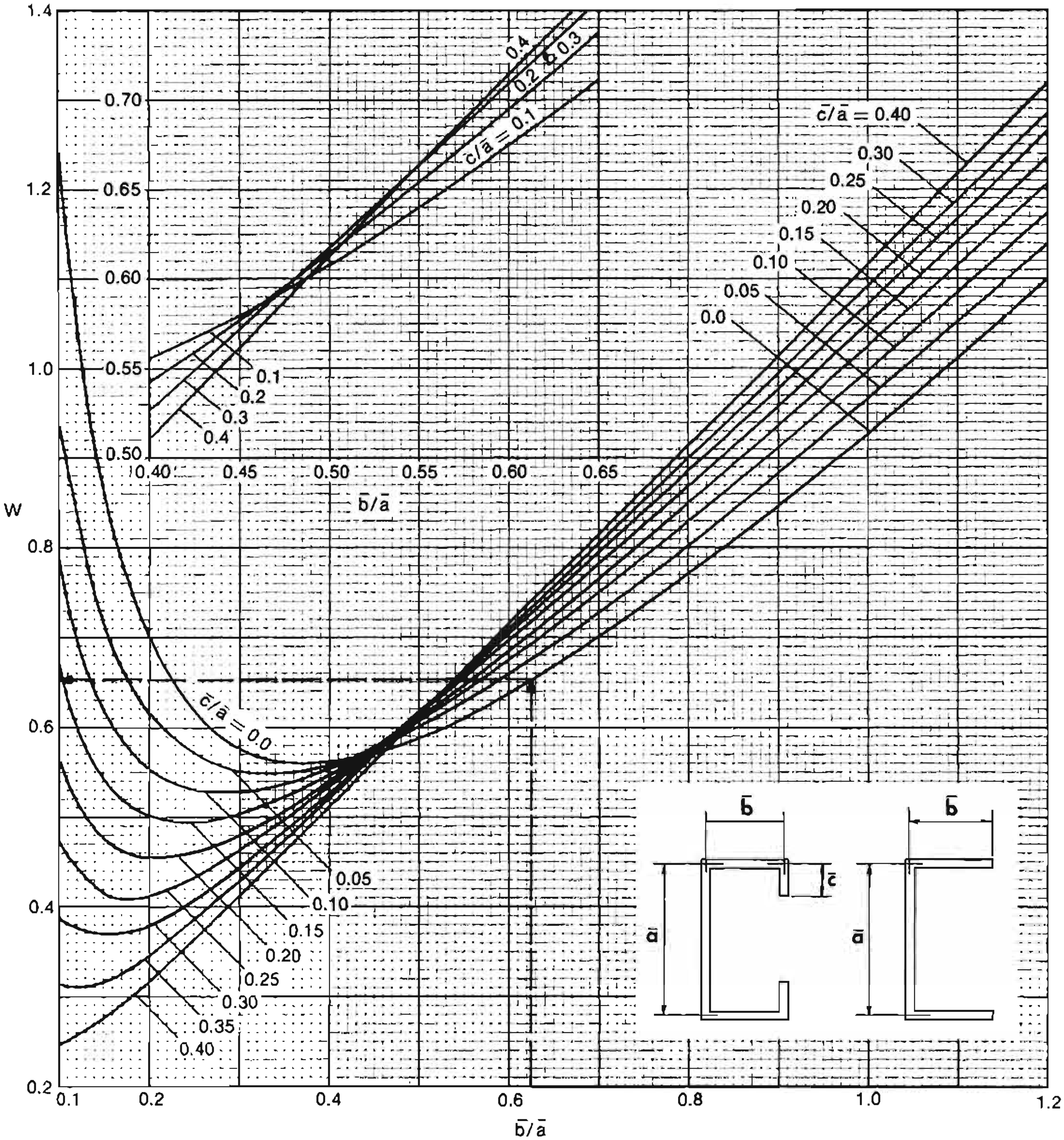


CHART V-2.6
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
W-Factor for Channels (Singly-Symmetric), With and Without Lips

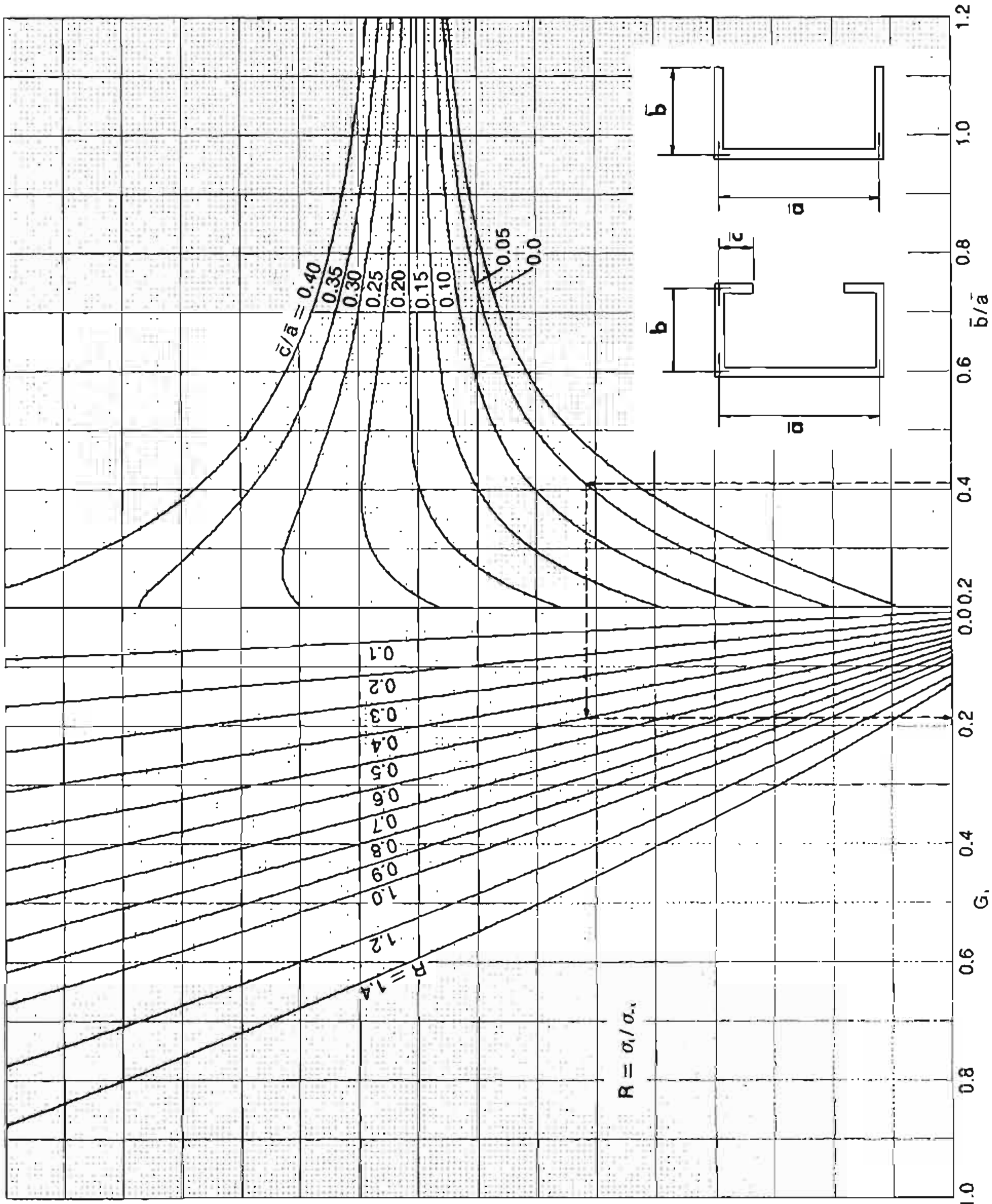


CHART V-2.7
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
G_t-Factor for Channels (Singly-Symmetric), With and Without Lips

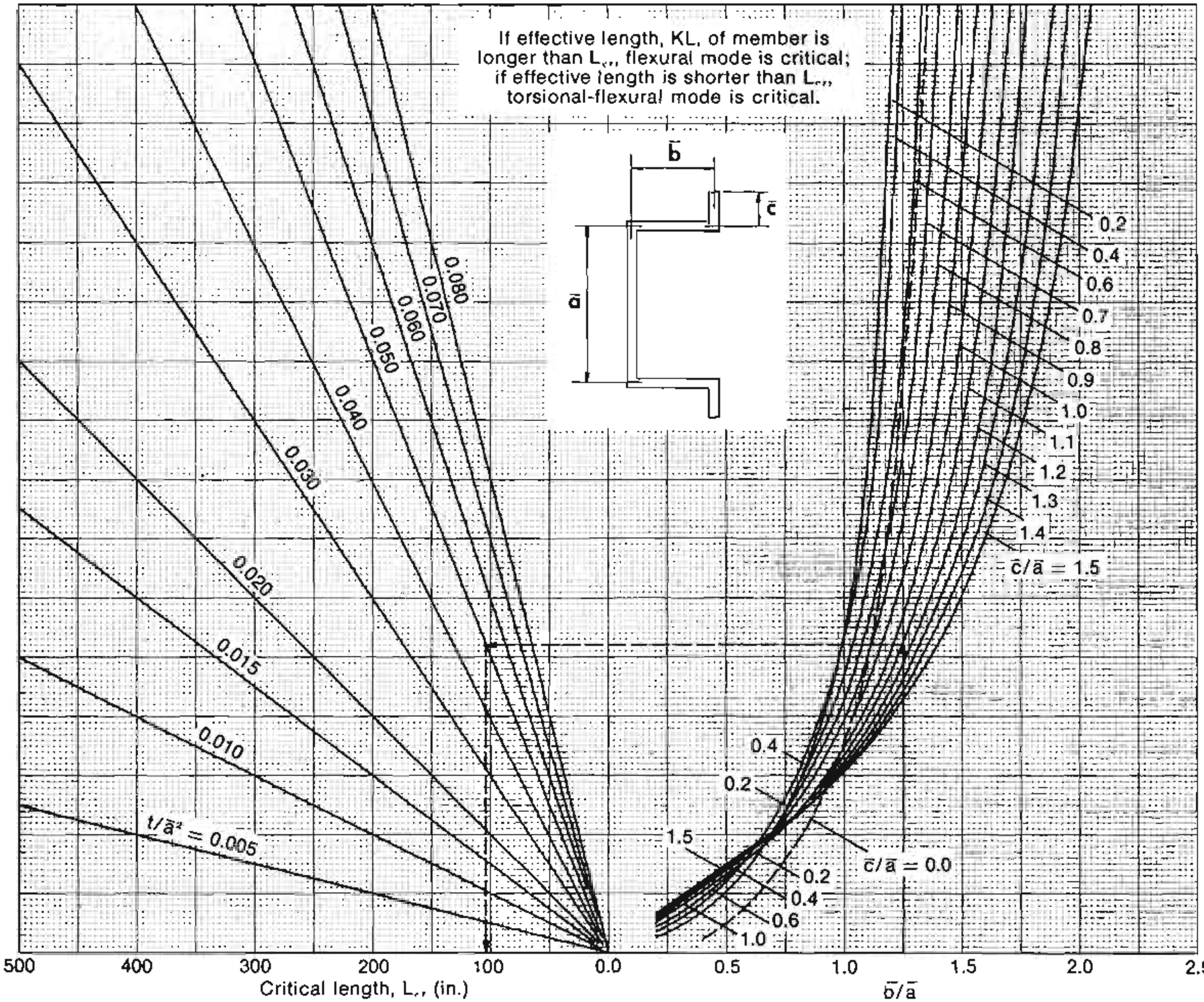


CHART V-3.1

Torsional-Flexural Buckling

(See Part III, Section 2 for application)

Buckling Mode for Hat Sections (Singly-Symmetric)

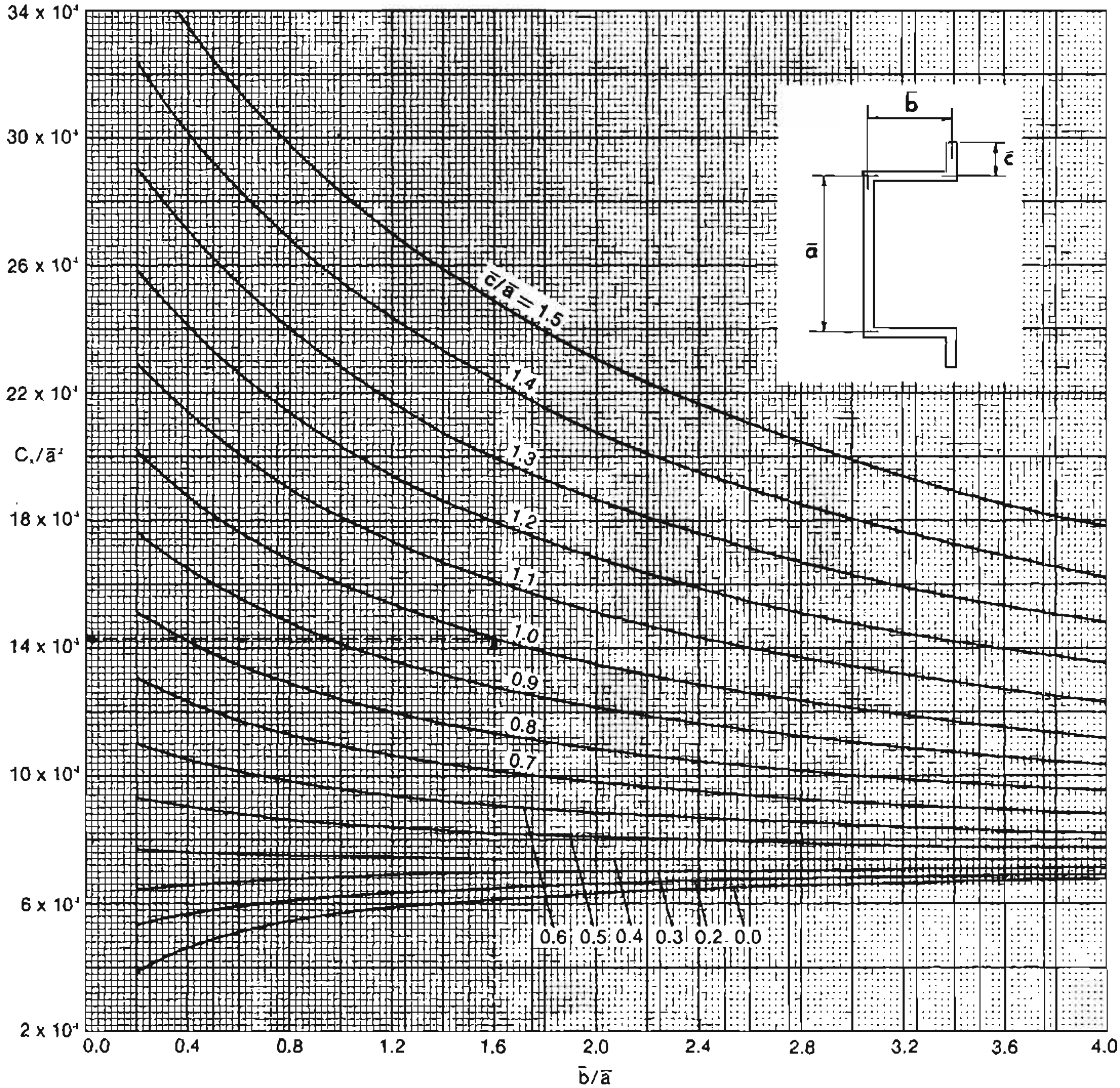


CHART V-3.2

Torsional-Flexural Buckling
(See Part III, Section 2 for application)
$C_s/\bar{a}^2$ for Hat Sections (Singly-Symmetric)

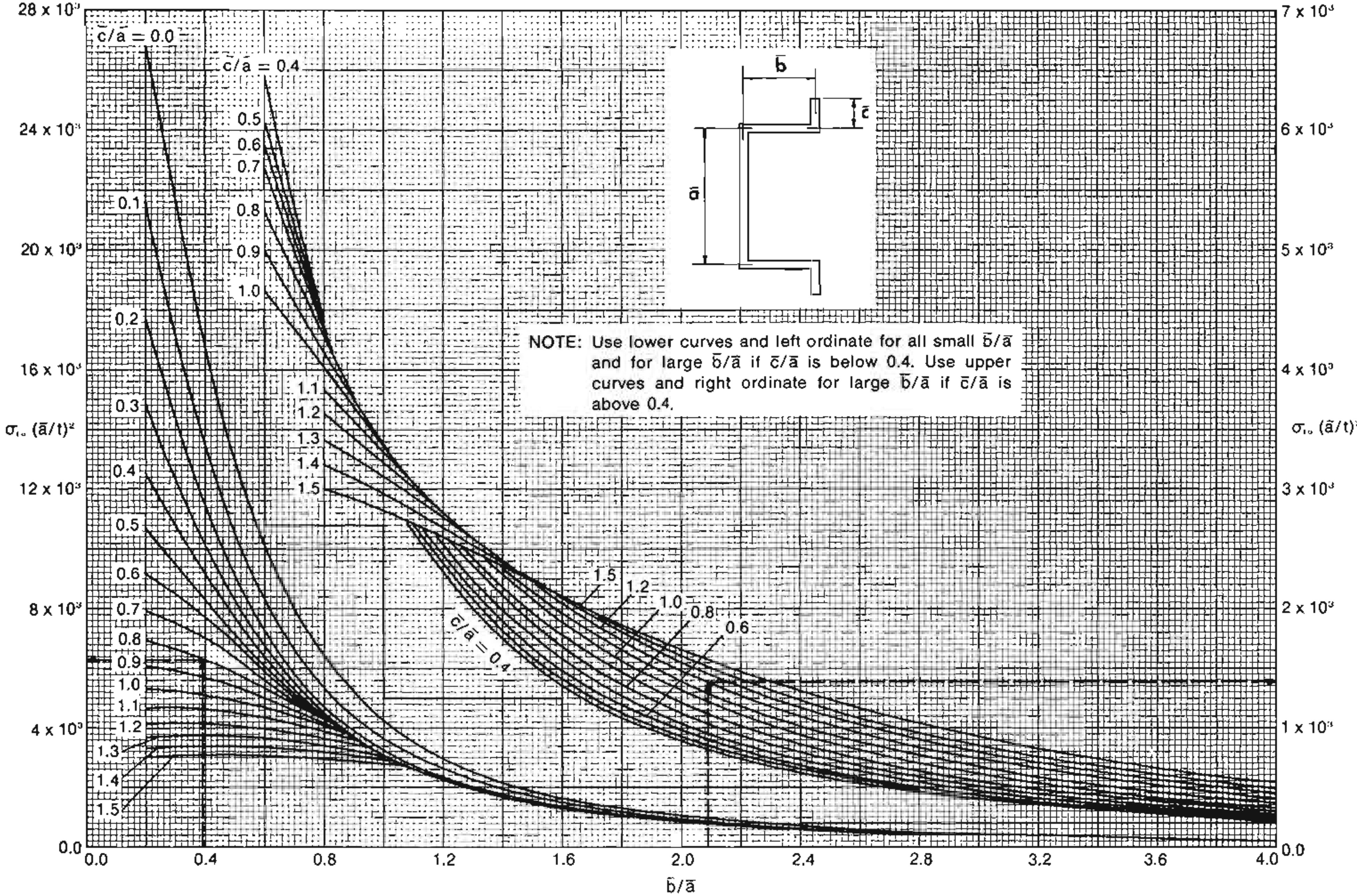


CHART V-3.3
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
$\sigma_{to}\,(\bar{a}/t)^2$ for Hat Sections (Singly-Symmetric)

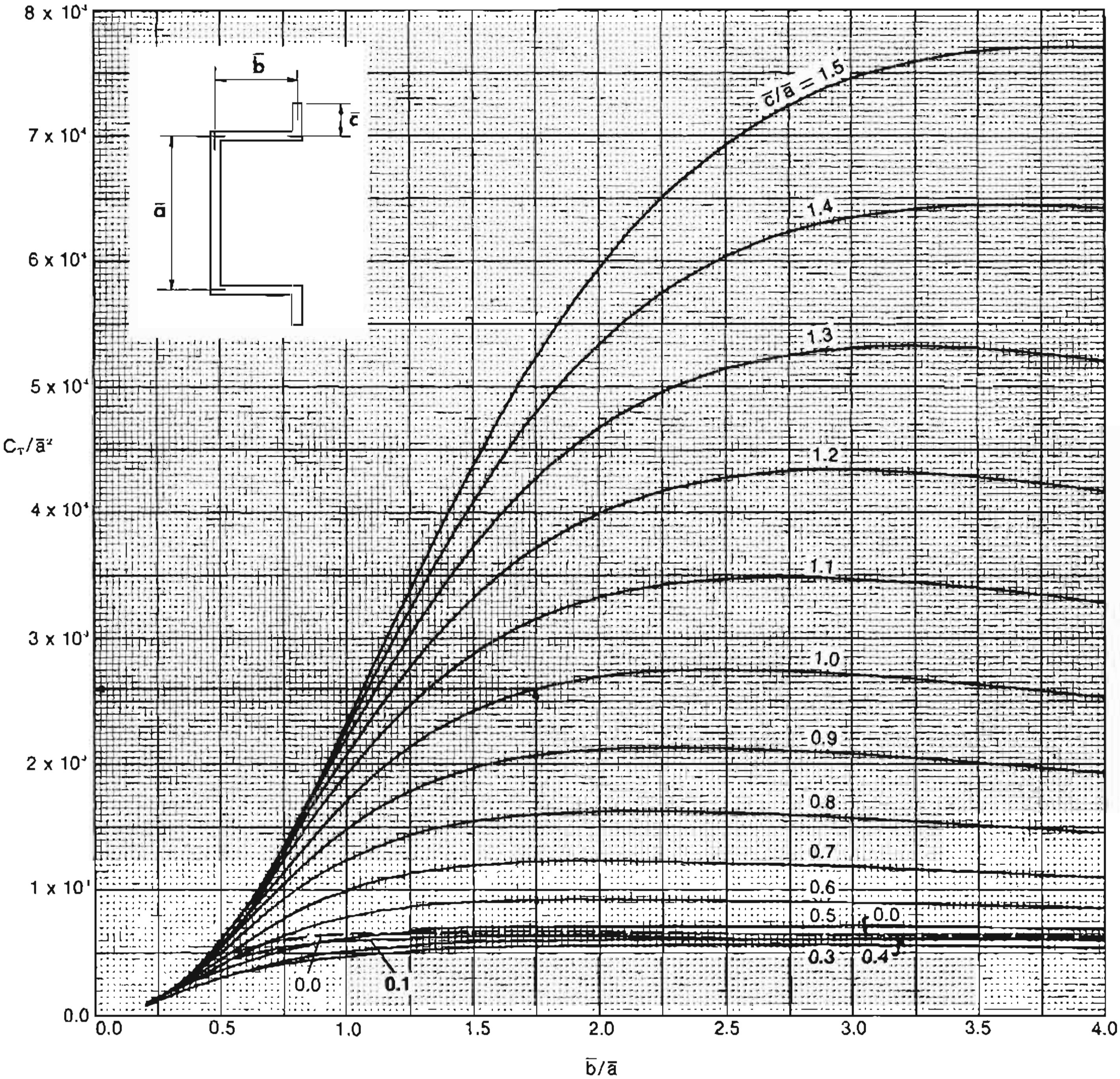


CHART V-3.4

Torsional-Flexural Buckling

(See Part III, Section 2 for application)

$C_T/\bar{a}^2$ for Hat Sections (Singly-Symmetric)

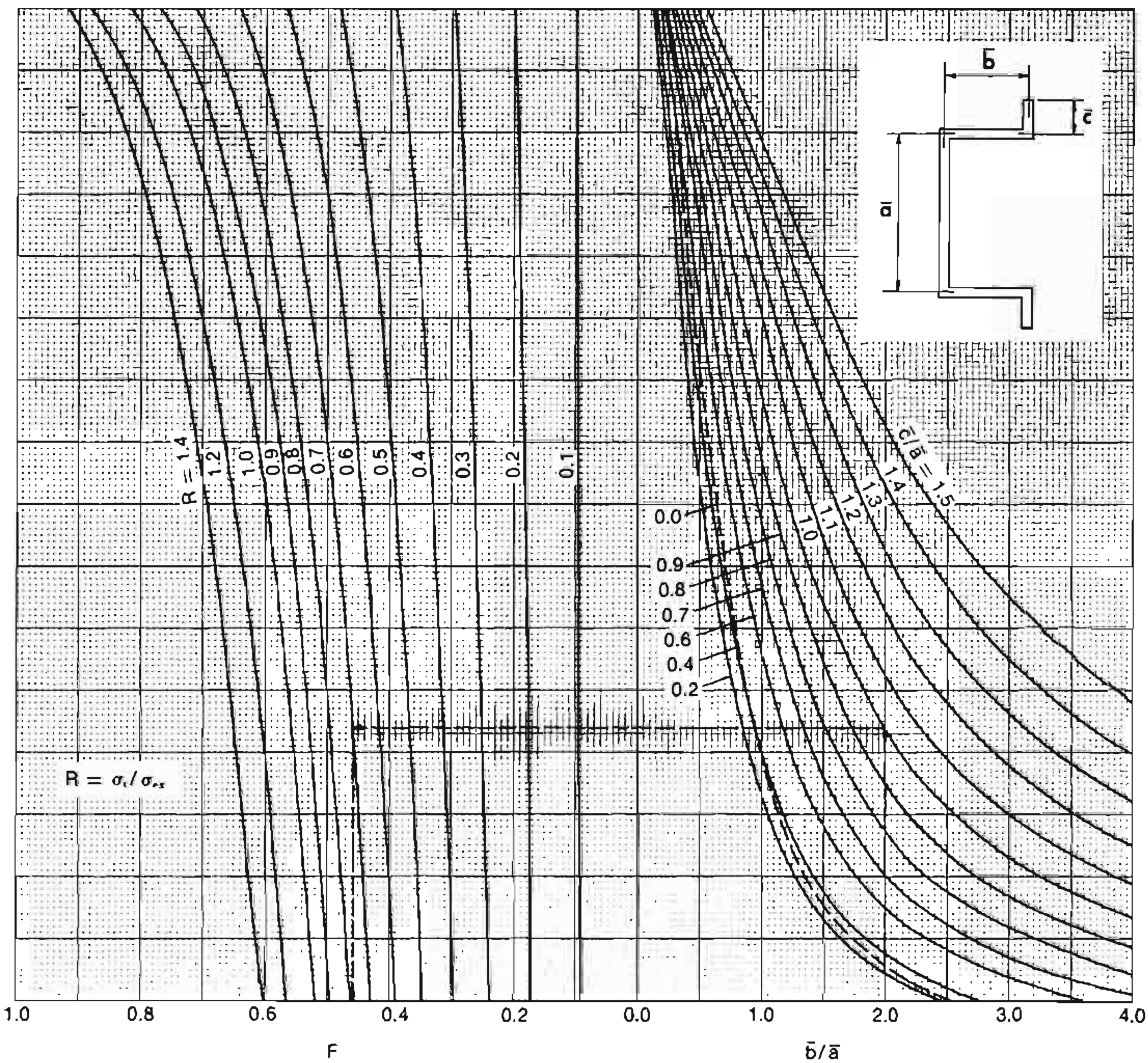


CHART V-3.5

Torsional-Flexural Buckling
(See Part III, Section 2 for application)
F-Factor for Hat Sections (Singly-Symmetric)

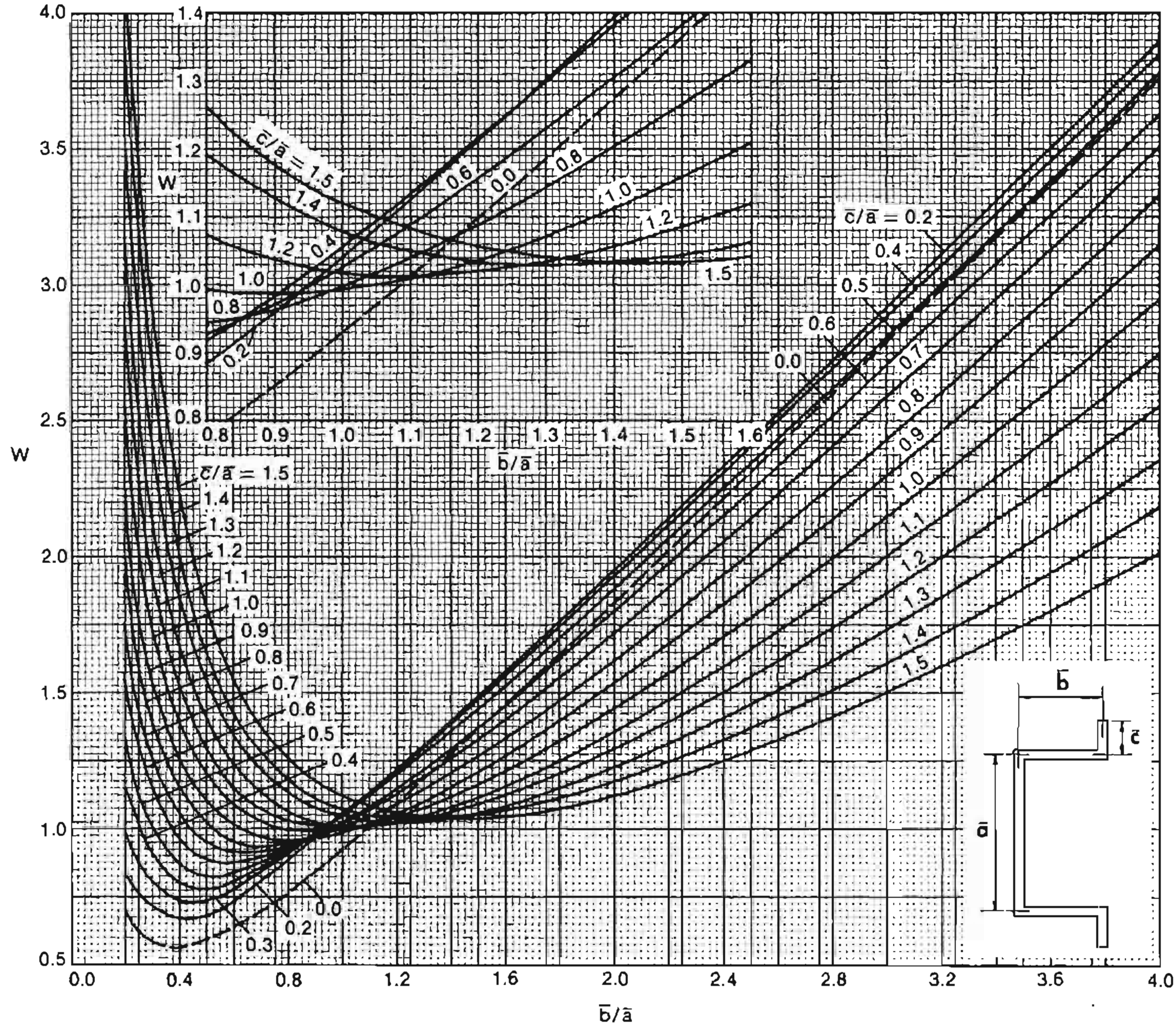


CHART V-3.6
Torsional-Flexural Buckling
(See Part III, Section 2 for application)
W-Factor for Hat Sections (Singly-Symmetric)

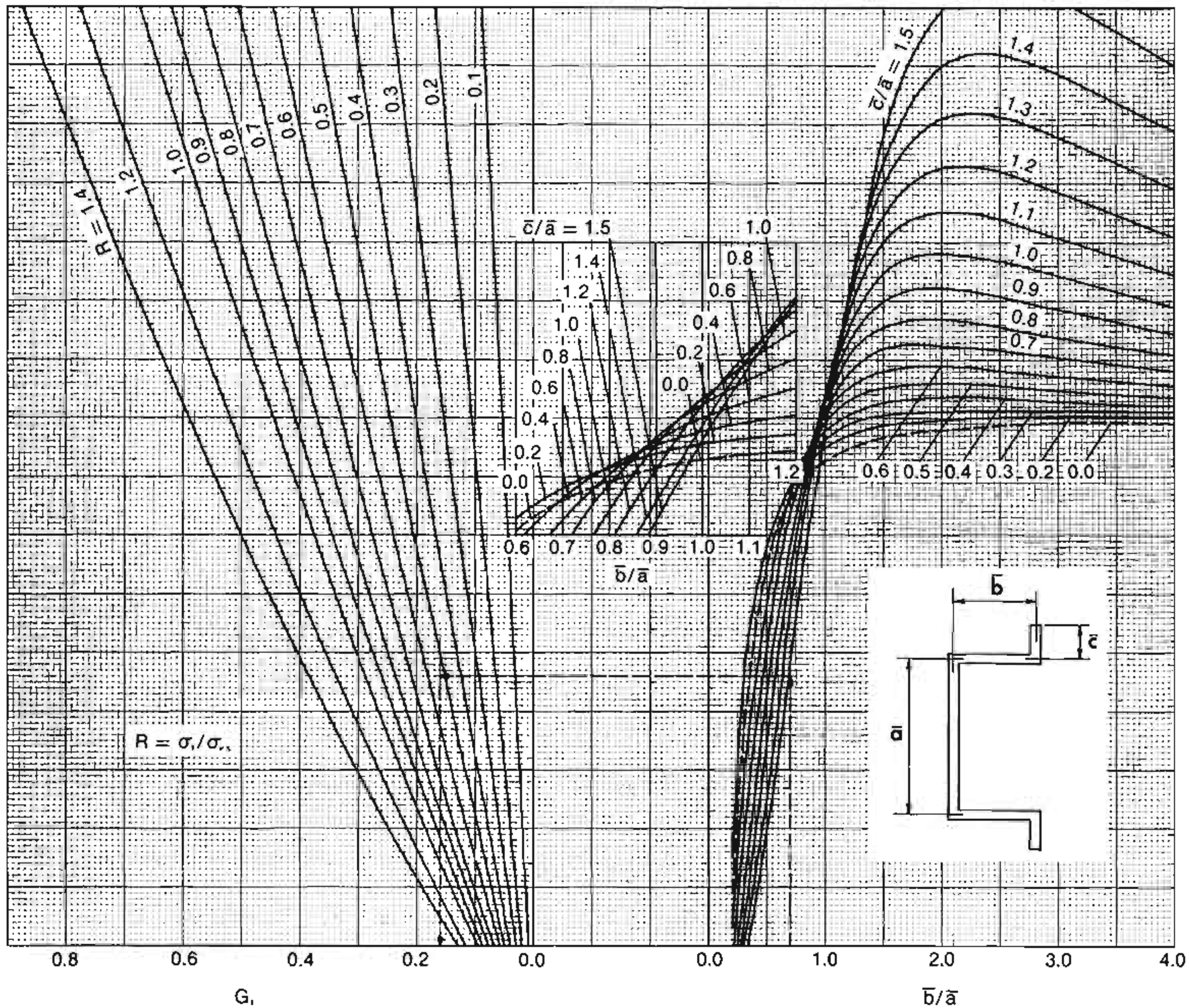


CHART V-3.7

Torsional-Flexural Buckling
(See Part III, Section 2 for application)
G_{u}-Factor for Hat Sections (Singly-Symmetric)

GROUP B
TABLES OF SECTION PROPERTIES
GENERAL NOTES

(a) The specific sections listed in these tables are not necessarily stock sections. They are included primarily as a guide in the design of cold-formed steel structural members.

(b) The effective section modulus values are calculated as the ratio of effective moment of inertia at the indicated stress level and the distance to the extreme fiber. In calculating the maximum moment capacity of these sections, additional checks such as the provisions of Chapter C of the *Specification* and the information on laterally unbraced compression flanges in Part III should also be taken into account where applicable.

(c) As a general rule, tabulated section properties are shown to three significant figures, while dimensions are given to three decimal places. However, in some cases space limitations made it impractical to adhere strictly to this guideline.

(d) The weight of these sections is calculated based on steel as weighing 40.8 pounds per square foot per inch thickness.

(e) Where they apply, the algebraic formulae presented in Part III formed the basis of the calculations for these tables.

(f) The properties of Tables 5, 6, 14, and 15 apply only when the channels are adequately joined together. See Chapter D of the *Specification*.

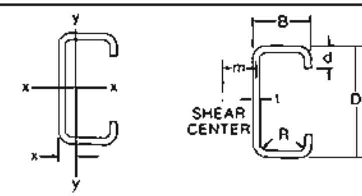
(g) Tables 1-9 incl. are Full Area Tables. The effective section properties listed in Tables 10-15 incl. were computed for bending about x-axis using $F_y = 33$ksi and 50ksi.

(h) The Full Area Tables were prepared at Cornell University by Shyh Hann Ji.

TABLE 1

CHANNEL WITH STIFFENED FLANGES

TABLE 1



See notes on page V-25

Size D	Size B	t	d	R	Area	Wgt. per Foot	Axis x-x I_x	Axis x-x S_x	Axis x-x r_x	Axis y-y I_y	Axis y-y S_y	Axis y-y r_y	Axis y-y x	m	J	C_w	j	r_o	x_o
In.	In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.	In.	In.^{4}	In.^{6}	In.	In.	In.
12.0	3.50	0.135	1.01	0.1875	2.706	9.20	56.267	9.378	4.56	4.037	1.560	1.222	0.912	1.487	0.01640	119.00	6.78	5.27	-2.33
		0.105	0.90	0.1875	2.097	7.13	43.836	7.306	4.57	3.090	1.181	1.214	0.883	1.461	0.00771	89.90	6.84	5.26	-2.29
10.0	3.50	0.135	1.01	0.1875	2.436	8.28	36.526	7.305	3.87	3.823	1.533	1.253	1.006	1.582	0.01480	79.90	5.59	4.79	-2.52
		0.105	0.90	0.1875	1.887	6.41	28.506	5.701	3.89	2.929	1.160	1.246	0.975	1.553	0.00693	60.10	5.64	4.77	-2.48
		0.075	0.72	0.09375	1.344	4.57	20.533	4.107	3.91	2.035	0.792	1.231	0.932	1.473	0.00252	39.60	5.58	4.73	-2.37
9.0	3.25	0.135	1.00	0.1875	2.230	7.58	27.158	6.035	3.49	3.072	1.344	1.174	0.965	1.507	0.01360	53.20	5.03	4.40	-2.40
		0.105	0.84	0.1875	1.717	5.84	21.077	4.684	3.50	2.300	0.987	1.157	0.921	1.458	0.00631	38.50	5.10	4.36	-2.33
		0.075	0.70	0.09375	1.228	4.18	15.287	3.397	3.53	1.627	0.689	1.151	0.888	1.393	0.00230	25.90	5.03	4.34	-2.24
		0.060	0.61	0.09375	0.976	3.32	12.182	2.707	3.53	1.265	0.530	1.138	0.862	1.364	0.00117	19.80	5.06	4.31	-2.20
8.0	3.00	0.135	0.93	0.1875	2.009	6.83	19.372	4.843	3.11	2.356	1.127	1.083	0.909	1.409	0.01220	32.60	4.51	3.98	-2.25
		0.105	0.81	0.1875	1.553	5.28	15.125	3.781	3.12	1.794	0.844	1.075	0.876	1.375	0.00571	24.10	4.56	3.97	-2.20
		0.075	0.70	0.09375	1.116	3.79	11.046	2.761	3.15	1.290	0.600	1.075	0.851	1.321	0.00209	16.50	4.48	3.95	-2.14
		0.060	0.60	0.09375	0.885	3.01	8.791	2.198	3.15	0.997	0.458	1.061	0.822	1.288	0.00106	12.50	4.51	3.92	-2.08
8.0	1.625	0.105	0.82	0.1875	1.266	4.31	10.646	2.662	2.90	0.391	0.331	0.556	0.411	0.707	0.00465	5.79	4.95	3.12	-1.07
		0.075	0.82	0.09375	0.927	3.15	7.996	1.999	2.94	0.319	0.261	0.587	0.415	0.707	0.00174	4.43	4.69	3.18	-1.08
		0.060	0.60	0.09375	0.720	2.45	6.190	1.548	2.93	0.222	0.176	0.555	0.369	0.645	0.000864	2.96	5.00	3.14	-0.986
		0.048	0.50	0.09375	0.569	1.94	4.889	1.222	2.93	0.166	0.135	0.541	0.347	0.618	0.000437	2.20	5.13	3.13	-0.943
7.0	2.75	0.135	0.88	0.1875	1.793	6.10	13.266	3.790	2.72	1.777	0.940	0.996	0.859	1.320	0.01090	19.20	3.99	3.58	-2.11
		0.105	0.88	0.1875	1.410	4.79	10.553	3.015	2.74	1.438	0.761	1.010	0.859	1.335	0.00518	15.70	3.99	3.62	-2.14
		0.075	0.70	0.09375	1.003	3.41	7.659	2.188	2.76	1.000	0.517	0.999	0.815	1.251	0.00188	10.10	3.95	3.57	-2.03
		0.060	0.60	0.09375	0.795	2.70	6.099	1.742	2.77	0.772	0.393	0.986	0.786	1.217	0.000954	7.56	3.98	3.54	-1.97
6.0	2.50	0.135	0.82	0.1875	1.574	5.35	8.575	2.858	2.33	1.293	0.764	0.906	0.808	1.228	0.00956	10.50	3.48	3.19	-1.97
		0.105	0.82	0.1875	1.240	4.22	6.843	2.281	2.35	1.051	0.621	0.921	0.808	1.244	0.00456	8.64	3.49	3.22	-2.00
		0.075	0.82	0.09375	0.909	3.09	5.124	1.708	2.37	0.805	0.478	0.941	0.814	1.232	0.00170	6.51	3.41	3.25	-2.01
		0.060	0.60	0.09375	0.705	2.40	4.012	1.337	2.39	0.583	0.333	0.909	0.752	1.148	0.000846	4.31	3.46	3.16	-1.87
6.0	1.625	0.105	0.82	0.1875	1.056	3.59	5.246	1.748	2.23	0.360	0.323	0.584	0.481	0.795	0.00388	3.16	3.29	2.60	-1.22
		0.075	0.82	0.09375	0.777	2.79	3.972	1.324	2.26	0.293	0.256	0.615	0.486	0.791	0.00146	2.43	3.15	2.65	-1.24
		0.060	0.60	0.09375	0.600	1.68	3.085	1.028	2.27	0.205	0.172	0.585	0.437	0.722	0.000720	1.58	3.33	2.60	-1.12
		0.048	0.50	0.09375	0.473	1.61	2.441	0.814	2.27	0.154	0.128	0.571	0.412	0.693	0.000363	1.56	3.41	2.58	-1.08
5.0	2.00	0.135	0.70	0.1875	1.272	4.32	4.684	1.874	1.92	0.648	0.478	0.714	0.644	0.986	0.00773	3.73	2.89	2.58	-1.56
		0.105	0.70	0.1875	1.005	3.42	3.761	1.504	1.93	0.533	0.393	0.728	0.643	1.001	0.00369	3.10	2.90	2.61	-1.59
		0.075	0.60	0.09375	0.726	2.47	2.797	1.119	1.96	0.389	0.283	0.733	0.622	0.949	0.00136	2.10	2.83	2.60	-1.53
		0.060	0.50	0.09375	0.573	1.95	2.227	0.891	1.97	0.298	0.212	0.721	0.594	0.915	0.000687	1.54	2.86	2.57	-1.48
		0.048	0.50	0.09375	0.461	1.57	1.804	0.722	1.98	0.244	0.173	0.727	0.594	0.921	0.000354	1.26	2.86	2.58	-1.49
4.0	2.00	0.135	0.70	0.1875	1.137	3.87	2.752	1.376	1.56	0.598	0.464	0.725	0.712	1.054	0.00691	2.38	2.52	2.41	-1.70
		0.105	0.70	0.1875	0.900	3.06	2.219	1.110	1.57	0.492	0.382	0.740	0.712	1.069	0.00331	1.99	2.53	2.45	-1.73
		0.075	0.60	0.09375	0.651	2.21	1.664	0.832	1.60	0.361	0.275	0.745	0.689	1.009	0.00122	1.33	2.48	2.42	-1.66
		0.060	0.50	0.09375	0.513	1.74	1.330	0.665	1.61	0.277	0.206	0.734	0.660	0.972	0.000615	0.958	2.49	2.39	-1.60
		0.048	0.50	0.09375	0.413	1.40	1.079	0.539	1.62	0.226	0.169	0.740	0.660	0.979	0.000317	0.789	2.49	2.40	-1.61

Properties of Full Section

TABLE 1 (continued)

CHANNEL WITH STIFFENED FLANGES

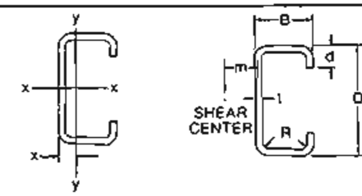


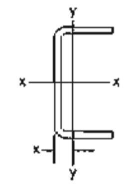
TABLE 1 (continued)

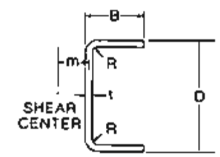
See notes on page V-25

							Properties of Full Section												
Size		t	d	R	Area	Wgt. per Foot	Axis x-x			Axis y-y				m	J	C_w	j	r_o	x_o
D	B						I_x	S_x	r_x	I_y	S_y	r_y	x						
In.	In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.	In.	In.^{4}	In.^{6}	In.	In.	In.
4.0	1.625	0.075	0.60	0.09375	0.594	2.02	1.443	0.722	1.56	0.218	0.202	0.608	0.539	0.818	0.00113	0.817	2.26	2.12	-1.32
		0.060	0.50	0.09375	0.468	1.59	1.151	0.576	1.57	0.166	0.150	0.599	0.511	0.758	0.000558	0.586	2.29	2.10	-1.27
		0.048	0.50	0.09375	0.377	1.28	0.936	0.467	1.58	0.137	0.123	0.603	0.510	0.791	0.000290	0.485	2.29	2.12	-1.28
		0.036	0.50	0.09375	0.285	0.97	0.713	0.356	1.58	0.106	0.0949	0.609	0.511	0.799	0.000123	0.376	2.30	2.13	-1.29
3.625	1.625	0.075	0.60	0.09375	0.566	1.93	1.146	0.631	1.42	0.211	0.199	0.611	0.564	0.843	0.00107	0.672	2.13	2.07	-1.37
		0.060	0.50	0.09375	0.445	1.51	0.915	0.505	1.45	0.161	0.147	0.602	0.536	0.807	0.000531	0.479	2.15	2.04	-1.31
		0.048	0.50	0.09375	0.357	1.21	0.744	0.410	1.44	0.132	0.121	0.608	0.536	0.813	0.000274	0.395	2.15	2.05	-1.33
		0.036	0.50	0.09375	0.271	0.922	0.567	0.314	1.45	0.102	0.0939	0.615	0.536	0.819	0.000118	0.307	2.16	2.05	-1.34
3.5	2.00	0.135	0.70	0.1875	1.069	3.64	2.003	1.145	1.37	0.568	0.456	0.729	0.753	1.092	0.00650	1.85	2.39	2.36	-1.78
		0.105	0.70	0.1875	0.847	2.88	1.620	0.926	1.38	0.468	0.375	0.743	0.753	1.107	0.00311	1.55	2.40	2.39	-1.81
		0.075	0.60	0.09375	0.613	2.08	1.222	0.698	1.41	0.344	0.271	0.749	0.729	1.043	0.00115	1.02	2.35	2.36	-1.73
		0.060	0.50	0.09375	0.483	1.64	0.979	0.559	1.42	0.264	0.203	0.739	0.699	1.005	0.000579	0.728	2.35	2.32	-1.67
		0.048	0.50	0.09375	0.389	1.32	0.795	0.454	1.43	0.216	0.166	0.745	0.699	1.011	0.000299	0.601	2.36	2.33	-1.69
3.0	1.75	0.105	0.70	0.1875	0.742	2.52	1.017	0.678	1.17	0.318	0.300	0.654	0.689	1.015	0.00273	0.861	2.10	2.13	-1.65
		0.075	0.53	0.09375	0.528	1.79	0.767	0.512	1.21	0.224	0.202	0.651	0.644	0.918	0.000989	0.495	2.04	2.05	-1.52
		0.060	0.53	0.09375	0.426	1.45	0.628	0.418	1.21	0.185	0.167	0.658	0.644	0.926	0.000512	0.413	2.05	2.07	-1.54
		0.048	0.41	0.09375	0.332	1.13	0.499	0.332	1.22	0.138	0.120	0.644	0.607	0.877	0.000255	0.277	2.05	2.01	-1.46
2.5	1.625	0.075	0.60	0.09375	0.482	1.64	0.480	0.383	1.00	0.183	0.189	0.618	0.656	0.924	0.000895	0.343	1.86	1.94	-1.54
		0.060	0.50	0.09375	0.378	1.29	0.387	0.310	1.01	0.141	0.142	0.612	0.626	0.886	0.000450	0.233	1.86	1.90	-1.48
		0.048	0.50	0.09375	0.305	1.04	0.316	0.253	1.02	0.116	0.116	0.616	0.626	0.890	0.000233	0.195	1.87	1.92	-1.50
		0.036	0.50	0.09375	0.231	0.786	0.242	0.193	1.02	0.0895	0.0896	0.622	0.626	0.897	0.0000991	0.153	1.87	1.92	-1.51

TABLE 2 TABLE 2

CHANNEL
WITH UNSTIFFENED FLANGES



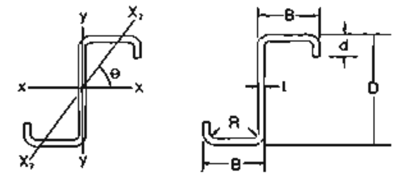


See notes on page V-25

Size		t	R	Area	Wgt. per Foot	Properties of Full Section												
						Axis x-x			Axis y-y			x	m	J	C_w	j	r_o	x_o
D	B					I_x	S_x	r_x	I_y	S_y	r_y							
In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.	In.	In.^{4}	In.^{6}	In.	In.	In.
8.0	2.00	0.135	0.18750	1.554	5.284	13.076	3.269	2.901	0.4853	0.3019	0.559	0.393	0.596	0.00944	5.56	4.84	3.09	−0.922
		0.105	0.18750	1.216	4.135	10.335	2.584	2.915	0.3862	0.2385	0.563	0.381	0.600	0.00447	4.45	4.86	3.11	−0.929
		0.075	0.09375	0.880	2.993	7.599	1.900	2.938	0.2830	0.1732	0.567	0.366	0.597	0.00165	3.27	4.89	3.13	−0.925
		0.060	0.09375	0.706	2.402	6.127	1.532	2.945	0.2290	0.1396	0.569	0.360	0.599	0.000848	2.66	4.89	3.14	−0.929
7.0	1.50	0.135	0.18750	1.284	4.366	7.840	2.240	2.471	0.2044	0.1681	0.399	0.284	0.416	0.00780	1.82	4.61	2.58	−0.633
		0.105	0.18750	1.006	3.421	6.218	1.777	2.486	0.1639	0.1335	0.404	0.272	0.421	0.00370	1.47	4.62	2.60	−0.640
		0.075	0.09375	0.730	2.483	4.603	1.315	2.511	0.1211	0.0975	0.407	0.257	0.418	0.00137	1.09	4.65	2.62	−0.638
		0.060	0.09375	0.586	1.994	3.718	1.062	2.518	0.0983	0.0788	0.410	0.251	0.420	0.000704	0.888	4.66	2.63	−0.641
6.0	1.50	0.135	0.18750	1.149	3.907	5.334	1.778	2.155	0.1973	0.1657	0.414	0.310	0.447	0.00698	1.26	3.61	2.30	−0.689
		0.105	0.18750	0.901	3.064	4.240	1.413	2.169	0.1582	0.1316	0.419	0.298	0.451	0.00331	1.02	3.62	2.32	−0.696
		0.075	0.09375	0.655	2.228	3.150	1.050	2.193	0.1171	0.0962	0.423	0.283	0.447	0.00123	0.758	3.65	2.34	−0.692
		0.060	0.09375	0.526	1.790	2.547	0.849	2.200	0.0951	0.0777	0.425	0.277	0.449	0.000632	0.618	3.66	2.35	−0.696
		0.048	0.09375	0.423	1.437	2.055	0.685	2.205	0.0770	0.0627	0.427	0.272	0.451	0.000325	0.503	3.66	2.35	−0.698
5.0	1.25	0.105	0.18750	0.744	2.529	2.399	0.960	1.796	0.0894	0.0900	0.347	0.256	0.376	0.00273	0.399	3.00	1.92	−0.580
		0.075	0.09375	0.543	1.846	1.797	0.719	1.820	0.0667	0.0661	0.350	0.241	0.372	0.00102	0.299	3.04	1.94	−0.575
		0.060	0.09375	0.436	1.484	1.456	0.583	1.827	0.0543	0.0535	0.353	0.235	0.374	0.000524	0.245	3.04	1.95	−0.579
		0.048	0.09375	0.351	1.192	1.177	0.471	1.832	0.0441	0.0432	0.355	0.230	0.376	0.000269	0.199	3.05	1.96	−0.582
4.0	1.125	0.105	0.18750	0.613	2.083	1.286	0.643	1.449	0.0623	0.0713	0.319	0.251	0.356	0.00225	0.175	2.27	1.58	−0.555
		0.060	0.09375	0.449	1.527	0.973	0.486	1.472	0.0467	0.0525	0.323	0.235	0.351	0.000842	0.132	2.30	1.60	−0.549
		0.075	0.09375	0.361	1.229	0.791	0.395	1.479	0.0382	0.0426	0.325	0.229	0.353	0.000434	0.108	2.31	1.61	−0.553
		0.048	0.09375	0.291	0.988	0.640	0.320	1.485	0.0310	0.0345	0.327	0.225	0.355	0.000223	0.0885	2.31	1.62	−0.555
3.0	1.125	0.105	0.18750	0.508	1.726	0.636	0.424	1.120	0.0573	0.0688	0.336	0.292	0.398	0.00187	0.0873	1.60	1.33	−0.637
		0.075	0.09375	0.374	1.272	0.487	0.324	1.141	0.0432	0.0508	0.340	0.275	0.390	0.000701	0.0663	1.62	1.35	−0.627
		0.060	0.09375	0.301	1.025	0.397	0.265	1.147	0.0353	0.0412	0.342	0.269	0.392	0.000362	0.0547	1.63	1.35	−0.631
		0.048	0.09375	0.243	0.825	0.322	0.215	1.153	0.0287	0.0334	0.344	0.264	0.393	0.000186	0.0448	1.64	1.36	−0.634
2.0	1.125	0.105	0.18750	0.403	1.369	0.241	0.241	0.773	0.0497	0.0645	0.351	0.355	0.450	0.00148	0.0326	1.17	1.13	−0.752
		0.075	0.09375	0.299	1.017	0.188	0.188	0.792	0.0379	0.0480	0.356	0.335	0.438	0.000561	0.0251	1.19	1.14	−0.735
		0.060	0.09375	0.241	0.821	0.154	0.154	0.798	0.0310	0.0389	0.358	0.329	0.440	0.000290	0.0208	1.20	1.15	−0.738
		0.048	0.09375	0.195	0.661	0.126	0.126	0.804	0.0253	0.0315	0.360	0.324	0.441	0.000149	0.0171	1.21	1.15	−0.741

TABLE 3 | TABLE 3

Z-SECTION WITH STIFFENED FLANGES

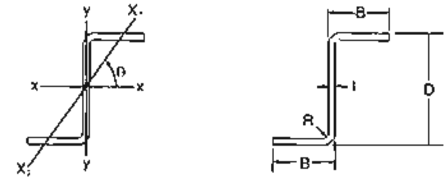


See notes on page V-25

Size		t	d	R	Area	Wgt. per Foot	Beam Strength Properties of Full Section										
							Axis x-x			Axis y-y			I_{xy}	Axis x_2-x_2	$90°-\theta$	J	C_w
D	B						I_x	S_x	r_x	I_y	S_y	r_y		r_{min}			
In.	In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.^{4}	In.	Deg.	In.^{4}	In.^{6}
12.0	3.50	0.135	1.01	0.1875	2.706	9.20	56.267	9.378	4.56	5.968	1.739	1.485	13.152	1.006	13.80	0.0164	162.00
		0.105	0.90	0.1875	2.097	7.13	43.836	7.306	4.57	4.535	1.316	1.471	10.097	0.999	13.60	0.00771	123.00
10.0	3.50	0.135	1.01	0.1875	2.436	8.28	36.526	7.305	3.87	5.968	1.739	1.565	10.867	1.013	17.71	0.0148	108.00
		0.105	0.90	0.1875	1.887	6.41	28.506	5.701	3.89	4.535	1.316	1.550	8.356	1.006	17.44	0.00693	82.20
		0.075	0.72	0.09375	1.344	4.57	20.533	4.107	3.91	3.109	0.898	1.521	5.842	0.996	16.92	0.00252	54.80
9.0	3.25	0.135	1.00	0.1875	2.230	7.58	27.158	6.035	3.49	4.869	1.530	1.477	8.506	0.945	18.68	0.0136	71.60
		0.105	0.84	0.1875	1.717	5.84	21.077	4.684	3.50	3.594	1.124	1.447	6.423	0.931	18.15	0.00631	52.50
		0.075	0.70	0.09375	1.228	4.18	15.287	3.397	3.53	2.516	0.783	1.431	4.557	0.927	17.76	0.00230	35.70
		0.060	0.61	0.09375	0.976	3.32	12.182	2.707	3.53	1.941	0.603	1.410	3.566	0.918	17.43	0.00117	27.50
8.0	3.00	0.135	0.93	0.1875	2.009	6.83	19.372	4.843	3.11	3.779	1.288	1.371	6.358	0.868	19.60	0.0122	43.70
		0.105	0.81	0.1875	1.553	5.28	15.125	3.781	3.12	2.846	0.965	1.354	4.868	0.860	19.21	0.00571	32.80
		0.075	0.70	0.09375	1.116	3.79	11.046	2.761	3.15	2.028	0.685	1.348	3.500	0.862	18.91	0.00209	22.70
		0.060	0.60	0.09375	0.885	3.01	8.791	2.198	3.15	1.552	0.523	1.324	2.725	0.851	18.49	0.00106	17.30
7.0	2.75	0.135	0.88	0.1875	1.793	6.10	13.266	3.790	2.72	2.901	1.082	1.272	4.638	0.794	20.91	0.0109	25.60
		0.105	0.88	0.1875	1.410	4.79	10.553	3.015	2.74	2.355	0.873	1.292	3.728	0.805	21.14	0.00518	20.80
		0.075	0.70	0.09375	1.003	3.41	7.659	2.188	2.76	1.607	0.592	1.266	2.612	0.796	20.40	0.00188	13.70
		0.060	0.60	0.09375	0.795	2.70	6.099	1.742	2.77	1.227	0.451	1.242	2.033	0.785	19.92	0.000954	10.40
6.0	2.50	0.135	0.82	0.1875	1.574	5.35	8.575	2.858	2.33	2.156	0.886	1.170	3.236	0.716	22.62	0.00956	13.90
		0.105	0.82	0.1875	1.240	4.22	6.843	2.281	2.35	1.758	0.718	1.191	2.611	0.728	22.88	0.00456	11.40
		0.075	0.82	0.09375	0.909	3.09	5.124	1.708	2.37	1.353	0.549	1.220	1.977	0.746	23.18	0.00170	8.53
		0.060	0.60	0.09375	0.705	2.40	4.012	1.337	2.39	0.950	0.385	1.161	1.463	0.718	21.85	0.000846	5.87
5.0	2.00	0.135	0.70	0.1875	1.272	4.32	4.684	1.874	1.92	1.071	0.554	0.917	1.681	0.567	21.47	0.00773	4.90
		0.105	0.70	0.1875	1.005	3.42	3.761	1.504	1.93	0.884	0.454	0.938	1.369	0.579	21.79	0.00369	4.06
		0.075	0.60	0.09375	0.726	2.47	2.797	1.119	1.96	0.637	0.325	0.937	0.998	0.583	21.38	0.00136	2.80
		0.060	0.50	0.09375	0.573	1.95	2.227	0.891	1.97	0.480	0.244	0.915	0.772	0.573	20.73	0.000687	2.09
		0.048	0.50	0.09375	0.461	1.57	1.804	0.722	1.98	0.393	0.199	0.924	0.629	0.577	20.86	0.000354	1.72
4.0	2.00	0.135	0.70	0.1875	1.137	3.87	2.752	1.376	1.56	1.071	0.554	0.970	1.315	0.555	28.70	0.00691	3.00
		0.105	0.70	0.1875	0.900	3.06	2.219	1.110	1.57	0.884	0.454	0.991	1.072	0.566	29.04	0.00331	2.50
		0.075	0.60	0.09375	0.651	2.21	1.664	0.832	1.60	0.637	0.325	0.990	0.786	0.570	28.43	0.00122	1.72
		0.060	0.50	0.09375	0.513	1.74	1.330	0.665	1.61	0.480	0.244	0.967	0.610	0.561	27.57	0.000615	1.27
		0.048	0.50	0.09375	0.413	1.40	1.079	0.539	1.62	0.393	0.199	0.976	0.498	0.565	27.72	0.000317	1.05
3.5	2.00	0.135	0.70	0.1875	1.069	3.64	2.003	1.145	1.37	1.071	0.554	1.001	1.131	0.541	33.80	0.00650	2.24
		0.105	0.70	0.1875	0.847	2.88	1.620	0.926	1.38	0.884	0.454	1.021	0.924	0.552	34.13	0.00311	1.87
		0.075	0.60	0.09375	0.613	2.08	1.222	0.698	1.41	0.637	0.325	1.020	0.680	0.556	33.37	0.00115	1.28
		0.060	0.50	0.09375	0.483	1.64	0.979	0.559	1.42	0.480	0.244	0.997	0.529	0.546	32.38	0.000579	0.947
		0.048	0.50	0.09375	0.389	1.32	0.795	0.454	1.43	0.393	0.199	1.005	0.432	0.550	32.52	0.000299	0.780
3.0	1.75	0.105	0.70	0.1875	0.742	2.52	1.017	0.678	1.17	0.618	0.364	0.913	0.611	0.486	35.96	0.00273	0.990
		0.075	0.53	0.09375	0.526	1.79	0.767	0.512	1.21	0.418	0.244	0.890	0.437	0.480	34.11	0.000989	0.617
		0.060	0.53	0.09375	0.426	1.45	0.628	0.418	1.21	0.346	0.201	0.900	0.360	0.485	34.30	0.000512	0.514
		0.048	0.41	0.09375	0.332	1.13	0.499	0.332	1.22	0.251	0.145	0.868	0.274	0.472	32.82	0.000255	0.362

TABLE 4 | TABLE 4

Z-SECTION WITH UNSTIFFENED FLANGES

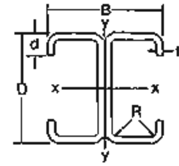


See notes on page V-25

Size		t	R	Area	Wgt. per Foot	Properties of Full Section										
						Axis x-x			Axis y-y			I_{xy}	Axis x_2-x_2	90°-θ	J	C_w
D	B					I_x	S_x	r_x	I_y	S_y	r_y		r_{min}			
In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.^{4}	In.	Deg.	In.^{4}	In.^{6}
8.0	2.000	0.135	0.1875	1.554	5.284	13.076	3.269	2.901	0.6496	0.3362	0.647	1.9872	0.467	8.87	0.00944	7.56
		0.105	0.1875	1.216	4.135	10.335	2.584	2.915	0.5171	0.2655	0.652	1.5752	0.472	8.90	0.00447	6.06
		0.075	0.09375	0.880	2.993	7.599	1.900	2.938	0.3779	0.1926	0.655	1.1453	0.477	8.80	0.00165	4.46
		0.060	0.09375	0.706	2.402	6.127	1.532	2.945	0.3058	0.1552	0.658	0.9249	0.479	8.81	0.000848	3.62
7.0	1.500	0.135	0.1875	1.284	4.366	7.840	2.240	2.471	0.2647	0.1848	0.454	0.9548	0.337	7.07	0.00780	2.43
		0.105	0.1875	1.006	3.421	6.218	1.777	2.486	0.2124	0.1467	0.459	0.7612	0.342	7.11	0.00370	1.96
		0.075	0.09375	0.730	2.483	4.603	1.315	2.511	0.1564	0.1070	0.463	0.5561	0.347	7.02	0.00137	1.46
		0.060	0.09375	0.586	1.994	3.718	1.062	2.518	0.1271	0.0864	0.465	0.4503	0.349	7.04	0.000704	1.19
6.0	1.500	0.135	0.1875	1.149	3.907	5.334	1.778	2.155	0.2647	0.1840	0.480	0.8157	0.345	8.92	0.00698	1.72
		0.105	0.1875	0.901	3.064	4.240	1.413	2.169	0.2124	0.1467	0.485	0.6508	0.349	8.95	0.00331	1.39
		0.075	0.09375	0.655	2.228	3.150	1.050	2.193	0.1564	0.1070	0.489	0.4758	0.355	8.82	0.00123	1.03
		0.060	0.09375	0.526	1.790	2.547	0.849	2.200	0.1271	0.0864	0.491	0.3854	0.357	8.84	0.000632	0.842
		0.048	0.09375	0.423	1.437	2.055	0.685	2.205	0.1029	0.0697	0.493	0.3115	0.359	8.85	0.000325	0.685
5.0	1.250	0.105	0.1875	0.744	2.529	2.399	0.960	1.796	0.1203	0.1004	0.402	0.3704	0.288	9.00	0.00273	0.543
		0.075	0.09375	0.543	1.846	1.797	0.719	1.820	0.0891	0.0735	0.405	0.2719	0.294	8.83	0.00102	0.407
		0.060	0.09375	0.436	1.484	1.456	0.583	1.827	0.0726	0.0595	0.408	0.2209	0.296	8.85	0.000524	0.333
		0.048	0.09375	0.351	1.192	1.177	0.471	1.832	0.0590	0.0481	0.410	0.1789	0.298	8.87	0.000269	0.272
4.0	1.500	0.060	0.09375	0.406	1.382	0.965	0.483	1.546	0.1271	0.0865	0.561	0.2558	0.370	15.74	0.000485	0.335
4.0	1.125	0.105	0.1875	0.613	2.083	1.286	0.643	1.449	0.0864	0.0806	0.376	0.2367	0.260	10.77	0.00225	0.240
		0.075	0.09375	0.449	1.527	0.973	0.486	1.472	0.0643	0.0591	0.378	0.1744	0.267	10.50	0.000842	0.181
		0.060	0.09375	0.361	1.229	0.791	0.395	1.479	0.0525	0.0480	0.381	0.1420	0.269	10.52	0.000434	0.149
		0.048	0.09375	0.291	0.988	0.640	0.320	1.485	0.0427	0.0388	0.383	0.1151	0.271	10.54	0.000223	0.122
3.0	1.500	0.060	0.09375	0.346	1.178	0.494	0.330	1.198	0.1271	0.0865	0.607	0.1906	0.365	23.06	0.000413	0.171
3.0	1.125	0.105	0.1875	0.508	1.726	0.636	0.424	1.120	0.0864	0.0806	0.413	0.1759	0.263	16.30	0.00187	0.123
		0.075	0.09375	0.374	1.272	0.487	0.324	1.141	0.0643	0.0591	0.415	0.1300	0.271	15.80	0.000701	0.0936
		0.060	0.09375	0.301	1.025	0.397	0.265	1.147	0.0525	0.0480	0.417	0.1059	0.273	15.80	0.000362	0.0771
		0.048	0.09375	0.243	0.825	0.322	0.215	1.153	0.0427	0.0388	0.420	0.0860	0.275	15.80	0.000186	0.0632
2.0	1.125	0.105	0.1875	0.403	1.369	0.241	0.241	0.773	0.0864	0.0806	0.463	0.1150	0.250	28.07	0.00148	0.0467
		0.075	0.09375	0.299	1.017	0.188	0.188	0.792	0.0643	0.0591	0.464	0.0855	0.262	27.12	0.000561	0.0359
		0.060	0.09375	0.241	0.821	0.154	0.154	0.798	0.0525	0.0480	0.466	0.0699	0.264	27.03	0.000290	0.0298
		0.048	0.09375	0.195	0.661	0.126	0.126	0.804	0.0427	0.0388	0.469	0.0569	0.266	26.95	0.000149	0.0245
1.5	1.500	0.048	0.09375	0.207	0.702	0.084	0.112	0.641	0.1029	0.0697	0.709	0.0760	0.287	41.53	0.000159	0.0270
		0.036	0.09375	0.156	0.530	0.0652	0.0862	0.646	0.0781	0.0527	0.710	0.0580	0.291	41.70	0.0000670	0.0209

TABLE 5 | TABLE 5

TWO CHANNELS WITH STIFFENED FLANGES BACK-TO-BACK

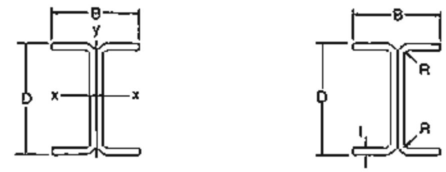


See notes on page V-25

Size		t	d	R	Area	Wgt. per Foot	Properties of Full Section									
							Axis x-x			Axis y-y			J	C_w		
D	B						I_x	S_x	r_x	I_y	S_y	r_y			I_y	r_y
In.	In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{6}	In.^{4}	In.
12.0	7.00	0.135	1.01	0.1875	5.411	18.40	113.0	18.756	4.56	12.578	3.594	1.525	0.0329	478.00	44.31	2.86
		0.105	0.90	0.1875	4.193	14.26	87.7	14.612	4.57	9.448	2.699	1.501	0.0154	361.00	34.90	2.88
10.0	7.00	0.135	1.01	0.1875	4.871	16.56	73.1	14.611	3.87	12.575	3.593	1.607	0.0296	336.00	37.95	2.79
		0.105	0.90	0.1875	3.773	12.83	57.0	11.402	3.89	9.447	2.699	1.582	0.0139	253.00	29.91	2.82
		0.075	0.72	0.09375	2.687	9.14	41.1	8.213	3.91	6.403	1.829	1.544	0.00504	167.00	21.79	2.85
9.0	6.50	0.135	1.00	0.1875	4.461	15.17	54.3	12.070	3.49	10.299	3.169	1.519	0.0271	225.00	29.43	2.57
		0.105	0.84	0.1875	3.433	11.67	42.2	9.368	3.50	7.511	2.311	1.479	0.0126	163.00	23.22	2.60
		0.075	0.70	0.09375	2.456	8.35	30.6	6.794	3.53	5.192	1.597	1.454	0.00461	110.00	16.96	2.63
		0.060	0.61	0.09375	1.952	6.64	24.4	5.414	3.53	3.981	1.225	1.428	0.00234	84.00	13.66	2.65
8.0	6.00	0.135	0.93	0.1875	4.018	13.66	38.7	9.686	3.11	8.032	2.677	1.414	0.0244	138.00	22.28	2.35
		0.105	0.81	0.1875	3.105	10.56	30.3	7.563	3.12	5.968	1.989	1.386	0.0114	103.00	17.60	2.38
		0.075	0.70	0.09375	2.231	7.59	22.1	5.523	3.15	4.196	1.399	1.371	0.00418	70.70	12.88	2.40
		0.060	0.60	0.09375	1.770	6.02	17.6	4.395	3.15	3.189	1.063	1.342	0.00212	53.50	10.39	2.42
7.0	5.50	0.135	0.88	0.1875	3.586	12.19	26.5	7.581	2.72	6.202	2.255	1.315	0.0218	82.30	16.38	2.14
		0.105	0.88	0.1875	2.820	9.59	21.1	6.030	2.74	4.957	1.803	1.326	0.0104	67.20	12.96	2.14
		0.075	0.70	0.09375	2.006	6.82	15.3	4.377	2.76	3.333	1.212	1.289	0.00376	43.50	9.51	2.18
		0.060	0.60	0.09375	1.590	5.40	12.2	3.485	2.77	2.527	0.919	1.261	0.00191	32.70	7.68	2.20
6.0	5.00	0.135	0.82	0.1875	3.149	10.71	17.2	5.717	2.33	4.641	1.856	1.214	0.0191	45.50	11.60	1.92
		0.105	0.82	0.1875	2.480	8.43	13.7	4.562	2.35	3.720	1.488	1.225	0.00911	37.30	9.20	1.93
		0.075	0.82	0.09375	1.817	6.18	10.2	3.416	2.37	2.814	1.125	1.244	0.00341	28.10	6.78	1.93
		0.060	0.60	0.09375	1.410	4.79	8.02	2.674	2.39	1.962	0.785	1.180	0.00169	18.90	5.48	1.97
5.0	4.00	0.135	0.70	0.1875	2.544	8.65	9.37	3.747	1.92	2.351	1.175	0.961	0.0155	15.90	5.98	1.53
		0.105	0.70	0.1875	2.009	6.83	7.52	3.009	1.93	1.898	0.949	0.972	0.00738	13.20	4.76	1.54
		0.075	0.60	0.09375	1.451	4.93	5.59	2.238	1.96	1.340	0.670	0.961	0.00272	9.05	3.53	1.56
		0.060	0.50	0.09375	1.146	3.90	4.45	1.781	1.97	0.999	0.500	0.934	0.00137	6.68	2.86	1.58
		0.048	0.50	0.09375	0.922	3.14	3.61	1.443	1.98	0.812	0.406	0.939	0.000708	5.49	2.31	1.58
4.0	4.00	0.135	0.70	0.1875	2.274	7.73	5.50	2.752	1.56	2.349	1.175	1.016	0.0138	10.40	4.97	1.48
		0.105	0.70	0.1875	1.799	6.12	4.44	2.219	1.57	1.897	0.949	1.027	0.00661	8.69	3.97	1.48
		0.075	0.60	0.09375	1.301	4.42	3.33	1.664	1.60	1.340	0.670	1.015	0.00244	5.91	2.96	1.51
		0.060	0.50	0.09375	1.026	3.49	2.66	1.330	1.61	0.999	0.500	0.987	0.00123	4.34	2.40	1.53
		0.048	0.50	0.09375	0.826	2.81	2.16	1.079	1.62	0.812	0.406	0.992	0.000634	3.57	1.94	1.53
3.5	4.00	0.135	0.70	0.1875	2.139	7.27	4.01	2.289	1.37	2.349	1.174	1.048	0.0130	8.09	4.46	1.44
		0.105	0.70	0.1875	1.694	5.76	3.24	1.852	1.38	1.897	0.949	1.058	0.00623	6.77	3.57	1.45
		0.075	0.60	0.09375	1.226	4.17	2.44	1.397	1.41	1.340	0.670	1.046	0.00230	4.59	2.67	1.48
		0.060	0.50	0.09375	0.966	3.28	1.96	1.119	1.42	0.999	0.500	1.017	0.00116	3.36	2.16	1.50
		0.048	0.50	0.09375	0.778	2.65	1.59	0.909	1.43	0.812	0.406	1.022	0.000598	2.77	1.75	1.50
3.0	3.50	0.105	0.70	0.1875	1.484	5.05	2.03	1.356	1.17	1.340	0.766	0.950	0.00545	3.64	2.31	1.25
		0.075	0.53	0.09375	1.055	3.59	1.53	1.023	1.21	0.885	0.506	0.916	0.00198	2.22	1.74	1.28
		0.060	0.53	0.09375	0.853	2.90	1.26	0.837	1.21	0.724	0.414	0.921	0.00102	1.86	1.41	1.29
		0.048	0.41	0.09375	0.665	2.26	0.997	0.665	1.22	0.520	0.297	0.885	0.000511	1.29	1.14	1.31

TABLE 6 TABLE 6

2 CHANNELS WITH UNSTIFFENED FLANGES BACK-TO-BACK



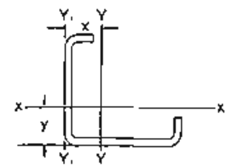
See notes on page V-25

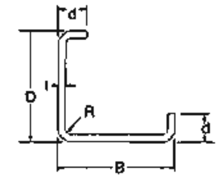
Size		t	R	Area	Weight per Foot	Properties of Full Section							
						Axis x-x			Axis y-y			J	C_w
D	B					I_x	S_x	r_x	I_y	S_y	r_y		
In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{6}
8.0	4.00	0.135	0.1875	3.108	10.57	26.151	6.538	2.901	1.4499	0.7249	0.683	0.0189	20.10
		0.105	0.1875	2.433	8.27	20.670	5.168	2.915	1.1247	0.5624	0.680	0.00894	16.10
		0.075	0.09375	1.761	5.99	15.198	3.799	2.938	0.8017	0.4008	0.675	0.00330	11.90
		0.060	0.09375	1.413	4.80	12.254	3.064	2.945	0.6409	0.3204	0.673	0.00170	9.64
7.0	3.00	0.135	0.1875	2.568	8.73	15.680	4.480	2.471	0.6162	0.4108	0.490	0.0156	6.23
		0.105	0.1875	2.013	6.84	12.436	3.553	2.486	0.4767	0.3178	0.487	0.00740	5.05
		0.075	0.09375	1.461	4.97	9.206	2.630	2.511	0.3390	0.2260	0.482	0.00274	3.75
		0.060	0.09375	1.173	3.99	7.435	2.124	2.518	0.2708	0.1805	0.480	0.00141	3.06
6.0	3.00	0.135	0.1875	2.298	7.81	10.669	3.556	2.155	0.6149	0.4100	0.517	0.0140	4.55
		0.105	0.1875	1.803	6.13	8.481	2.827	2.169	0.4761	0.3174	0.514	0.00662	3.69
		0.075	0.09375	1.311	4.46	6.300	2.100	2.193	0.3388	0.2258	0.508	0.00246	2.75
		0.060	0.09375	1.053	3.58	5.094	1.698	2.200	0.2707	0.1804	0.507	0.00126	2.24
		0.048	0.09375	0.845	2.87	4.110	1.370	2.205	0.2163	0.1442	0.506	0.000649	1.82
5.0	2.50	0.105	0.1875	1.488	5.06	4.798	1.919	1.796	0.2764	0.2212	0.431	0.00547	1.44
		0.075	0.09375	1.086	3.69	3.595	1.438	1.820	0.1964	0.1571	0.425	0.00204	1.08
		0.060	0.09375	0.873	2.97	2.913	1.165	1.827	0.1568	0.1254	0.424	0.00105	0.886
		0.048	0.09375	0.701	2.38	2.354	0.942	1.832	0.1253	0.1002	0.423	0.000538	0.723
4.0	2.25	0.105	0.1875	1.225	4.17	2.572	1.286	1.449	0.2018	0.1793	0.406	0.00450	0.655
		0.075	0.09375	0.898	3.05	1.946	0.973	1.472	0.1432	0.1273	0.399	0.00168	0.495
		0.060	0.09375	0.723	2.46	1.581	0.791	1.479	0.1143	0.1016	0.398	0.000867	0.408
		0.048	0.09375	0.581	1.98	1.281	0.640	1.485	0.0914	0.0812	0.397	0.000446	0.334
3.0	2.25	0.105	0.1875	1.015	3.45	1.273	0.849	1.120	0.2012	0.1788	0.445	0.00373	0.362
		0.075	0.09375	0.748	2.54	0.973	0.649	1.141	0.1430	0.1271	0.437	0.00140	0.275
		0.060	0.09375	0.603	2.05	0.794	0.529	1.147	0.1142	0.1015	0.435	0.000723	0.227
		0.048	0.09375	0.485	1.65	0.645	0.430	1.153	0.0913	0.0812	0.434	0.000373	0.186
2.0	2.25	0.105	0.1875	0.805	2.74	0.482	0.482	0.773	0.2006	0.1783	0.499	0.00296	0.155
		0.075	0.09375	0.598	2.03	0.375	0.375	0.792	0.1428	0.1269	0.489	0.00112	0.119
		0.060	0.09375	0.483	1.64	0.308	0.308	0.798	0.1141	0.1014	0.486	0.000579	0.0988
		0.048	0.09375	0.389	1.32	0.251	0.251	0.804	0.0912	0.0811	0.484	0.000299	0.0814

TABLE 7 TABLE 7

EQUAL LEG ANGLE WITH STIFFENED FLANGES

See notes on page V-25



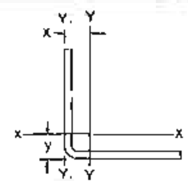


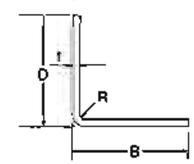
Size		t	d	R	Area	Wgt. per Foot	Properties of Full Section											
							Axis x-x and Axis y-y				r_{y_1}	J	C_w	r_o				
D	B						I	S	r	x = y					I_{y_2}	r_{y_2}	j	x_o
In.	In.	In.	In.	In.	In.²	Lb.	In.⁴	In.³	In.	In.	In.	In.⁴	In.⁶	In.	In.⁴	In.	In.	In.
4.0	4.0	0.135	1.10	0.1875	1.278	4.345	2.594	0.957	1.425	1.290	1.922	0.00776	1.1058	2.80	1.2445	0.987	3.11	−1.940
3.0	3.0	0.135	0.93	0.1875	0.962	3.271	1.083	0.541	1.061	0.999	1.457	0.00584	0.3503	2.12	0.5399	0.749	2.33	−1.498
		0.105	0.81	0.1875	0.735	2.498	0.828	0.407	1.062	0.964	1.434	0.00270	0.1910	2.07	0.3944	0.733	2.33	−1.423
2.5	2.5	0.135	0.82	0.1875	0.797	2.711	0.613	0.371	0.876	0.847	1.219	0.00484	0.1579	1.77	0.3114	0.625	1.93	−1.258
		0.105	0.82	0.1875	0.632	2.148	0.500	0.302	0.889	0.843	1.226	0.00232	0.1335	1.80	0.2547	0.635	1.95	−1.284
2.0	2.0	0.135	0.82	0.1875	0.662	2.252	0.325	0.255	0.700	0.725	1.008	0.00402	0.0972	1.49	0.1829	0.525	1.56	−1.119
		0.105	0.82	0.1875	0.527	1.791	0.268	0.209	0.713	0.721	1.014	0.00194	0.0828	1.53	0.1510	0.535	1.58	−1.144
		0.075	0.60	0.09375	0.360	1.226	0.184	0.137	0.714	0.659	0.972	0.000676	0.0244	1.43	0.0907	0.502	1.56	−1.005
		0.060	0.50	0.09375	0.280	0.951	0.141	0.103	0.710	0.631	0.950	0.000336	0.0118	1.37	0.0658	0.485	1.55	−0.936

TABLE 8 TABLE 8

EQUAL LEG ANGLE WITH UNSTIFFENED FLANGES

See notes on page V-25

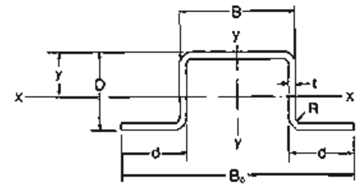




Size		t	R	Area	Wgt. per Foot	Properties of Full Section											
						Axis x-x and Axis y-y				r_{y_1}	J	C_w	r_o	Axis y			
D	B					I	S	r	x = y					I_{y_2}	r_{y_2}	j	x_o
In.	In.	In.	In.	In.²	Lb.	In.⁴	In.³	In.	In.	In.	In.⁴	In.⁶	In.	In.⁴	In.	In.	In.
4.0	4.0	0.135	0.1875	1.047	3.560	1.6947	0.5774	1.272	1.065	1.659	0.00636	0.000	2.22	0.6529	0.790	2.76	−1.305
3.0	3.0	0.135	0.1875	0.777	2.642	0.7003	0.3206	0.949	0.815	1.251	0.00472	0.000	1.65	0.2659	0.585	2.06	−0.952
		0.105	0.1875	0.608	2.068	0.5536	0.2520	0.954	0.803	1.247	0.00223	0.000	1.66	0.2110	0.589	2.07	−0.962
2.5	2.5	0.135	0.1875	0.642	2.183	0.3985	0.2202	0.788	0.691	1.048	0.00390	0.000	1.36	0.1494	0.482	1.71	−0.776
		0.105	0.1875	0.503	1.711	0.3161	0.1735	0.793	0.678	1.043	0.00185	0.000	1.37	0.1191	0.486	1.72	−0.786
2.0	2.0	0.135	0.1875	0.507	1.724	0.1989	0.1387	0.626	0.566	0.844	0.00308	0.000	1.07	0.0731	0.380	1.36	−0.599
		0.105	0.1875	0.398	1.354	0.1585	0.1096	0.631	0.554	0.840	0.00146	0.000	1.08	0.0586	0.384	1.37	−0.609
		0.075	0.09375	0.290	0.987	0.1170	0.0799	0.635	0.536	0.831	0.000544	0.000	1.11	0.0450	0.394	1.38	−0.650
		0.060	0.09375	0.233	0.793	0.0947	0.0644	0.637	0.530	0.829	0.000280	0.000	1.11	0.0365	0.396	1.38	−0.655

TABLE 9 TABLE 9

HAT SECTIONS

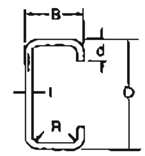


See notes on page V-25

Size D	Size B	t	d	R	Area	Wgt per Foot	I_x	S_x	r_x	I_y	S_y	r_y	y	J	C_w	j	r_o	x_o
In.	In.	In.	In.	In.	In.^{2}	Lb.	In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.	In.	In.^{4}	In.^{6}	In.	In.	In.
10.0	15.00	0.135	1.670	0.1875	5.044	17.149	67.50	10.3279	3.659	210.00	23.2710	6.456	3.461	0.0306	2340.00	11.21	10.87	−7.95
		0.105	1.340	0.1875	3.869	13.155	50.20	7.5430	3.602	160.00	18.2608	6.421	3.346	0.0142	1810.00	11.06	10.72	−7.79
10.0	10.00	0.105	1.340	0.1875	3.344	11.370	43.60	7.1050	3.611	66.80	10.7092	4.468	3.863	0.0123	685.00	10.16	10.29	−8.54
		0.075	1.050	0.09375	2.368	8.052	30.20	4.8420	3.572	46.90	7.8514	4.451	3.758	0.00444	493.00	9.99	10.14	−8.38
10.0	5.00	0.075	1.050	0.09375	1.993	6.777	24.10	4.3405	3.474	11.00	3.1722	2.352	4.458	0.00374	98.40	9.74	10.17	−9.27
8.0	12.00	0.135	1.670	0.1875	4.099	13.936	36.30	7.0912	2.977	111.00	14.7525	5.208	2.878	0.0249	762.00	9.13	8.85	−6.51
		0.105	1.340	0.1875	3.134	10.656	26.80	5.1291	2.926	83.60	11.5586	5.166	2.767	0.0115	597.00	9.00	8.71	−6.37
8.0	8.00	0.105	1.340	0.1875	2.714	9.228	23.30	4.8340	2.928	35.20	6.7247	3.602	3.188	0.00997	222.00	8.24	8.35	−6.94
		0.075	0.980	0.09375	1.908	6.486	15.80	3.2011	2.879	24.30	4.9542	3.569	3.059	0.00358	161.00	8.06	8.18	−6.78
8.0	4.00	0.075	0.980	0.09375	1.608	5.466	12.60	2.8710	2.796	5.74	1.9756	1.889	3.623	0.00301	319.00	7.80	8.17	−7.44
		0.060	0.840	0.09375	1.274	4.330	9.82	2.2149	2.777	4.51	1.6215	1.881	3.566	0.00153	25.80	7.80	8.14	−7.42
6.0	9.00	0.135	1.670	0.1875	3.154	10.723	16.40	4.4300	2.283	49.70	8.2408	3.971	2.290	0.0192	177.00	7.03	6.79	−5.02
		0.105	1.340	0.1875	2.399	8.157	12.10	3.1629	2.243	36.80	6.4229	3.918	2.185	0.00882	140.00	6.92	6.68	−4.92
6.0	6.00	0.105	1.340	0.1875	2.084	7.086	10.40	2.9828	2.236	15.70	3.7063	2.744	2.507	0.00766	51.60	6.28	6.37	−5.30
		0.075	0.915	0.09375	1.448	4.923	6.91	1.8989	2.185	10.50	2.7299	2.691	2.360	0.00271	37.70	6.12	6.22	−5.16
6.0	3.00	0.075	0.915	0.09375	1.223	4.158	5.47	1.7042	2.116	2.51	1.0707	1.431	2.787	0.00229	75.50	5.84	6.14	−5.58
		0.060	0.760	0.09375	0.964	3.278	4.24	1.2947	2.097	1.94	0.8812	1.418	2.726	0.00116	60.40	5.86	6.13	−5.59
		0.048	0.660	0.09375	0.764	2.599	3.32	1.0019	2.084	1.53	0.7221	1.412	2.686	0.000587	48.80	5.86	6.12	−5.58
4.0	6.00	0.135	1.670	0.1875	2.209	7.510	5.42	2.3437	1.567	16.90	3.7178	2.763	1.686	0.0134	22.60	4.86	4.66	−3.41
		0.105	1.340	0.1875	1.664	5.658	3.96	1.6444	1.543	12.00	2.8447	2.691	1.592	0.00612	17.90	4.79	4.60	−3.40
4.0	4.00	0.105	1.340	0.1875	1.454	4.944	3.39	1.5510	1.527	5.31	1.6409	1.911	1.814	0.00534	6.89	4.22	4.31	−3.54
		0.075	0.915	0.09375	0.998	3.393	2.23	0.9602	1.494	3.35	1.1788	1.832	1.680	0.00187	4.83	4.17	4.25	−3.53
4.0	2.00	0.075	0.915	0.09375	0.848	2.883	1.75	0.8631	1.437	0.841	0.4572	0.996	1.970	0.00159	1.15	3.77	4.02	−3.62
		0.060	0.750	0.09375	0.663	2.254	1.34	0.6428	1.423	0.626	0.3704	0.972	1.911	0.000795	0.829	3.86	4.08	−3.70
		0.048	0.618	0.09375	0.520	1.769	1.03	0.4831	1.409	0.476	0.3032	0.956	1.861	0.000400	0.640	3.91	4.10	−3.73
3.0	4.50	0.135	1.670	0.1875	1.736	5.904	2.47	1.5155	1.192	8.28	2.1885	2.184	1.371	0.0105	5.66	3.73	3.55	−2.53
		0.105	1.340	0.1875	1.297	4.408	1.80	1.0526	1.179	5.69	1.6327	2.095	1.287	0.00476	4.20	3.69	3.52	−2.57
3.0	3.00	0.105	1.340	0.1875	1.139	3.873	1.53	0.9921	1.159	2.62	0.9563	1.515	1.457	0.00419	1.89	3.12	3.20	−2.57
		0.075	0.915	0.09375	0.773	2.628	1.01	0.6033	1.140	1.54	0.6594	1.413	1.334	0.00145	1.14	3.15	3.22	−2.66
3.0	1.50	0.075	0.915	0.09375	0.660	2.245	0.784	0.5423	1.089	0.415	0.2611	0.793	1.555	0.00124	0.371	2.64	2.87	−2.53
		0.060	0.750	0.09375	0.513	1.744	0.599	0.3995	1.081	0.296	0.2054	0.759	1.500	0.000615	0.235	2.79	2.98	−2.68
		0.048	0.618	0.09375	0.400	1.361	0.459	0.2970	1.071	0.218	0.1650	0.738	1.453	0.000308	0.164	2.88	3.05	−2.76
2.0	4.00	0.105	1.340	0.1875	1.034	3.516	0.670	0.5948	0.805	3.72	1.1484	1.896	0.873	0.00380	1.13	3.08	2.64	−1.65
		0.075	0.915	0.09375	0.698	2.373	0.432	0.3531	0.787	2.19	0.7719	1.772	0.776	0.00131	0.799	2.85	2.54	−1.64
2.0	2.00	0.075	0.915	0.09375	0.548	1.863	0.328	0.3210	0.774	0.563	0.3062	1.014	0.978	0.00103	0.180	2.06	2.12	−1.70
		0.060	0.750	0.09375	0.423	1.438	0.249	0.2333	0.768	0.400	0.2368	0.973	0.931	0.000507	0.127	2.10	2.15	−1.75
2.0	1.00	0.060	0.750	0.09375	0.363	1.234	0.193	0.2093	0.728	0.116	0.0975	0.565	1.080	0.000435	0.0542	1.62	1.80	−1.55
		0.048	0.618	0.09375	0.280	0.953	0.147	0.1534	0.725	0.0795	0.0743	0.532	1.039	0.000215	0.0321	1.76	1.91	−1.69
1.5	3.00	0.105	1.340	0.1875	0.824	2.802	0.303	0.3892	0.606	1.96	0.7149	1.540	0.722	0.00303	0.312	2.47	2.04	−1.19
		0.075	0.915	0.09375	0.548	1.863	0.198	0.2293	0.601	1.06	0.4537	1.392	0.636	0.00103	0.187	2.24	1.96	−1.24
1.5	1.50	0.075	0.915	0.09375	0.435	1.480	0.147	0.2078	0.582	0.301	0.1893	0.831	0.791	0.000816	0.0606	1.49	1.55	−1.17
		0.060	0.750	0.09375	0.333	1.132	0.112	0.1500	0.581	0.202	0.1406	0.780	0.750	0.000399	0.0376	1.54	1.58	−1.25
1.5	0.75	0.060	0.750	0.09375	0.288	0.979	0.0855	0.1341	0.545	0.0674	0.0633	0.484	0.863	0.000345	0.0210	1.01	1.20	−0.953
		0.048	0.618	0.09375	0.220	0.749	0.0656	0.0975	0.546	0.0430	0.0455	0.442	0.827	0.000169	0.0122	1.15	1.31	−1.10

TABLE 10 TABLE 10

CHANNEL WITH STIFFENED FLANGES

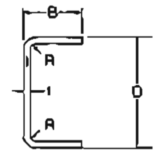


See notes on page V-25

Size D	Size B	t	d	R	Effective Section Properties: I_x, In.[4] $F_y=33$ksi	I_x, In.[4] $F_y=50$ksi	S_x, In.[3] $F_y=33$ksi	S_x, In.[3] $F_y=50$ksi
In.	In.	In.	In.	In.				
12.0	3.50	0.135	1.01	0.1875	56.266	55.387	9.378	9.134
		0.105	0.90	0.1875	43.231	40.904	7.139	6.546
10.0	3.50	0.135	1.01	0.1875	36.525	35.956	7.305	7.110
		0.105	0.90	0.1875	28.110	26.513	5.567	5.071
		0.075	0.72	0.09375	18.864	17.570	3.586	3.201
9.0	3.25	0.135	1.00	0.1875	27.157	27.157	6.035	6.035
		0.105	0.84	0.1875	20.917	20.029	4.623	4.307
		0.075	0.70	0.09375	14.234	13.551	3.027	2.807
		0.060	0.61	0.09375	10.860	9.703	2.257	1.874
8.0	3.00	0.135	0.93	0.1875	19.371	19.371	4.843	4.843
		0.105	0.81	0.1875	15.125	14.726	3.781	3.613
		0.075	0.70	0.09375	10.435	9.965	2.516	2.341
		0.060	0.60	0.09375	7.972	7.397	1.879	1.667
8.0	1.625	0.105	0.82	0.1875	10.647	10.647	2.662	2.662
		0.075	0.82	0.09375	7.996	7.996	1.999	1.999
		0.060	0.60	0.09375	6.190	6.190	1.548	1.548
		0.048	0.50	0.09375	4.889	4.447	1.222	1.041
7.0	2.75	0.135	0.88	0.1875	13.265	13.265	3.790	3.790
		0.105	0.88	0.1875	10.553	10.548	3.015	3.013
		0.075	0.70	0.09375	7.506	7.033	2.113	1.904
		0.060	0.60	0.09375	5.648	5.368	1.538	1.422
6.0	2.50	0.135	0.82	0.1875	8.574	8.574	2.858	2.858
		0.105	0.82	0.1875	6.843	6.843	2.281	2.281
		0.075	0.82	0.09375	5.124	4.944	1.708	1.607
		0.060	0.60	0.09375	3.766	3.606	1.205	1.125
6.0	1.625	0.105	0.82	0.1875	5.246	5.246	1.749	1.749
		0.075	0.82	0.09375	3.972	3.972	1.324	1.324
		0.060	0.60	0.09375	3.085	3.085	1.028	1.028
		0.048	0.50	0.09375	2.441	2.334	0.814	0.757
5.0	2.00	0.135	0.70	0.1875	4.683	4.683	1.873	1.873
		0.105	0.70	0.1875	3.760	3.760	1.504	1.504
		0.075	0.60	0.09375	2.797	2.760	1.119	1.093
		0.060	0.50	0.09375	2.190	2.053	0.865	0.780
		0.048	0.50	0.09375	1.710	1.632	0.660	0.614
4.0	2.00	0.135	0.70	0.1875	2.751	2.751	1.375	1.375
		0.105	0.70	0.1875	2.218	2.218	1.109	1.109
		0.075	0.60	0.09375	1.664	1.644	0.832	0.813
		0.060	0.50	0.09375	1.309	1.223	0.646	0.578
		0.048	0.50	0.09375	1.022	0.972	0.492	0.454
4.0	1.625	0.075	0.60	0.09375	1.448	1.448	0.724	0.724
		0.060	0.50	0.09375	1.165	1.143	0.577	0.566
		0.048	0.50	0.09375	0.938	0.895	0.469	0.433
		0.036	0.50	0.09375	0.695	0.662	0.341	0.316
3.625	1.625	0.075	0.60	0.09375	1.148	1.148	0.633	0.633
		0.060	0.50	0.09375	0.918	0.908	0.506	0.497
		0.048	0.50	0.09375	0.746	0.712	0.412	0.380
		0.036	0.50	0.09375	0.553	0.527	0.299	0.276
3.5	2.00	0.135	0.70	0.1875	2.002	2.002	1.144	1.144
		0.105	0.70	0.1875	1.619	1.619	0.925	0.925
		0.075	0.60	0.09375	1.222	1.208	0.698	0.683
		0.060	0.50	0.09375	0.964	0.899	0.544	0.484
		0.048	0.50	0.09375	0.753	0.715	0.413	0.380
3.0	1.75	0.105	0.70	0.1875	1.016	1.016	0.678	0.678
		0.075	0.53	0.9375	0.767	0.767	0.512	0.512
		0.060	0.53	0.9375	0.628	0.617	0.418	0.405
		0.048	0.41	0.9375	0.485	0.442	0.317	0.272
2.5	1.625	0.075	0.60	0.9375	0.481	0.481	0.384	0.384
		0.060	0.50	0.9375	0.388	0.385	0.311	0.305
		0.048	0.50	0.9375	0.317	0.303	0.254	0.233
		0.036	0.50	0.9375	0.236	0.224	0.184	0.169

TABLE 11 TABLE 11

CHANNEL WITH UNSTIFFENED FLANGES

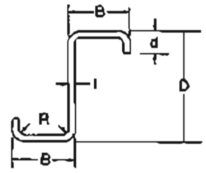


See notes on page V-25

Size		t	d	R	Effective Section Properties			
D	B				I_x, In.^{4}		S_x, In.^{3}	
In.	In.	In.	In.	In.	$F_y = 33$ksi	$F_y = 50$ksi	$F_y = 33$ksi	$F_y = 50$ksi
8.0	2.00	0.135	0	0.1875	13.075	12.690	3.269	3.123
		0.105	0	0.1875	10.335	9.580	2.584	2.305
		0.075	0	0.09375	6.764	6.519	1.596	1.513
		0.060	0	0.09375	5.231	4.819	1.211	1.066
7.0	1.50	0.135	0	0.1875	7.840	7.840	2.240	2.240
		0.105	0	0.1875	6.218	6.119	1.776	1.734
		0.075	0	0.09375	4.366	4.219	1.215	1.155
		0.060	0	0.09375	3.391	3.278	0.926	0.882
6.0	1.50	0.135	0	0.1875	5.334	5.334	1.778	1.778
		0.105	0	0.1875	4.240	4.168	1.413	1.377
		0.075	0	0.09375	2.976	2.868	0.963	0.912
		0.060	0	0.09375	2.306	2.223	0.731	0.693
		0.048	0	0.09375	1.784	1.716	0.555	0.524
5.0	1.25	0.105	0	0.1875	2.399	2.399	0.959	0.959
		0.075	0	0.09375	1.760	1.700	0.696	0.661
		0.060	0	0.09375	1.370	1.320	0.531	0.503
		0.048	0	0.09375	1.062	1.024	0.403	0.382
4.0	1.125	0.105	0	0.1875	1.286	1.286	0.643	0.643
		0.075	0	0.09375	0.971	0.938	0.485	0.460
		0.060	0	0.09375	0.758	0.730	0.371	0.350
		0.048	0	0.09375	0.588	0.565	0.281	0.265
3.0	1.125	0.105	0	0.1875	0.636	0.636	0.424	0.424
		0.075	0	0.09375	0.485	0.467	0.323	0.304
		0.060	0	0.09375	0.379	0.363	0.246	0.230
		0.048	0	0.09375	0.293	0.280	0.185	0.173
2.0	1.125	0.105	0	0.1875	0.240	0.240	0.240	0.240
		0.075	0	0.09375	0.187	0.179	0.187	0.174
		0.060	0	0.09375	0.146	0.139	0.141	0.130
		0.048	0	0.09375	0.113	0.107	0.105	0.097

TABLE 12 TABLE 12

Z-SECTION WITH STIFFENED FLANGES

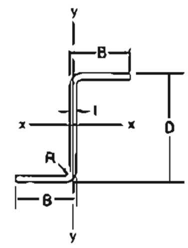


See notes on page V-25

Size		t	d	R	Effective Section Properties			
D	B				I_x, In.^{4}		S_x, In.^{3}	
In.	In.	In.	In.	In.	F_y=33ksi	F_y=50ksi	F_y=33ksi	F_y=50ksi
12.0	3.50	0.135	1.01	0.1875	56.266	55.387	9.378	9.134
		0.105	0.90	0.1875	43.231	40.904	7.139	6.546
10.0	3.50	0.135	1.01	0.1875	36.525	35.966	7.305	7.110
		0.105	0.90	0.1875	28.110	26.513	5.567	5.071
		0.075	0.72	0.09375	18.864	17.570	3.586	3.201
9.0	3.25	0.135	1.00	0.1875	27.157	27.157	6.035	6.035
		0.105	0.84	0.1875	20.917	20.029	4.623	4.307
		0.075	0.70	0.09375	14.234	13.551	3.027	2.807
		0.060	0.61	0.09375	10.860	9.703	2.257	1.874
8.0	3.00	0.135	0.93	0.1875	19.371	19.371	4.843	4.843
		0.105	0.81	0.1875	15.125	14.726	3.781	3.613
		0.075	0.70	0.09375	10.435	9.965	2.516	2.341
		0.060	0.60	0.09375	7.972	7.397	1.879	1.667
7.0	2.75	0.135	0.88	0.1875	13.265	13.265	3.790	3.790
		0.105	0.88	0.1875	10.553	10.548	3.015	3.013
		0.075	0.70	0.09375	7.506	7.033	2.113	1.904
		0.060	0.60	0.09375	5.648	5.368	1.538	1.422
6.0	2.50	0.135	0.82	0.1875	8.574	8.574	2.858	2.858
		0.105	0.82	0.1875	6.843	6.843	2.281	2.281
		0.075	0.82	0.09375	5.124	4.944	1.708	1.607
		0.060	0.60	0.09375	3.766	3.606	1.205	1.125
5.0	2.00	0.135	0.70	0.1875	4.683	4.683	1.873	1.873
		0.105	0.70	0.1875	3.760	3.760	1.504	1.504
		0.075	0.60	0.09375	2.797	2.760	1.119	1.093
		0.060	0.50	0.09375	2.190	2.053	0.865	0.780
		0.048	0.50	0.09375	1.710	1.632	0.660	0.614
4.0	2.00	0.135	0.70	0.1875	2.751	2.751	1.375	1.375
		0.105	0.70	0.1875	2.218	2.218	1.109	1.109
		0.075	0.60	0.09375	1.664	1.644	0.832	0.813
		0.060	0.50	0.09375	1.309	1.223	0.646	0.578
		0.048	0.50	0.09375	1.022	0.972	0.492	0.454
3.5	2.00	0.135	0.70	0.1875	2.002	2.002	1.144	1.144
		0.105	0.70	0.1875	1.619	1.619	0.925	0.925
		0.075	0.60	0.09375	1.222	1.208	0.698	0.683
		0.060	0.50	0.09375	0.964	0.899	0.544	0.484
		0.048	0.50	0.09375	0.753	0.715	0.413	0.380
3.0	1.75	0.105	0.70	0.1875	1.016	1.016	0.678	0.678
		0.075	0.53	0.9375	0.767	0.767	0.512	0.512
		0.060	0.53	0.9375	0.628	0.617	0.418	0.405
		0.048	0.41	0.9375	0.485	0.442	0.317	0.272

TABLE 13 TABLE 13

Z-SECTION WITH UNSTIFFENED FLANGES

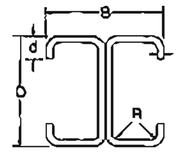


See notes on page V-25

Size		t	d	R	Effective Section Properties			
D	B				I_x, In.^{4}		S_x, In.^{3}	
In.	In.	In.	In.	In.	F_y = 33ksi	F_y = 50ksi	F_y = 33ksi	F_y = 50ksi
8.0	2.00	0.135	0	0.1875	13.075	12.690	3.269	3.123
		0.105	0	0.1875	10.335	9.580	2.584	2.305
		0.075	0	0.09375	6.764	6.519	1.596	1.513
		0.060	0	0.09375	5.231	4.819	1.211	1.066
7.0	1.50	0.135	0	0.1875	7.840	7.840	2.240	2.240
		0.105	0	0.1875	6.218	6.119	1.776	1.734
		0.075	0	0.09375	4.366	4.219	1.215	1.155
		0.060	0	0.09375	3.391	3.278	0.926	0.882
6.0	1.50	0.135	0	0.1875	5.334	5.334	1.778	0.778
		0.105	0	0.1875	4.240	4.168	1.413	1.377
		0.075	0	0.09375	2.976	2.868	0.963	0.912
		0.060	0	0.09375	2.306	2.223	0.731	0.693
		0.048	0	0.09375	1.784	1.716	0.555	0.524
5.0	1.25	0.105	0	0.1875	2.399	2.399	0.959	0.959
		0.075	0	0.09375	1.760	1.700	0.696	0.661
		0.060	0	0.09375	1.370	1.320	0.531	0.503
		0.048	0	0.09375	1.062	1.024	0.403	0.382
4.0	1.50	0.060	0	0.09375	0.858	0.820	0.402	0.376
4.0	1.125	0.105	0	0.1875	1.286	1.286	0.643	0.643
		0.075	0	0.09375	0.971	0.938	0.485	0.460
		0.060	0	0.09375	0.758	0.730	0.371	0.350
		0.048	0	0.09375	0.588	0.565	0.281	0.265
3.0	1.50	0.060	0	0.09375	0.434	0.412	0.268	0.248
3.0	1.125	0.105	0	0.1875	0.636	0.636	0.424	0.424
		0.075	0	0.09375	0.485	0.467	0.323	0.304
		0.060	0	0.09375	0.379	0.363	0.246	0.230
		0.048	0	0.09375	0.293	0.280	0.185	0.173
2.0	1.125	0.105	0	0.1875	0.240	0.240	0.240	0.240
		0.075	0	0.09375	0.187	0.179	0.187	0.174
		0.060	0	0.09375	0.146	0.139	0.141	0.130
		0.048	0	0.09375	0.113	0.107	0.105	0.097
1.5	1.50	0.048	0	0.09375	0.067	0.062	0.077	0.070
		0.036	0	0.09375	0.047	0.044	0.051	0.047

TABLE 14 TABLE 14

TWO CHANNELS WITH STIFFENED FLANGES BACK TO BACK

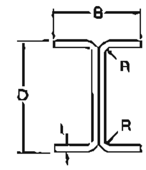


See notes on page V-25

Size		t	d	R	Effective Section Properties			
D	B				I_x, In.[4]		S_x, In.[3]	
In.	In.	In.	In.	In.	$F_y = 33$ksi	$F_y = 50$ksi	$F_y = 33$ksi	$F_y = 50$ksi
12.0	7.0	0.135	1.01	0.1875	112.53	110.77	18.755	18.269
		0.105	0.90	0.1875	86.463	81.808	14.279	13.092
10.0	7.0	0.135	1.01	0.1875	73.051	71.912	14.610	14.221
		0.105	0.90	0.1875	56.221	53.026	11.133	10.142
		0.075	0.72	0.09375	37.727	35.141	7.171	6.401
9.0	6.5	0.135	1.00	0.1875	54.313	54.313	12.070	12.070
		0.105	0.84	0.1875	41.834	40.058	9.245	8.614
		0.075	0.70	0.09375	28.469	27.102	6.054	5.613
		0.060	0.61	0.09375	21.720	19.405	4.514	3.749
8.0	6.0	0.135	0.93	0.1875	38.741	38.741	9.685	9.685
		0.105	0.81	0.1875	30.249	29.453	7.562	7.225
		0.075	0.70	0.09375	20.871	19.931	5.031	4.682
		0.060	0.60	0.09375	15.944	14.794	3.759	3.749
7.0	5.5	0.135	0.88	0.1875	26.530	26.530	7.580	7.580
		0.105	0.88	0.1875	21.105	21.096	6.030	6.025
		0.075	0.70	0.09375	15.012	14.066	4.226	3.809
		0.060	0.60	0.09375	11.295	10.735	3.075	2.844
6.0	5.0	0.135	0.82	0.1875	17.149	17.149	5.716	5.716
		0.105	0.82	0.1875	13.685	13.685	4.562	4.562
		0.075	0.82	0.09375	10.248	9.888	3.416	3.213
		0.060	0.60	0.09375	7.532	7.212	2.409	2.251
5.0	4.0	0.135	0.70	0.1875	9.365	9.365	3.746	3.746
		0.105	0.70	0.1875	7.520	7.520	3.008	3.008
		0.075	0.60	0.09375	5.594	5.520	2.237	2.185
		0.060	0.50	0.09375	4.380	4.107	1.731	1.561
		0.048	0.50	0.09375	3.420	3.264	1.321	1.227
4.0	4.0	0.135	0.70	0.1875	5.501	5.501	2.750	2.750
		0.105	0.70	0.1875	4.436	4.436	2.218	2.218
		0.075	0.60	0.09375	3.328	3.287	1.664	1.626
		0.060	0.50	0.09375	2.617	2.446	1.291	1.156
		0.048	0.50	0.09375	2.043	1.944	0.983	0.908
3.5	4.0	0.135	0.70	0.1875	4.004	4.004	2.288	2.288
		0.105	0.70	0.1875	3.239	3.239	1.851	1.851
		0.075	0.60	0.09375	2.444	2.416	1.397	1.366
		0.060	0.50	0.09375	1.928	1.798	1.087	0.969
		0.048	0.50	0.09375	1.506	1.430	0.827	0.761
3.0	3.5	0.105	0.70	0.1875	2.033	3.033	1.355	1.355
		0.075	0.53	0.9375	1.535	1.535	1.023	1.023
		0.060	0.53	0.9375	1.255	1.234	0.837	0.810
		0.048	0.41	0.9375	0.971	0.884	0.633	0.544

TABLE 15 TABLE 15

TWO CHANNELS WITH UNSTIFFENED FLANGES BACK TO BACK



See notes on page V-25

Size D	Size B	t	d	R	Effective Section Properties I_x, In.^{4}		S_x, In.^{3}	
In.	In.	In.	In.	In.	F_y=33ksi	F_y=50ksi	F_y=33ksi	F_y=50ksi
8.0	4.0	0.135	0	0.1875	26.150	25.380	6.537	6.247
		0.105	0	0.1875	20.669	19.161	5.167	4.609
		0.075	0	0.09375	13.529	13.038	3.191	3.025
		0.060	0	0.09375	10.462	9.639	2.422	2.132
7.0	3.0	0.135	0	0.1875	15.679	15.679	4.480	4.480
		0.105	0	0.1875	12.435	12.239	3.553	3.469
		0.075	0	0.09375	8.733	8.439	2.430	0.311
		0.060	0	0.09375	6.782	6.556	1.862	0.764
6.0	3.0	0.135	0	0.1875	10.668	10.668	3.556	3.556
		0.105	0	0.1875	8.480	8.336	2.827	2.754
		0.075	0	0.09375	5.952	5.736	1.927	1.823
		0.060	0	0.09375	4.612	4.445	1.462	1.386
		0.048	0	0.09375	3.569	3.431	1.110	1.048
5.0	2.5	0.105	0	0.1875	4.797	4.797	1.919	1.919
		0.075	0	0.09375	3.520	3.400	1.392	1.321
		0.060	0	0.09375	2.740	2.641	1.062	1.005
		0.048	0	0.09375	2.123	2.047	0.806	0.765
4.0	2.25	0.105	0	0.1875	2.572	2.572	1.285	1.285
		0.075	0	0.09375	1.941	1.877	0.969	0.921
		0.060	0	0.09375	1.516	1.460	0.741	0.700
		0.048	0	0.09375	1.176	1.131	0.562	0.531
3.0	2.25	0.105	0	0.1875	1.272	1.272	0.848	0.848
		0.075	0	0.09375	0.971	0.935	0.646	0.609
		0.060	0	0.09375	0.757	0.725	0.491	0.460
		0.048	0	0.09375	0.585	0.560	0.370	0.346
2.0	2.25	0.105	0	0.1875	0.481	0.481	0.481	0.481
		0.075	0	0.09375	0.374	0.358	0.373	0.348
		0.060	0	0.09375	0.292	0.278	0.282	0.261
		0.048	0	0.09375	0.225	0.213	0.210	0.194

COMPUTER AIDS

COMPUTER AIDS

FOR USE WITH THE
MARCH 16, 1991 EDITION OF THE

LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

LRFD Cold-Formed Steel Design Manual-Part VI

AMERICAN IRON AND STEEL INSTITUTE
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WASHINGTON, DC 20036-4700

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PREFACE

This document, *Part VI of the LRFD Cold-Formed Steel Design Manual*, contains many useful design aids; flow charts for many sections of the *Specification* are included. These charts will be very helpful in getting all users familiar with the new Specification. The charts serve to direct the user to the appropriate sections. They will also be useful to programmers.

These *Computer Aids* should be used in conjunction with the other parts of the *Design Manual*, which include *Commentary* (Part II), *Supplementary Information* (Part III), *Illustrative Examples* (Part IV), *Design Aids* (Part V), and *Test Procedures* (Part VII).

American Iron and Steel Institute
December 1991

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Computer Aids
Based on the
March 16, 1991 Edition of the
Load and Resistance Factor Design Specification
for Cold-Formed Steel Structural Members

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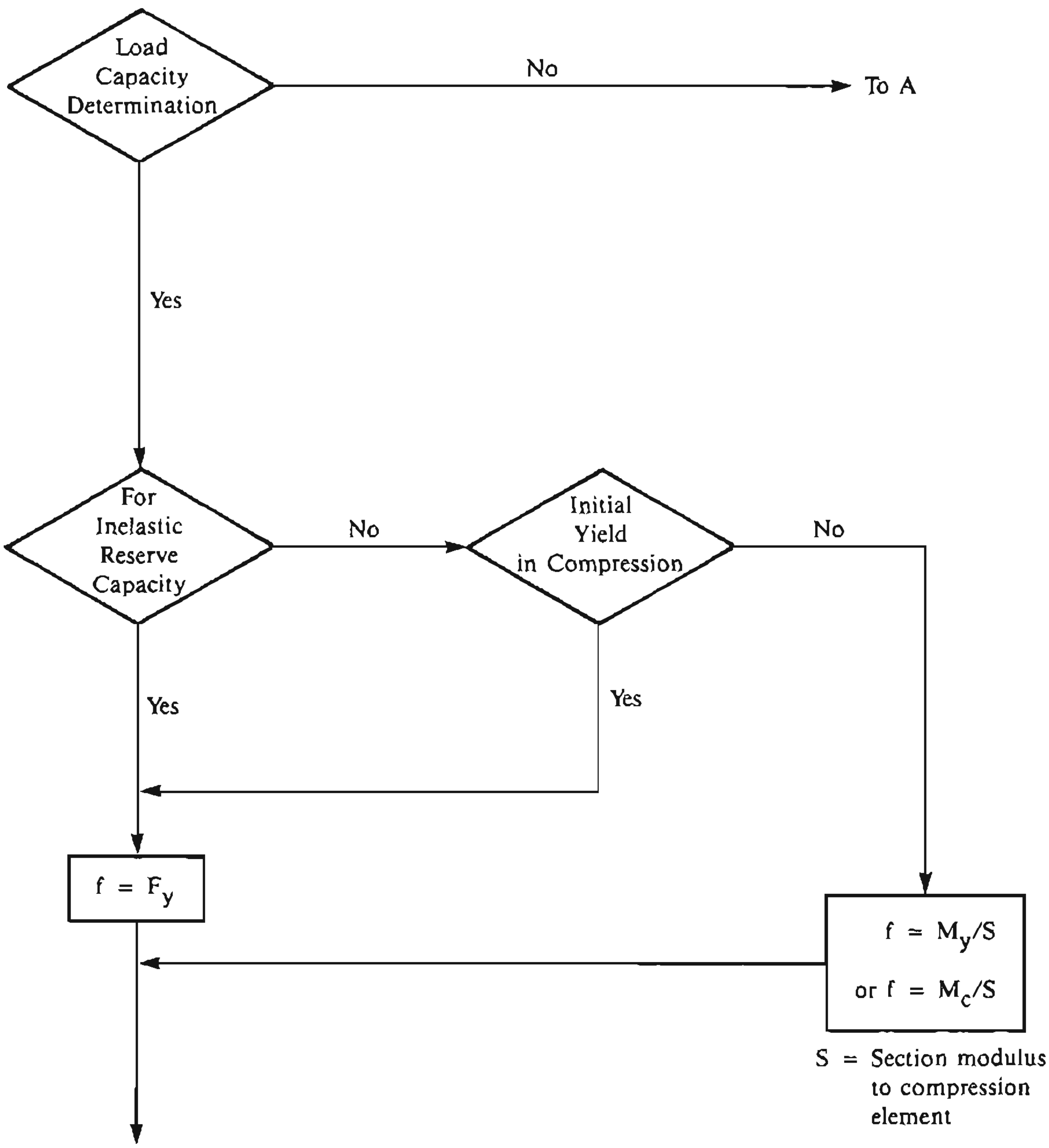
B2.1 Uniformly Compressed Stiffened Elements



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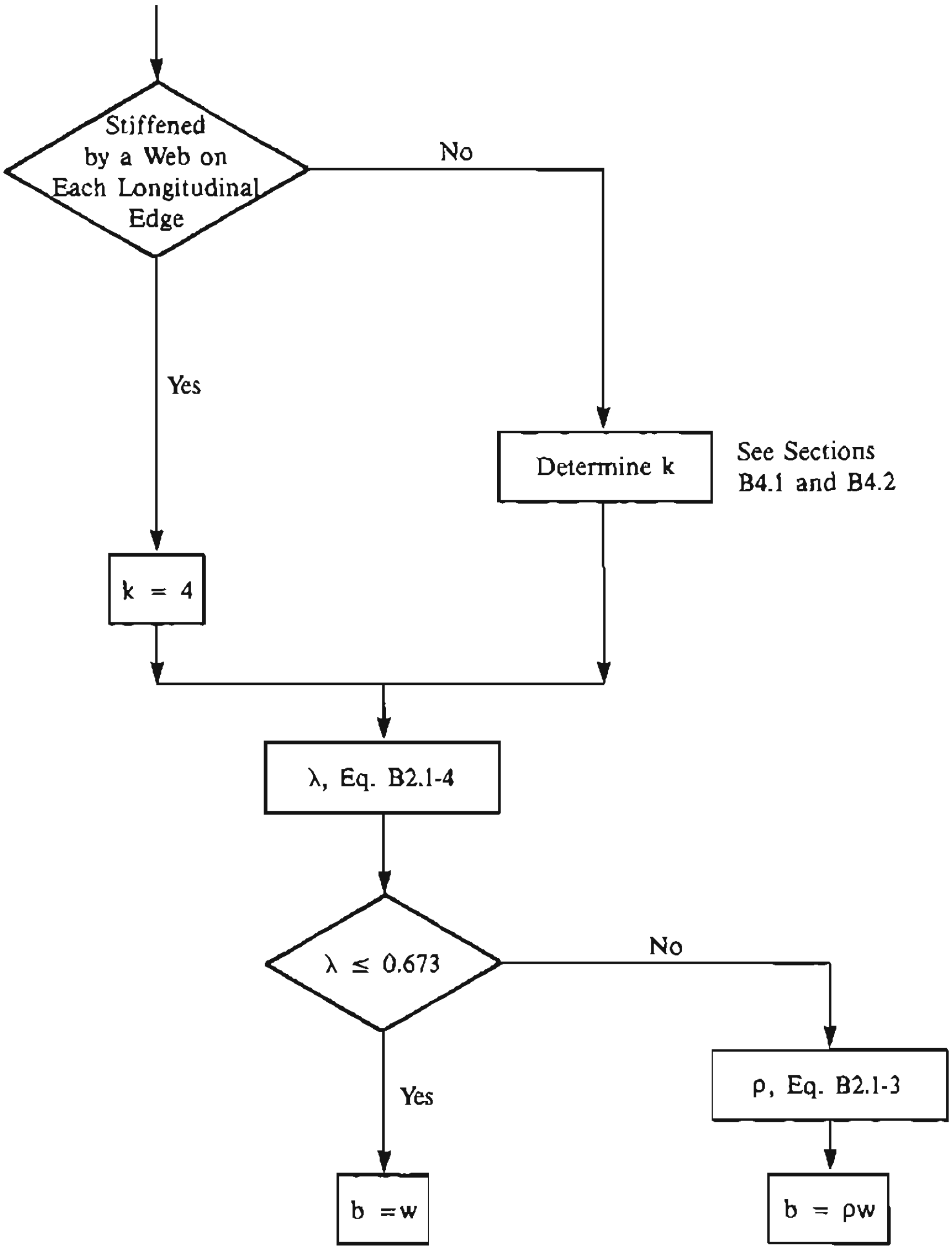
B2.1 Uniformly Compressed Stiffened Elements (continued)



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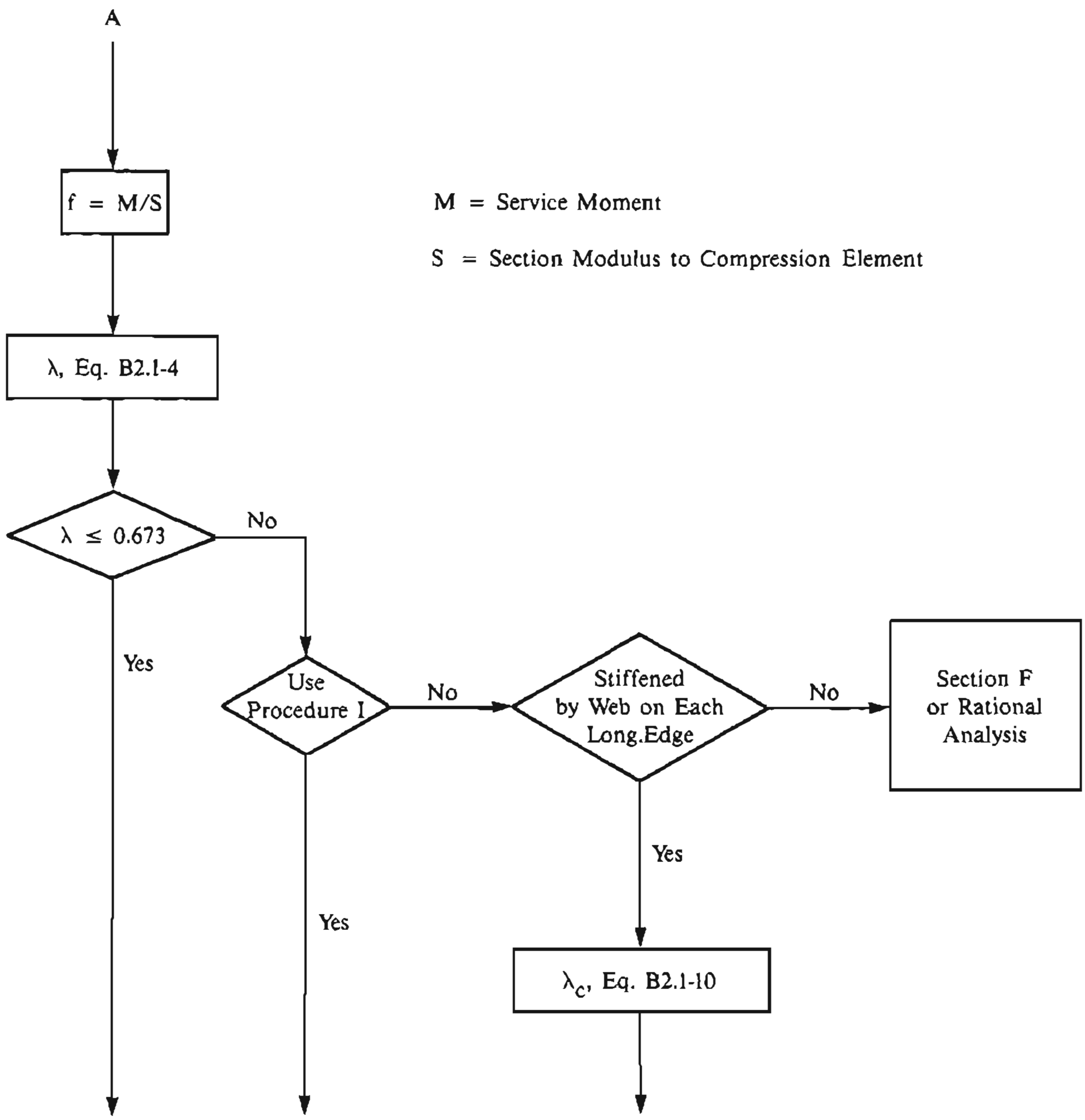
B2.1 Uniformly Compressed Stiffened Elements (continued)



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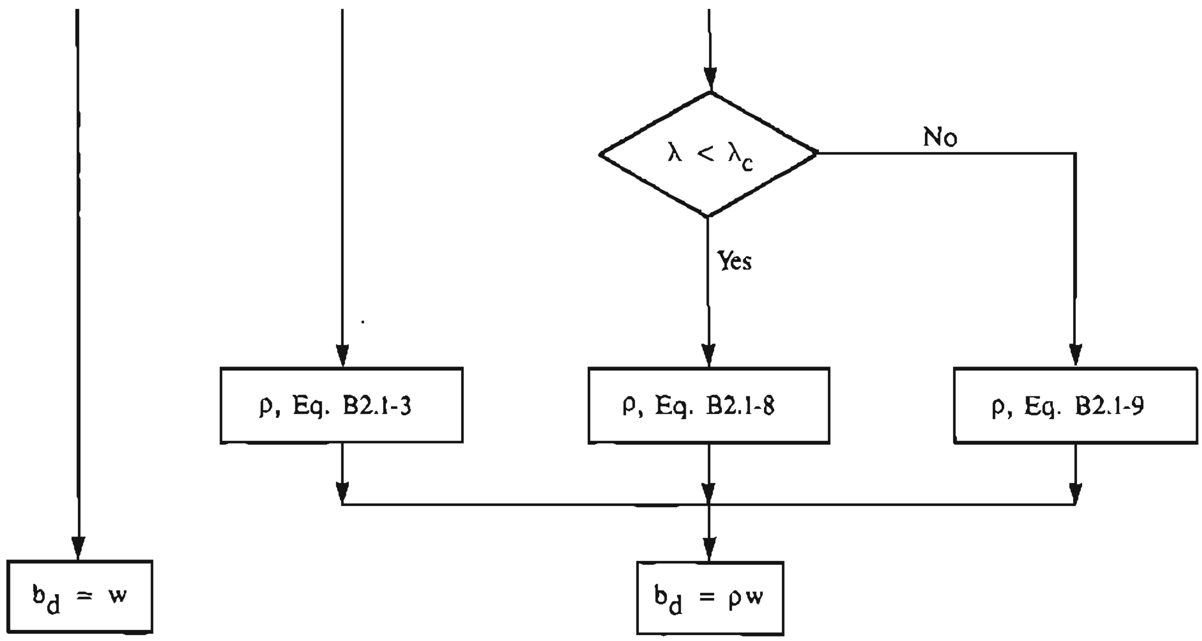
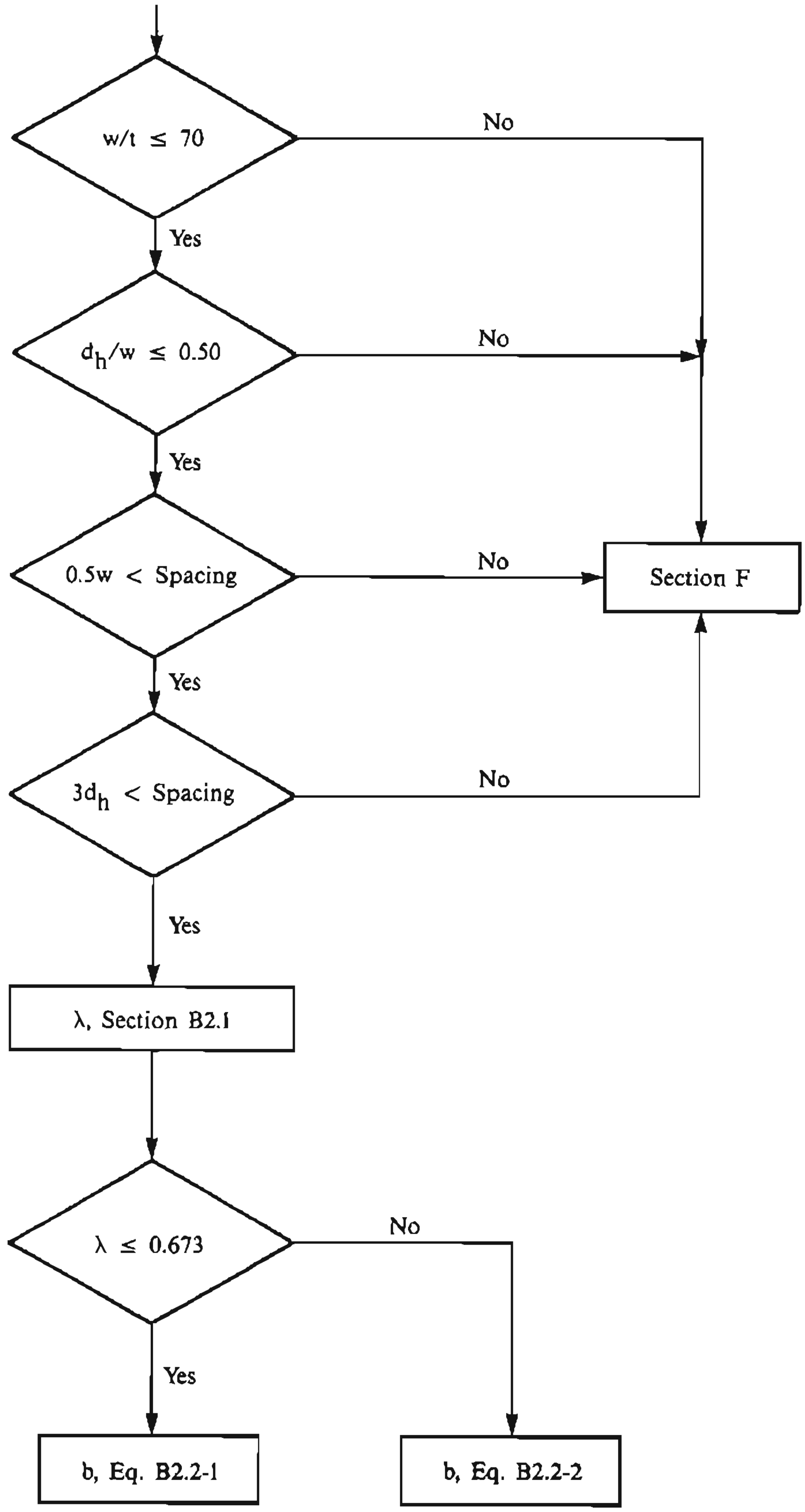
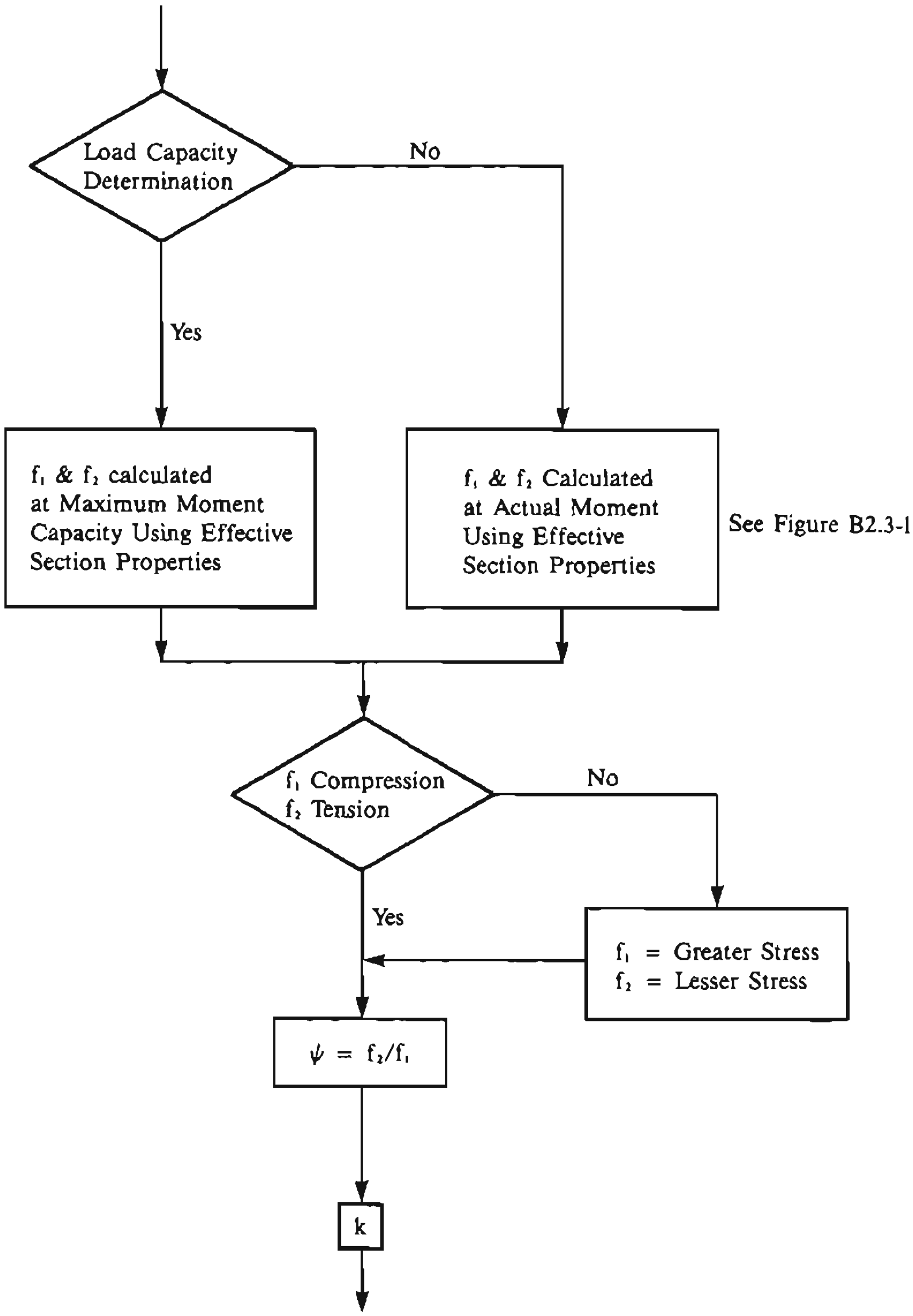
B2.1 Uniformly Compressed Stiffened Elements (continued)



B2.2 Uniformly Compressed Stiffened Elements with Circular Holes



B2.3 Effective Widths of Webs and Stiffened Elements with Stress Gradient



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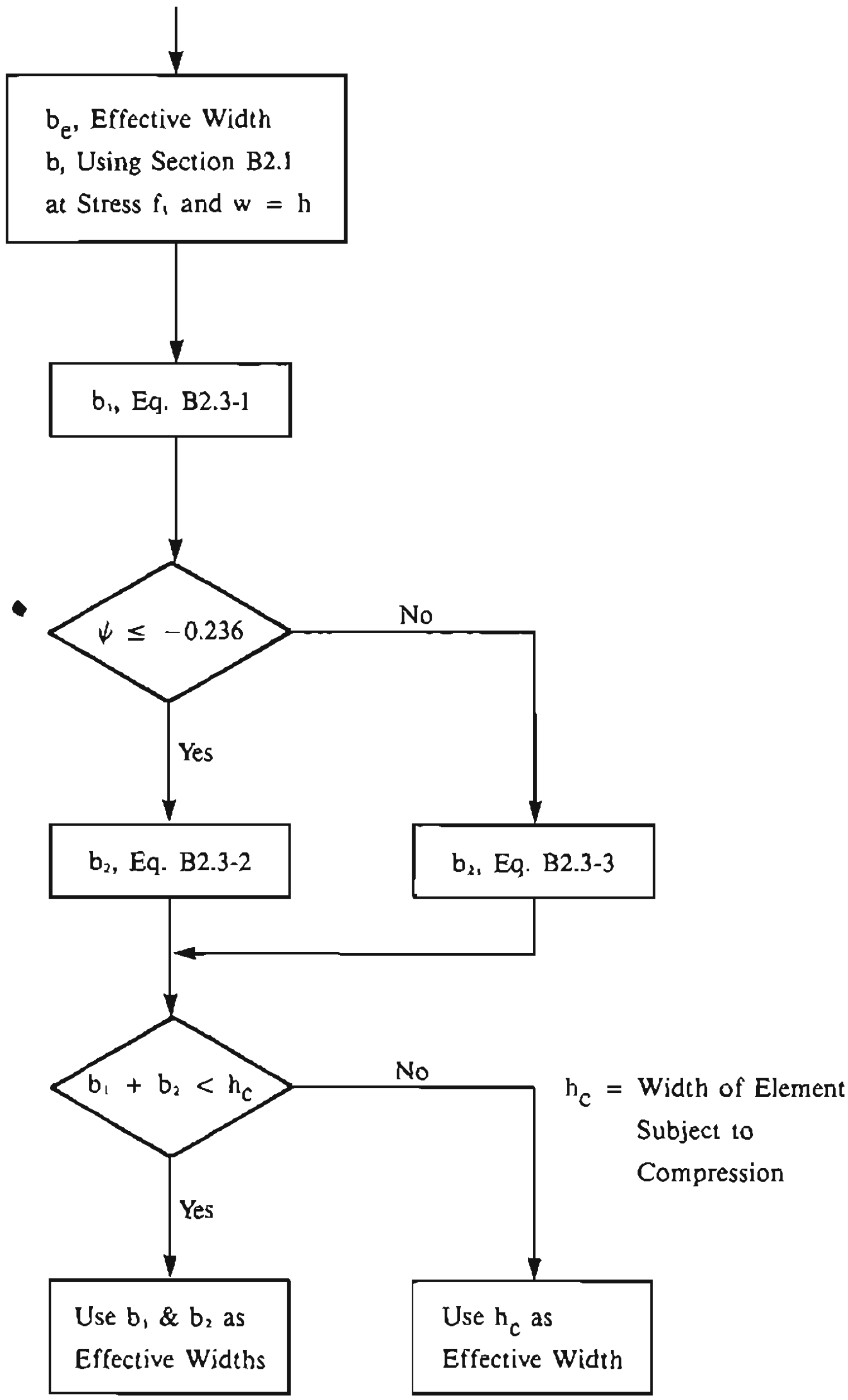
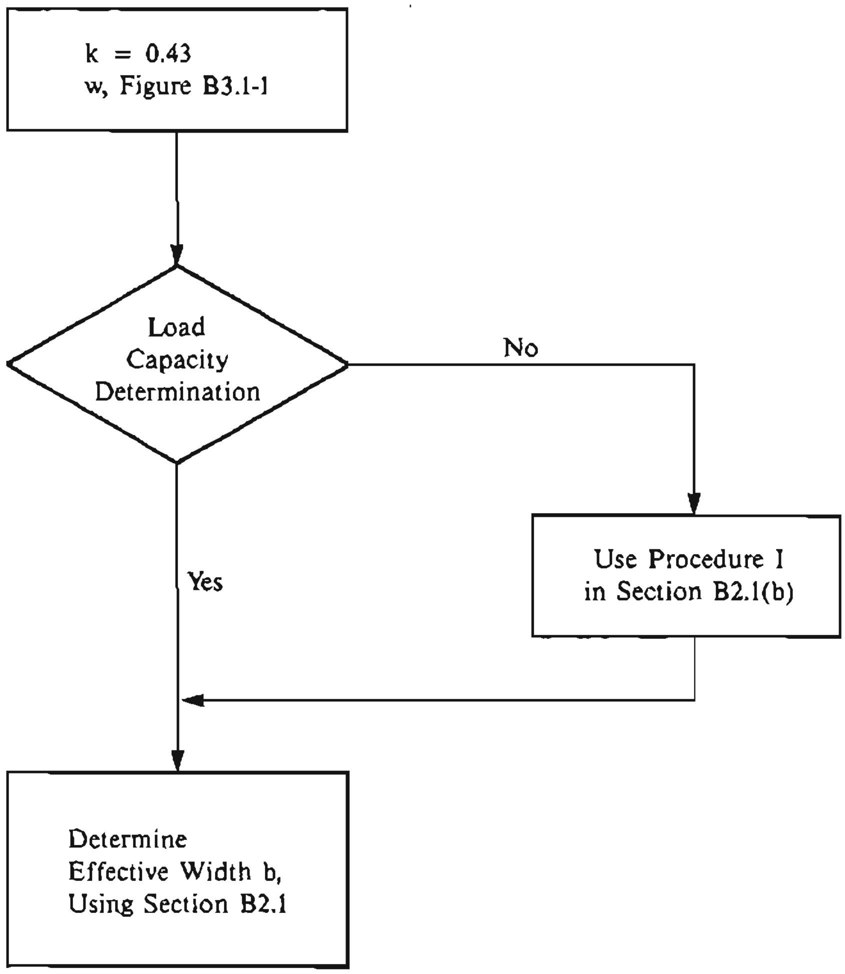
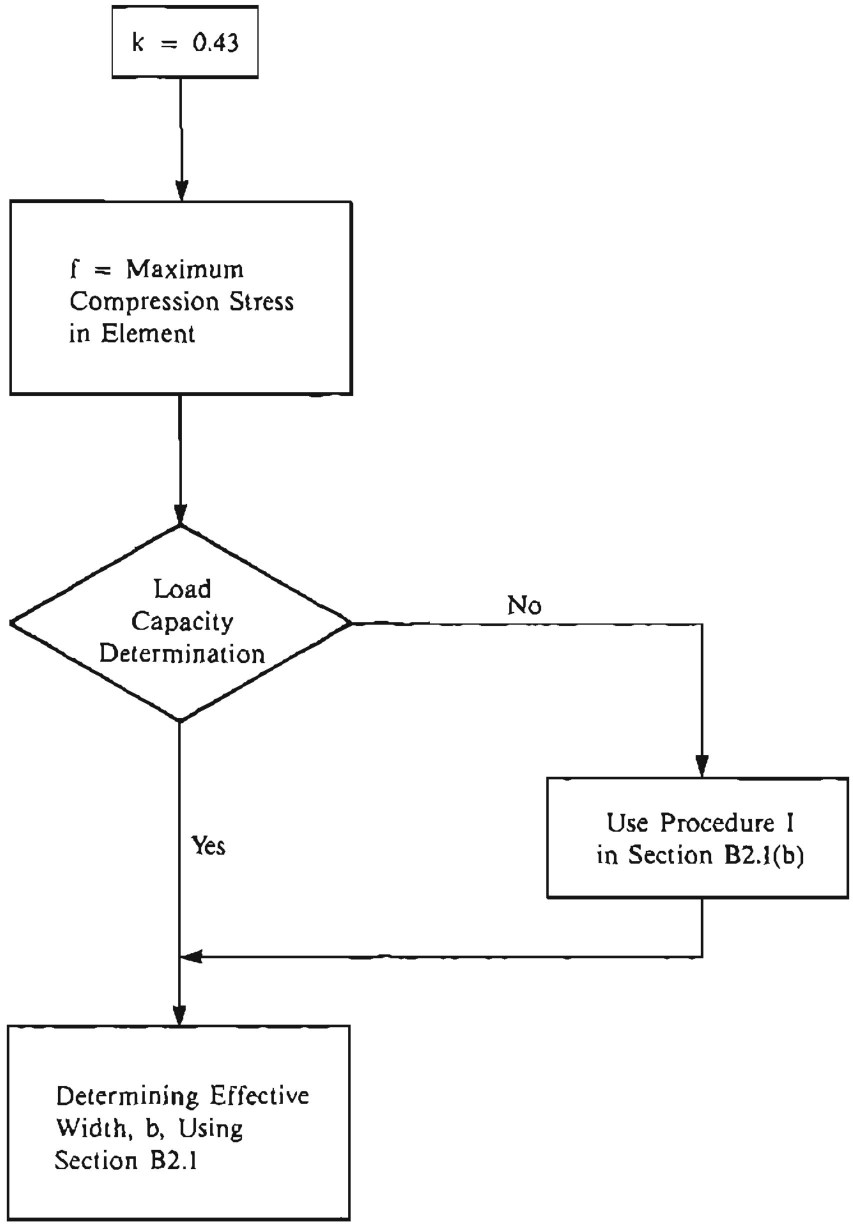
B2.3 Effective Widths of Webs and Stiffened Elements with Stress Gradient (continued)



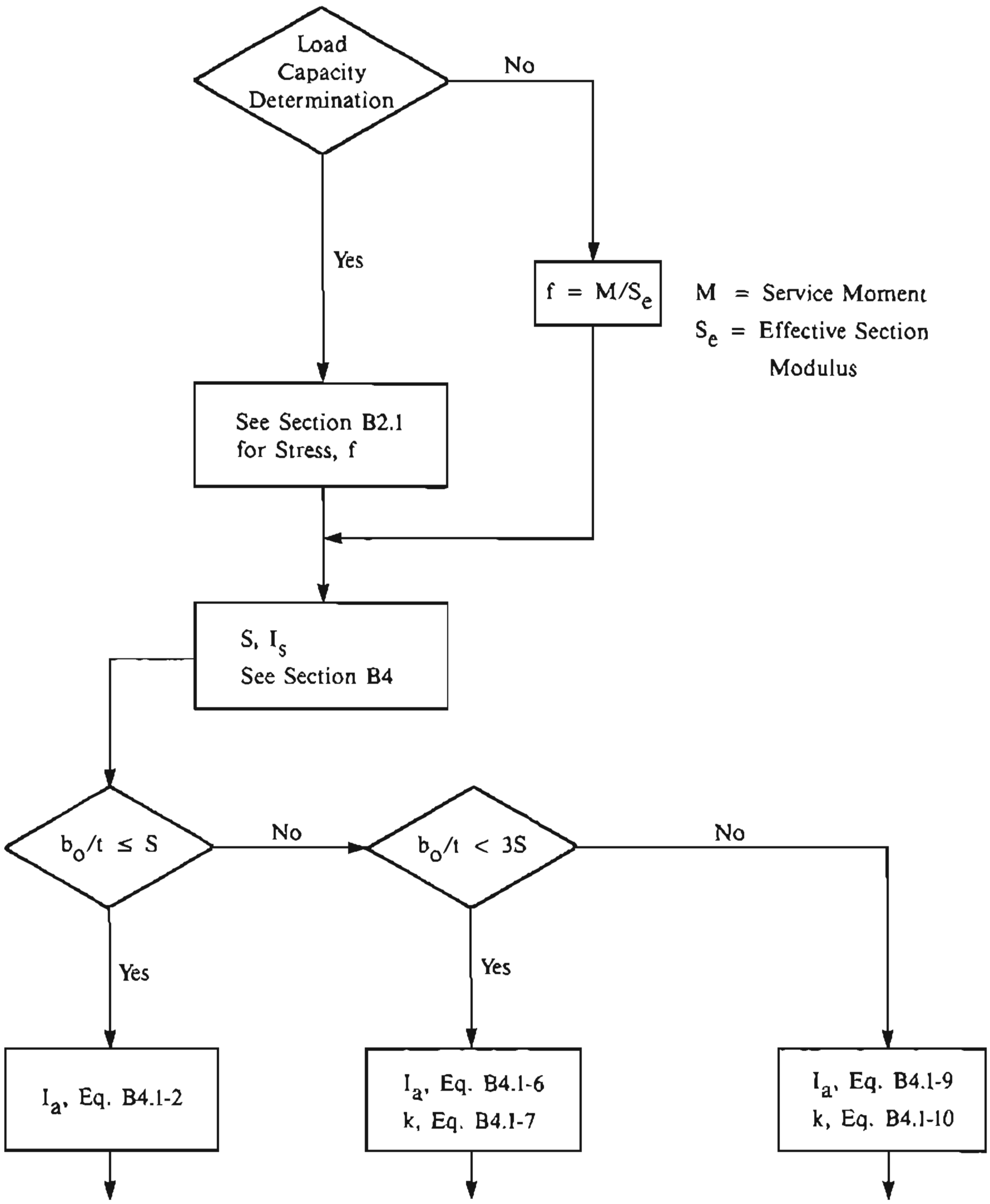
B3.1 Uniformly Compressed Unstiffened Elements



B3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient



B4.1 Uniformly Compressed Elements with an Intermediate Stiffener



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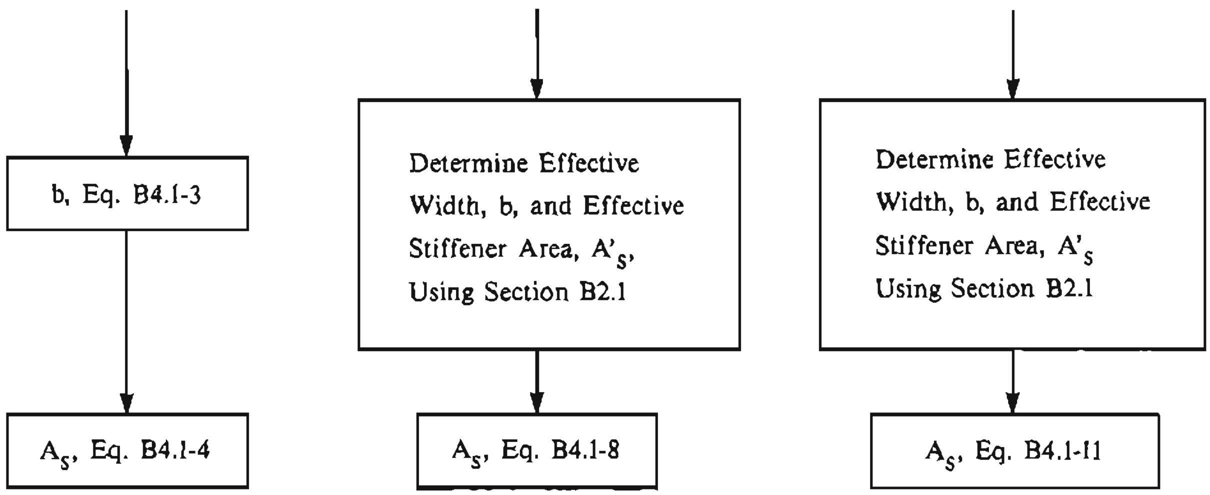
B4.1 Uniformly Compressed Elements with an Intermediate Stiffener (continued)



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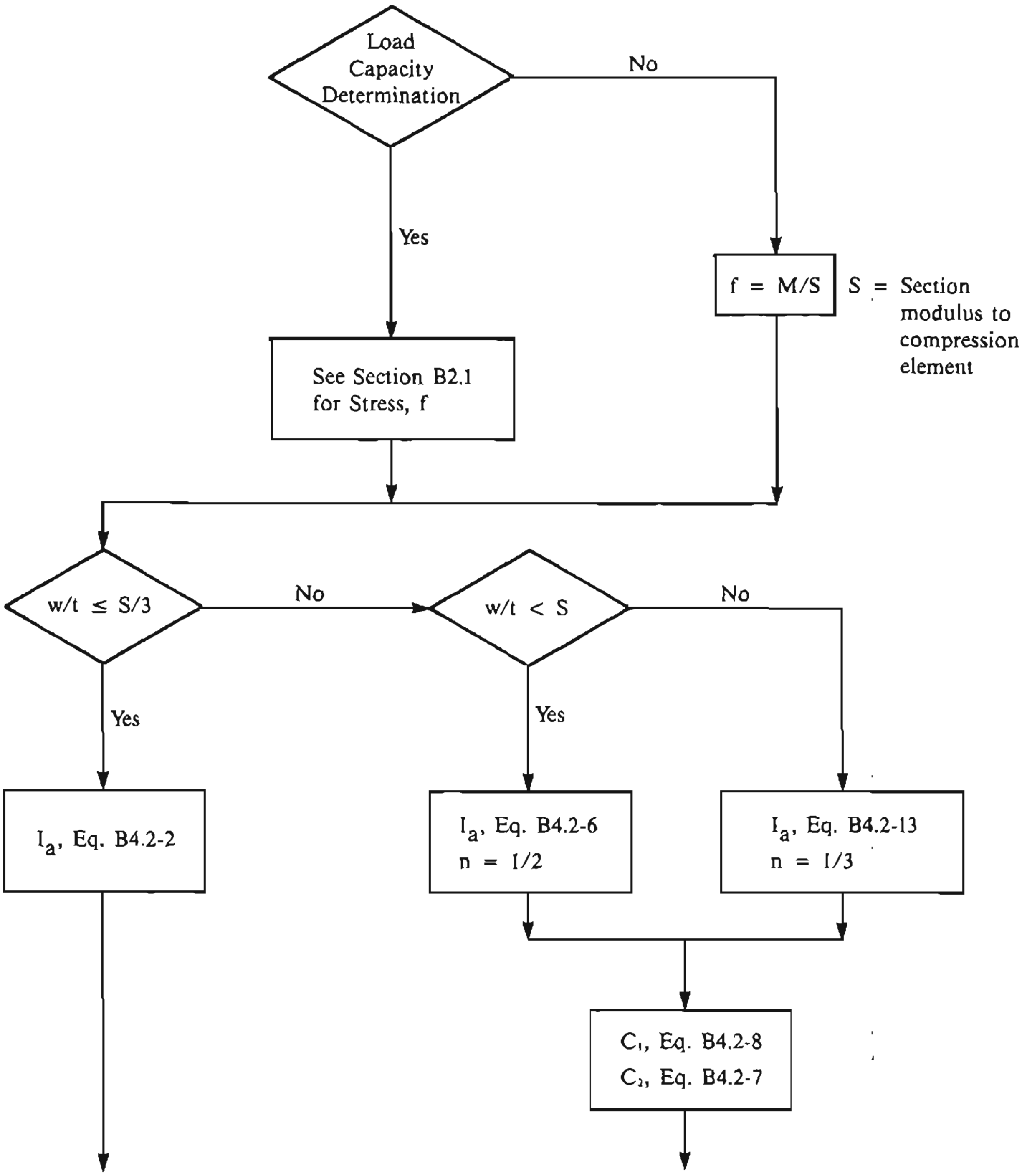

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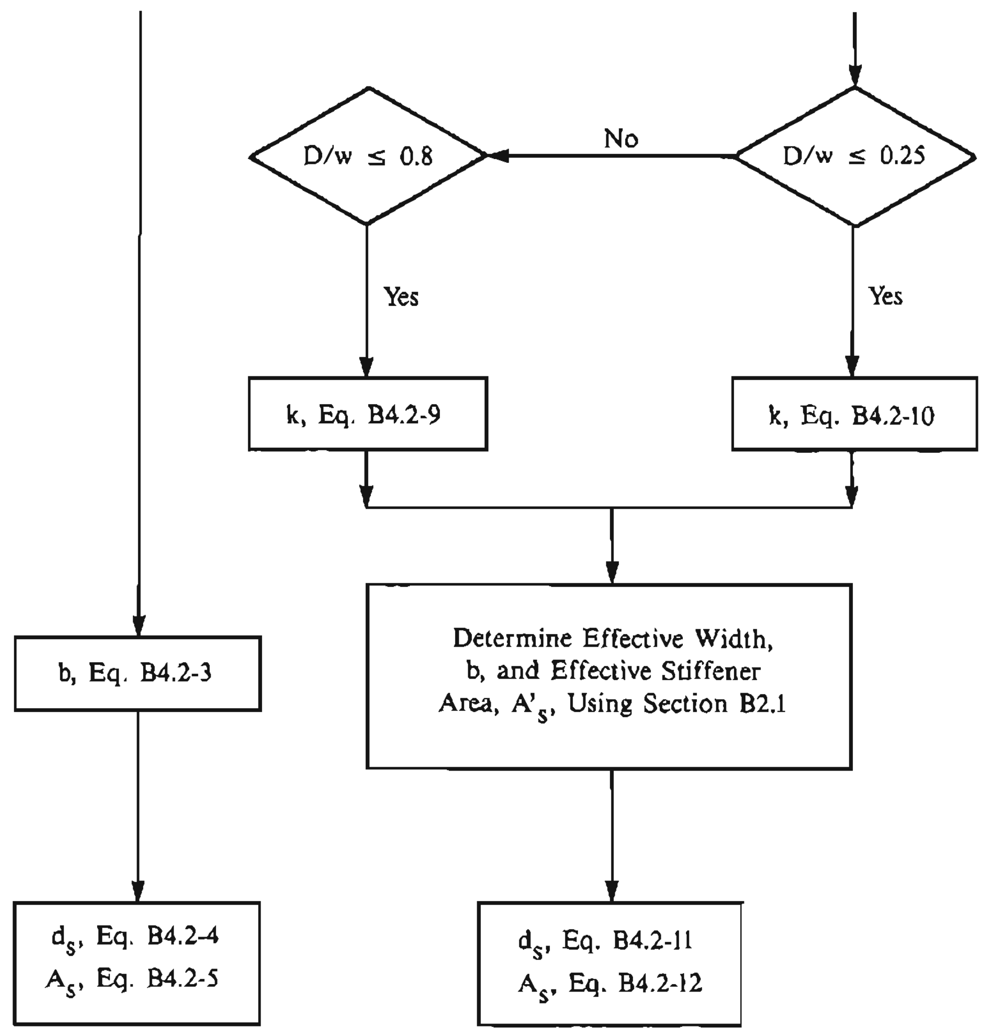
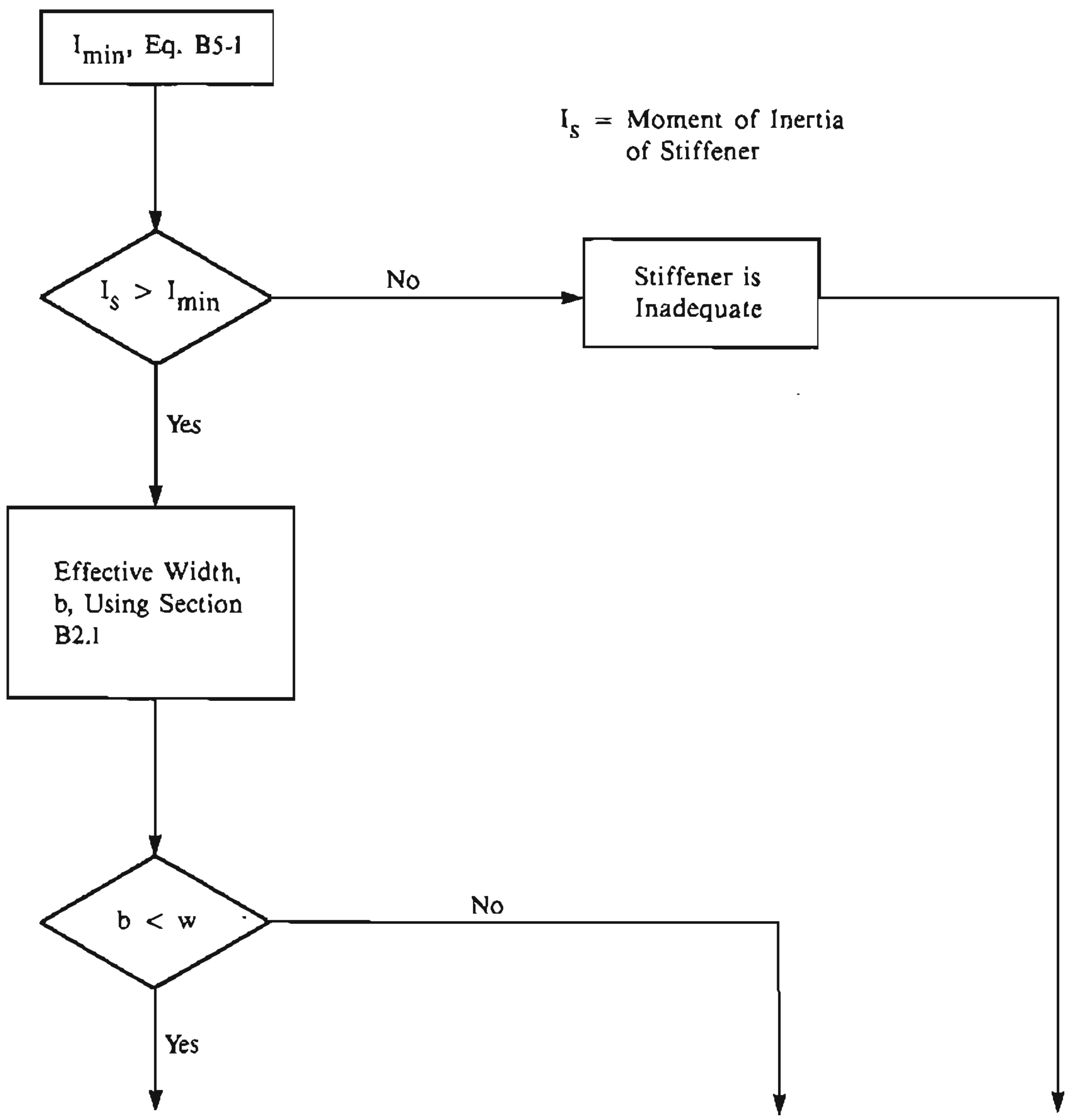
B4.2 Uniformly Compressed Elements with an Edge Stiffener



B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener



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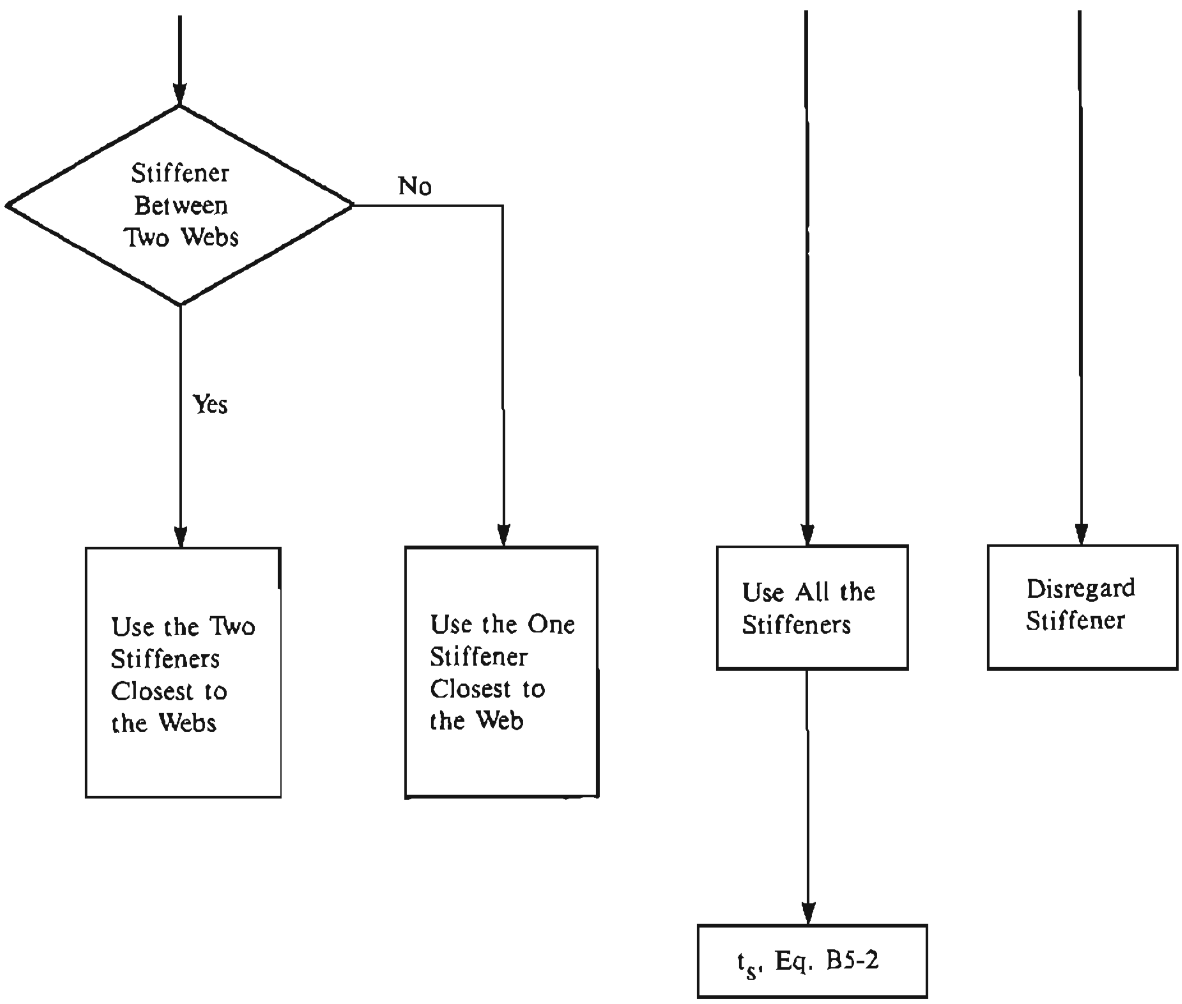
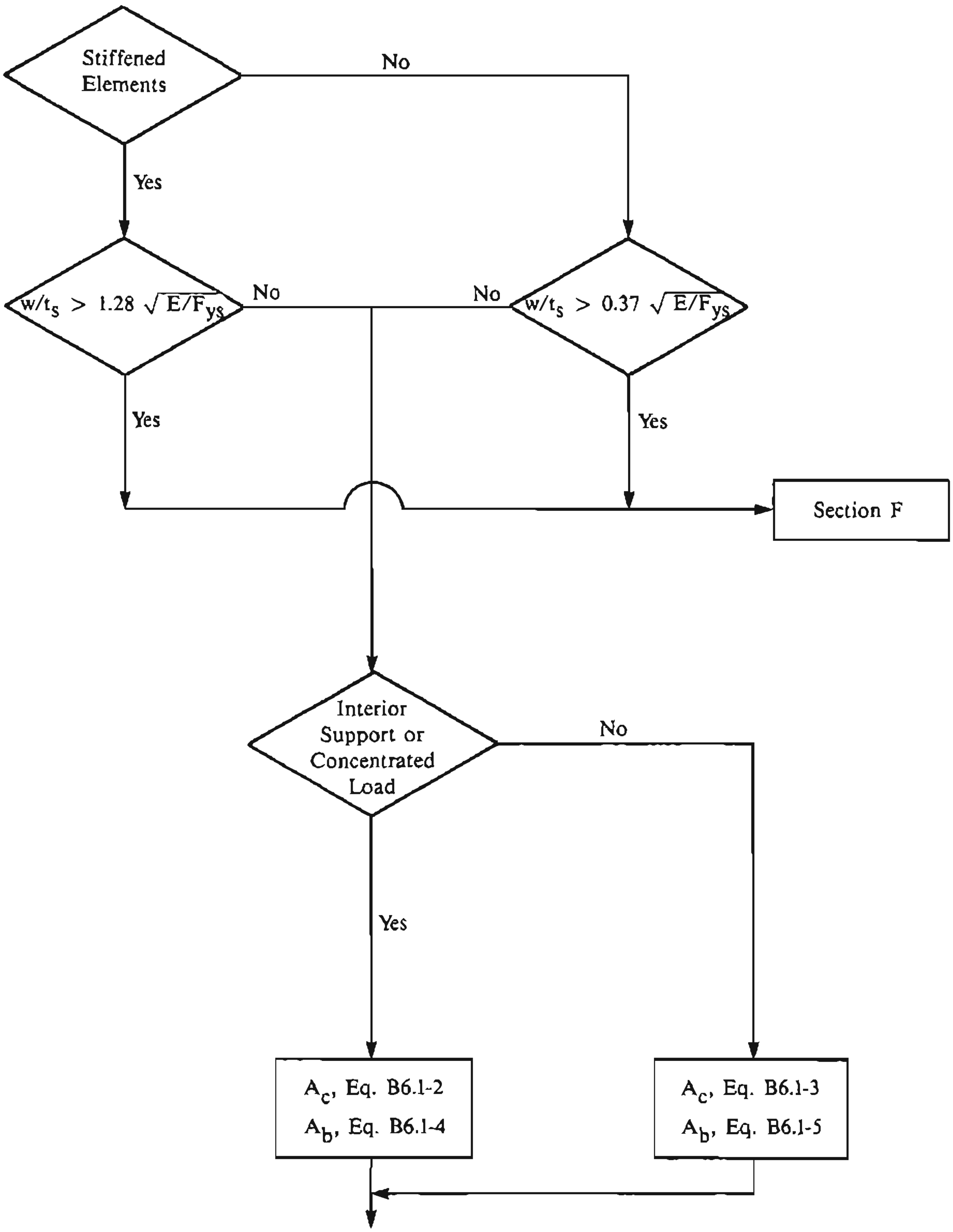
B5 Effective Widths of Edge Stiffened Elements with Intermediate Stiffeners or Stiffened Elements with More Than One Intermediate Stiffener (continued)



B6.1 Transverse Stiffeners



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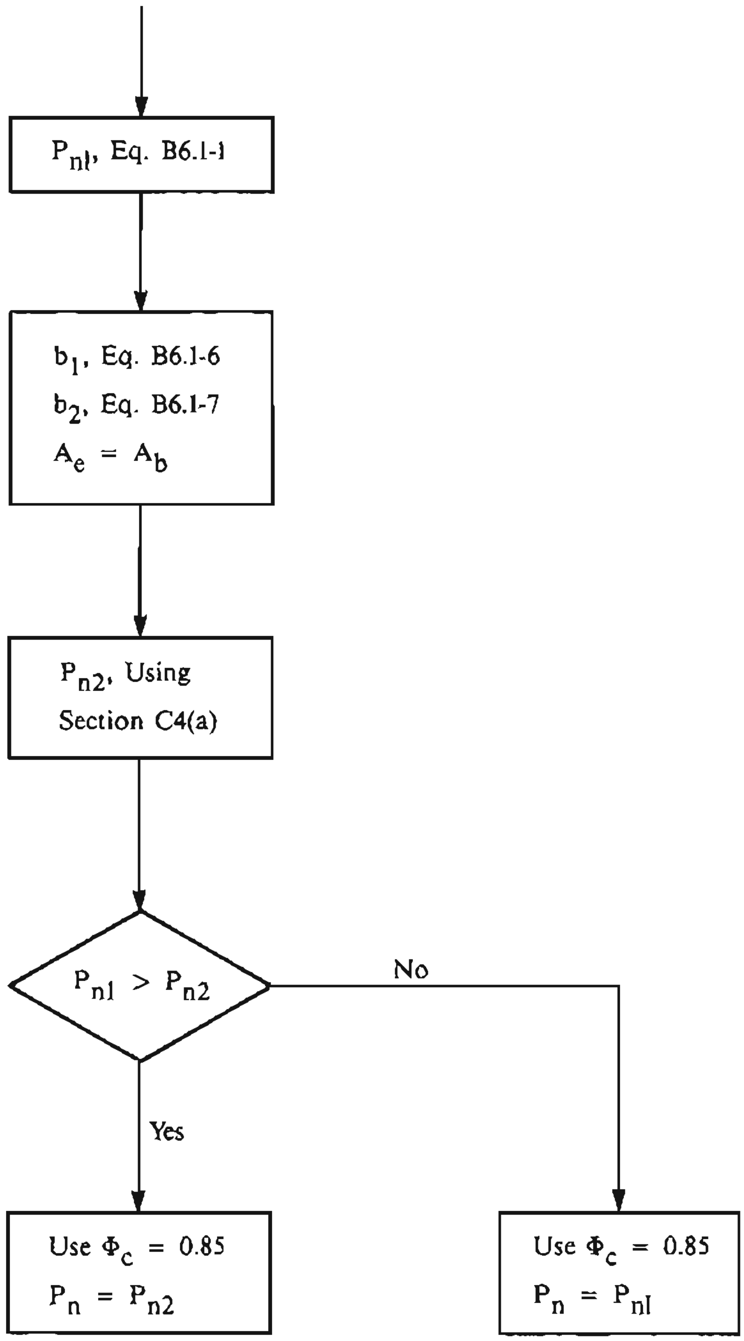

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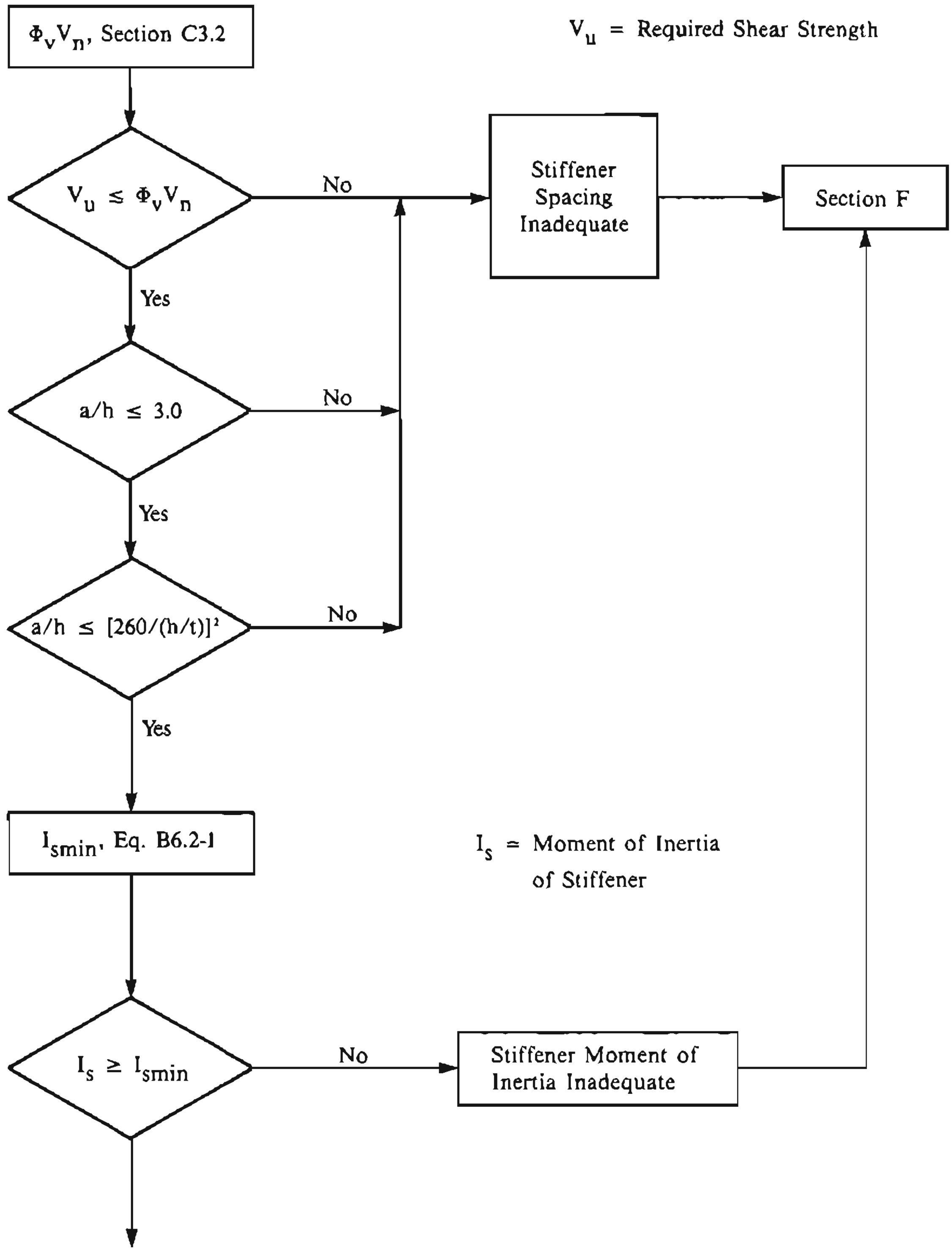

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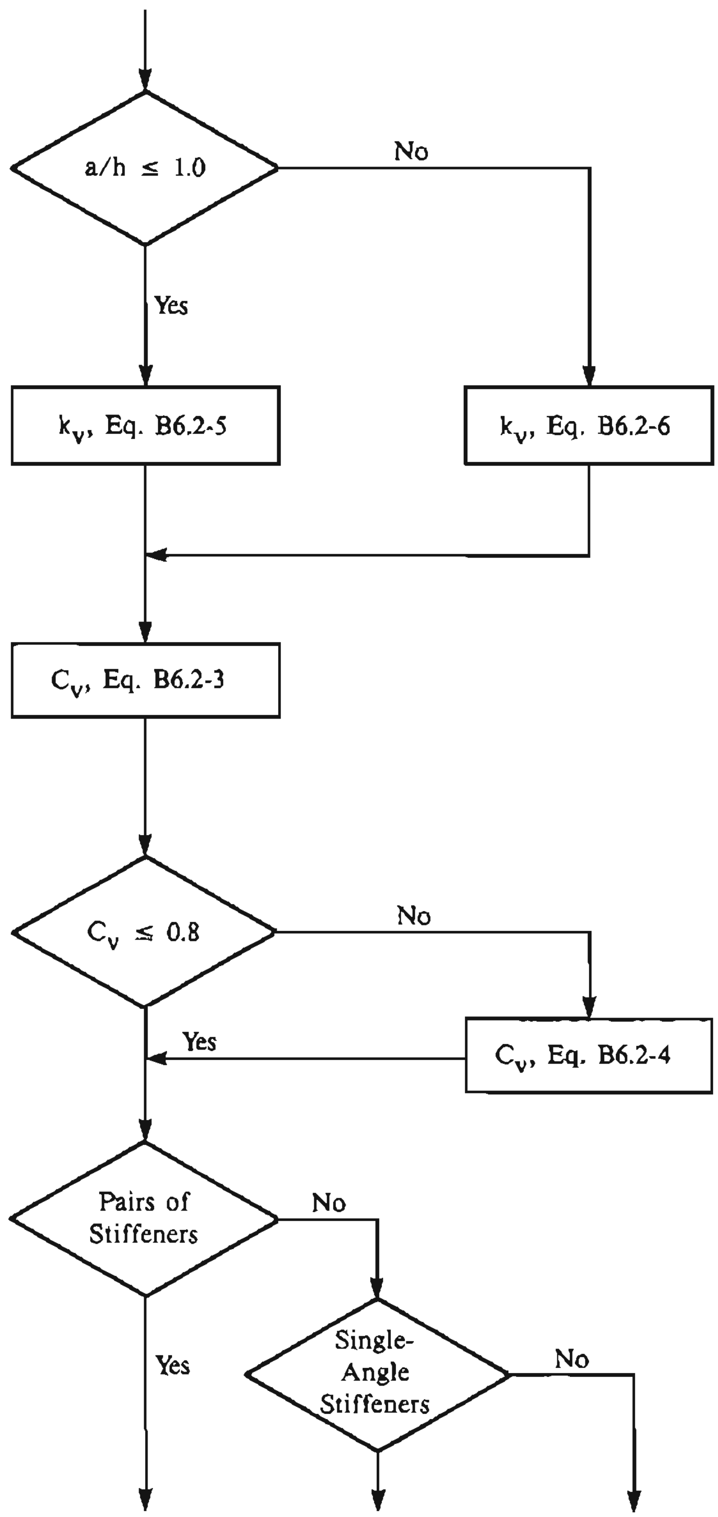

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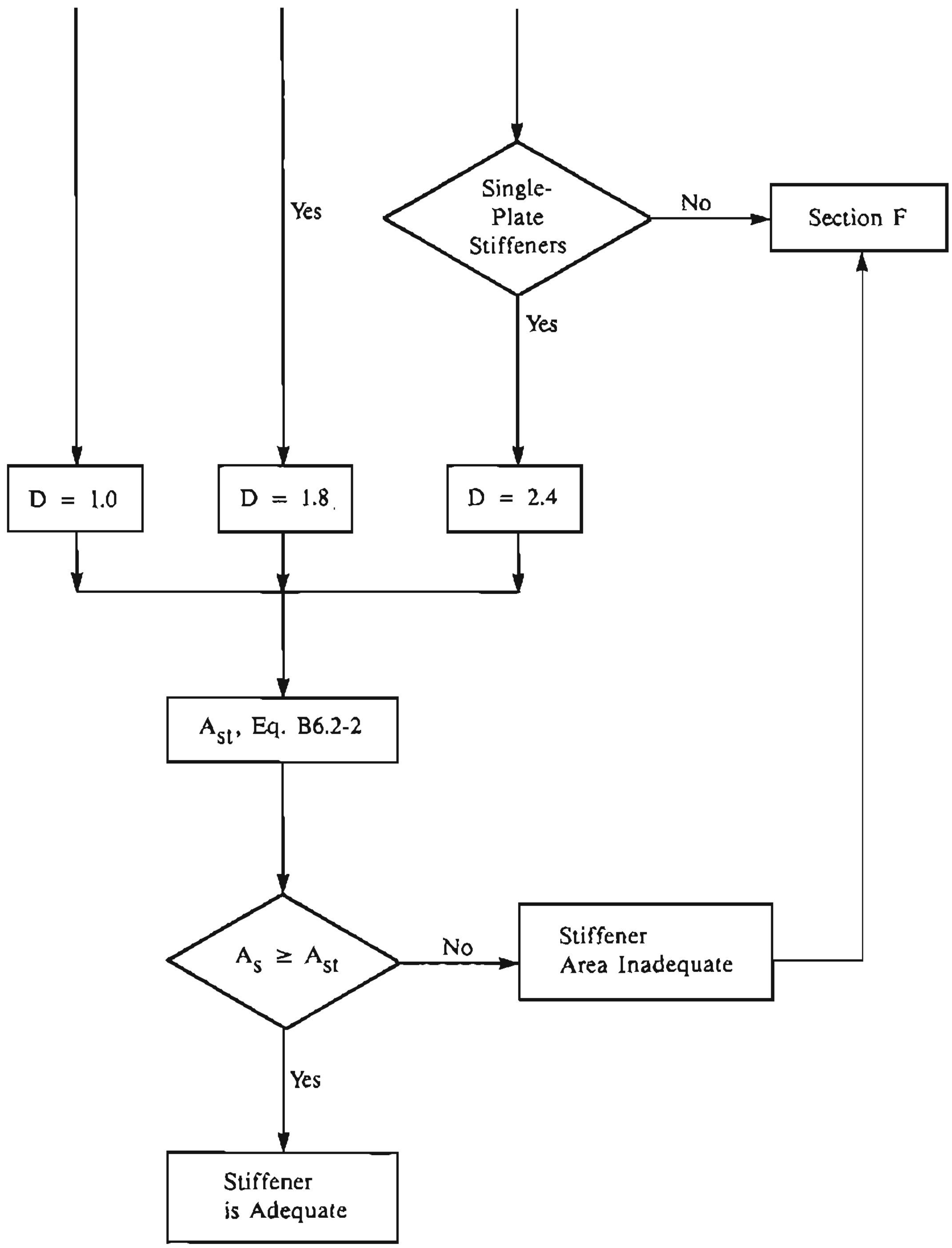
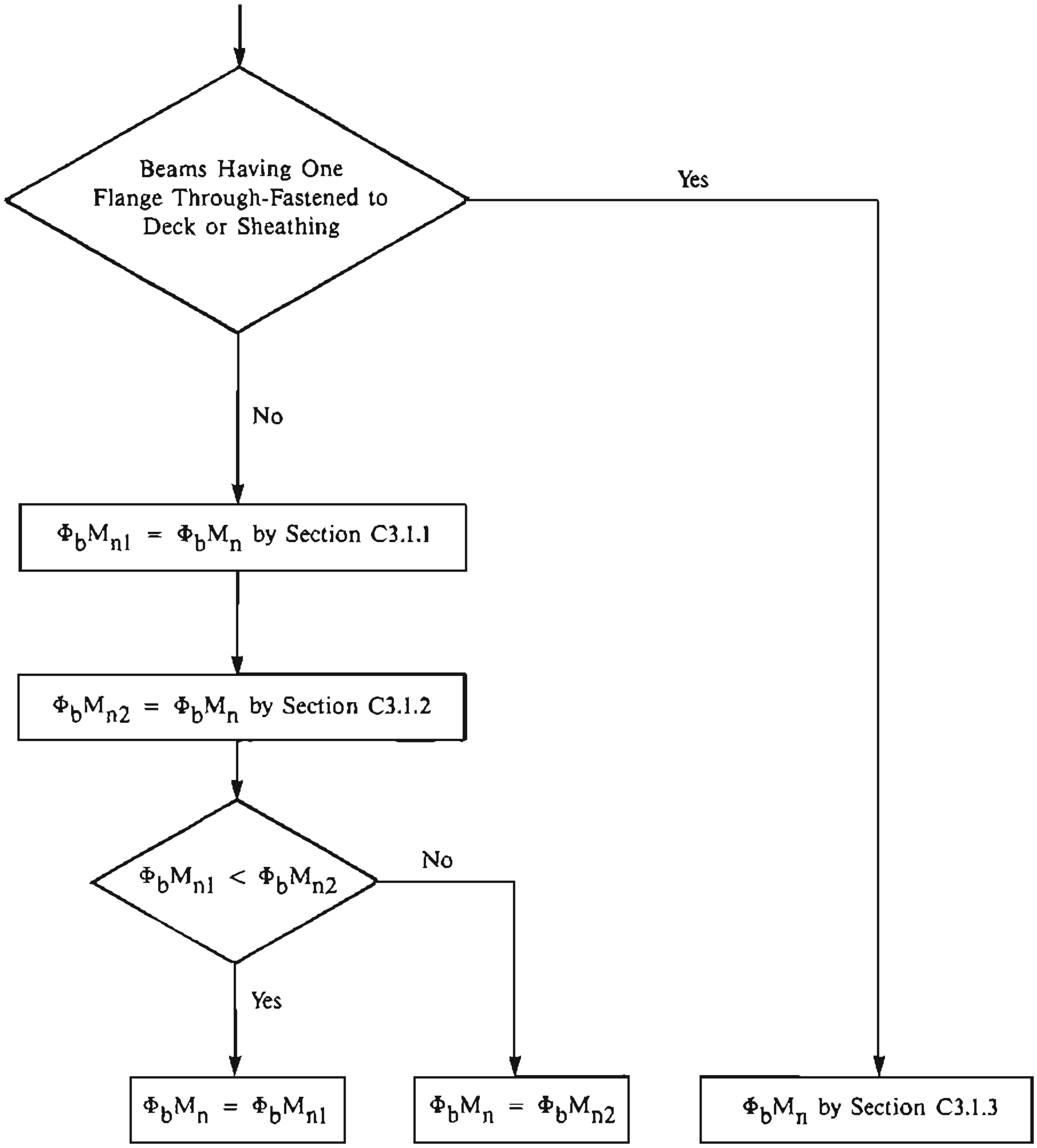


C3.1 Strength for Bending Only



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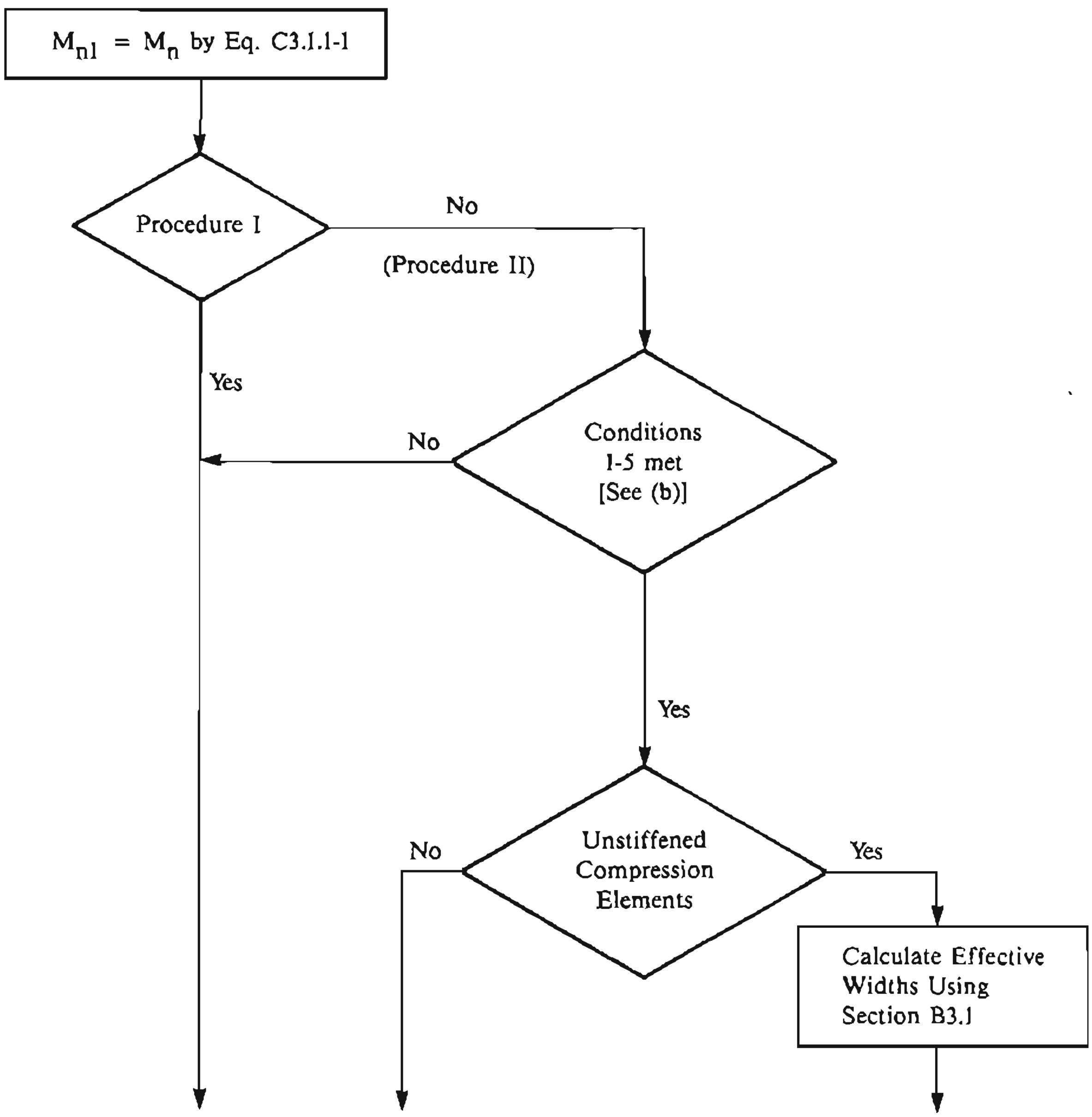
C3.1.1 Nominal Section Strength



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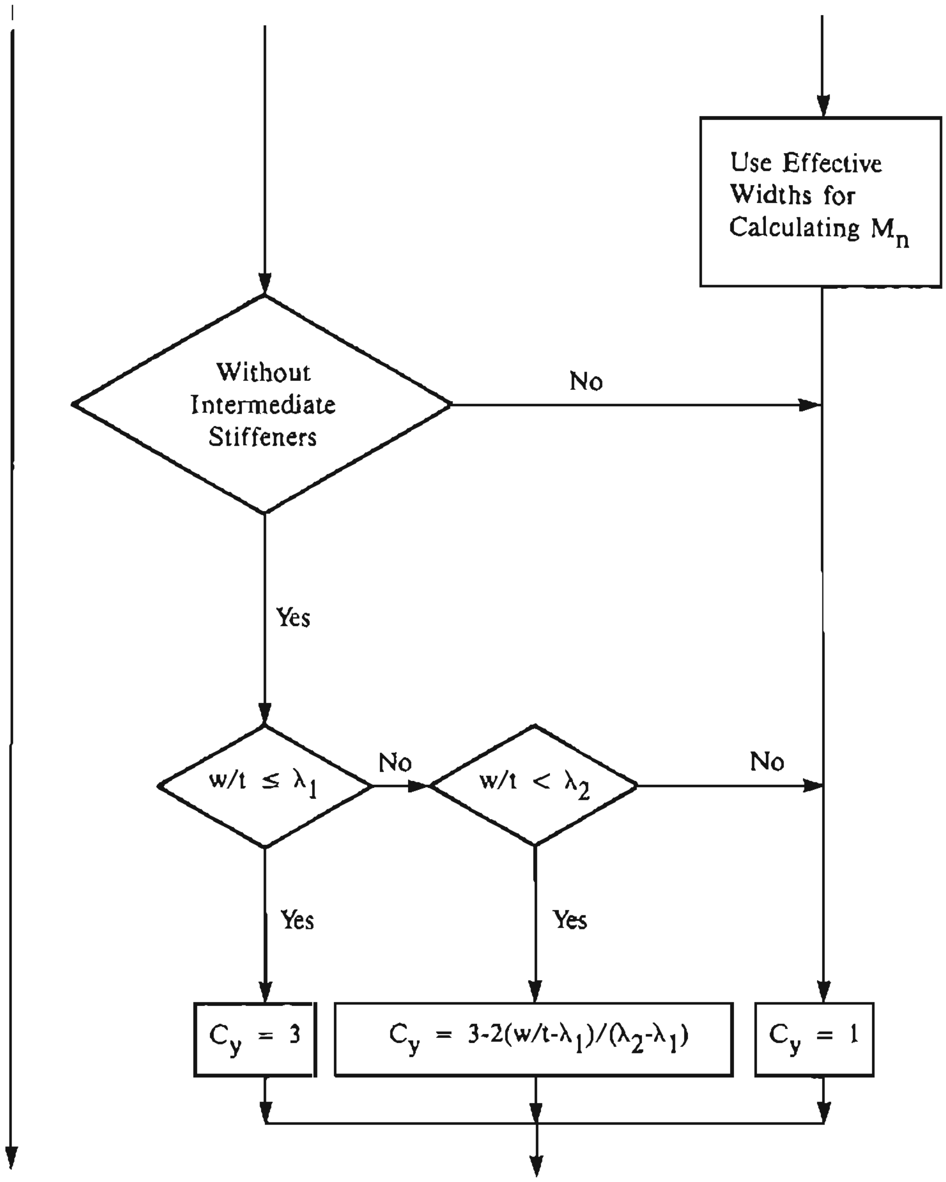

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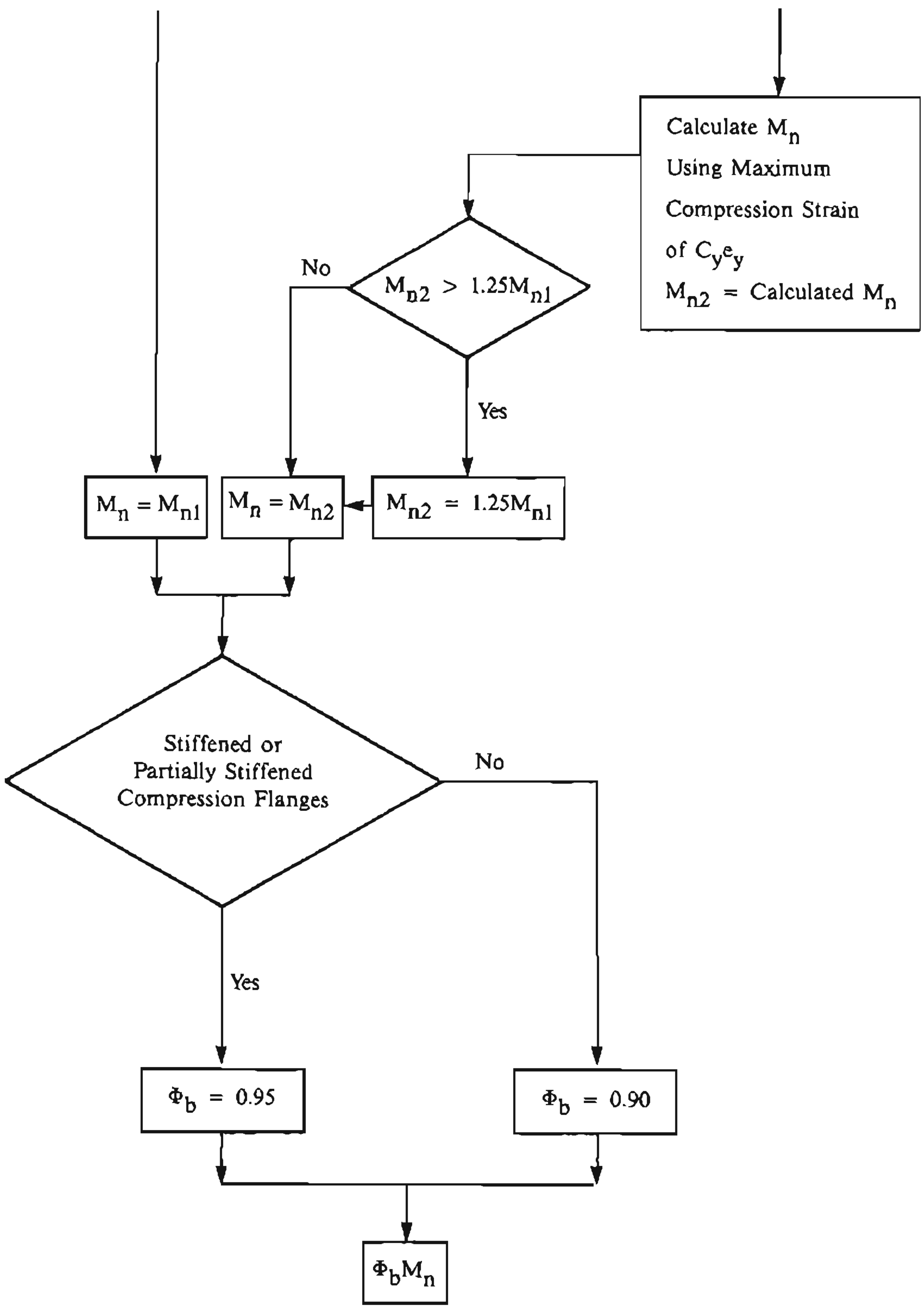
C3.1.1 Nominal Section Strength (continued)



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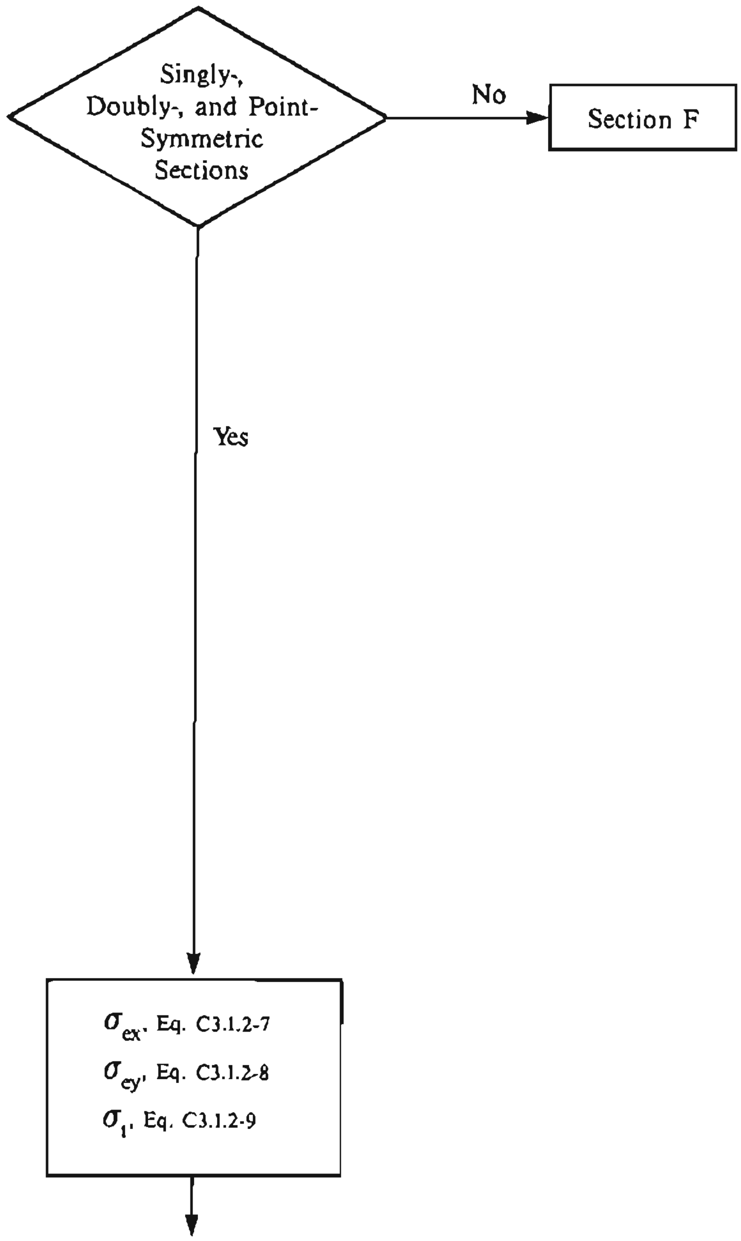
C3.1.2 Lateral Buckling Strength



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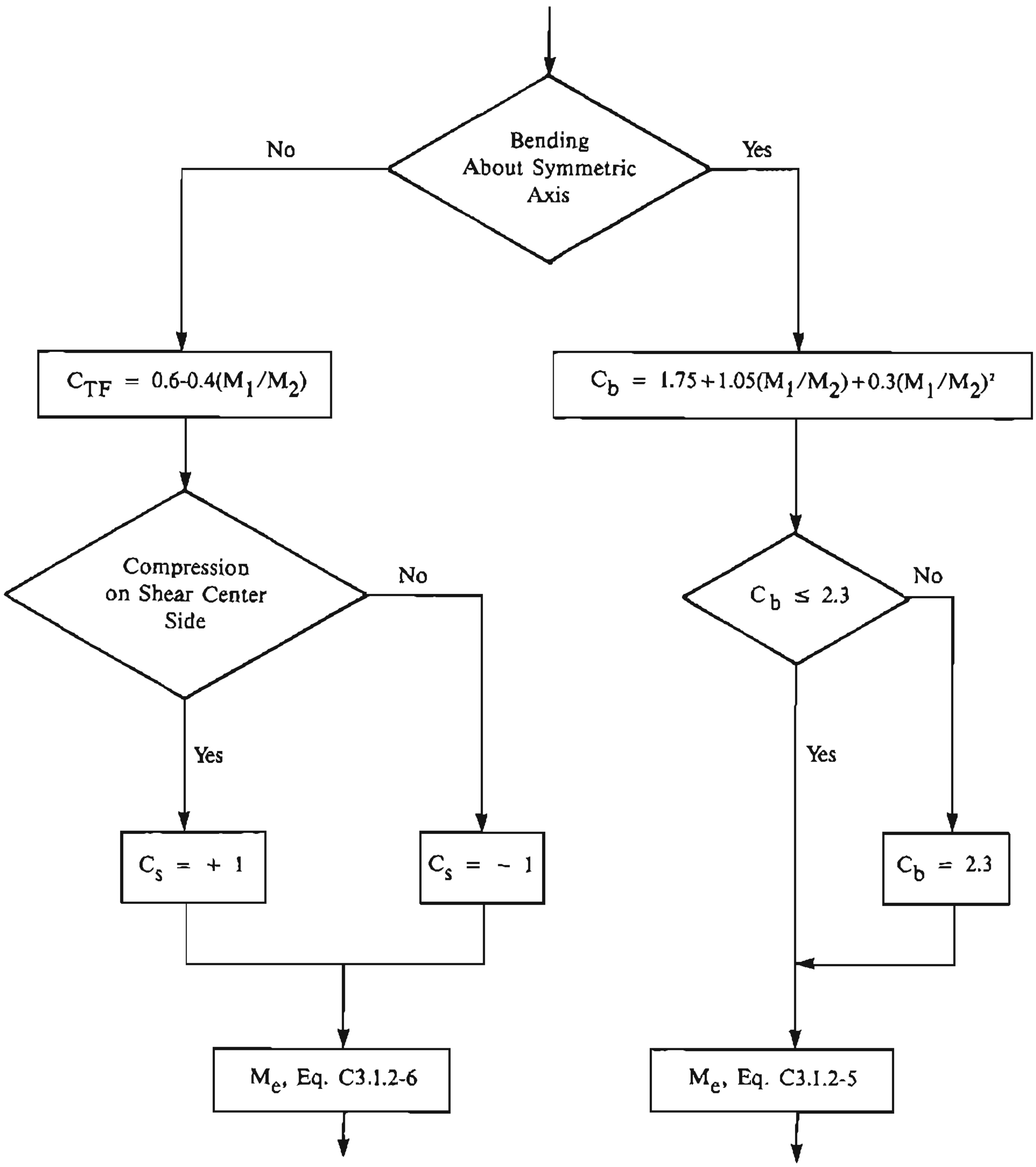
C3.1.2 Lateral Buckling Strength (continued)



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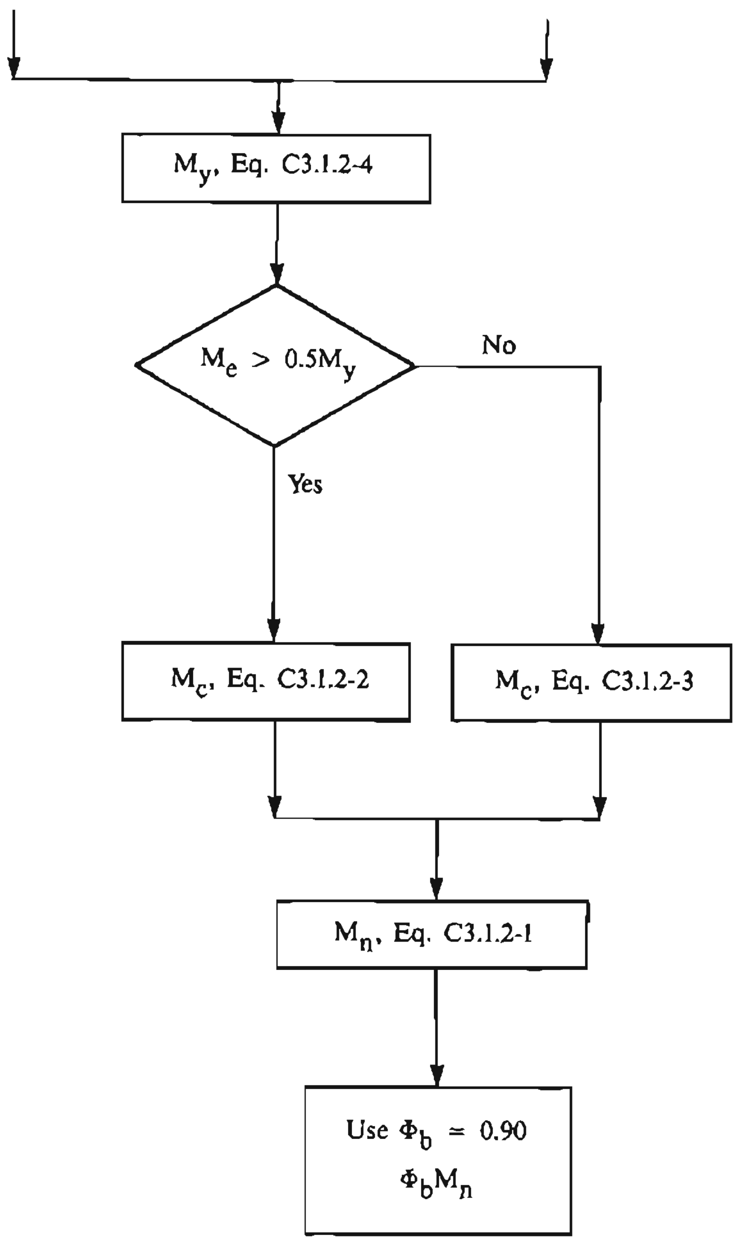
C3.1.2 Lateral Buckling Strength (continued)



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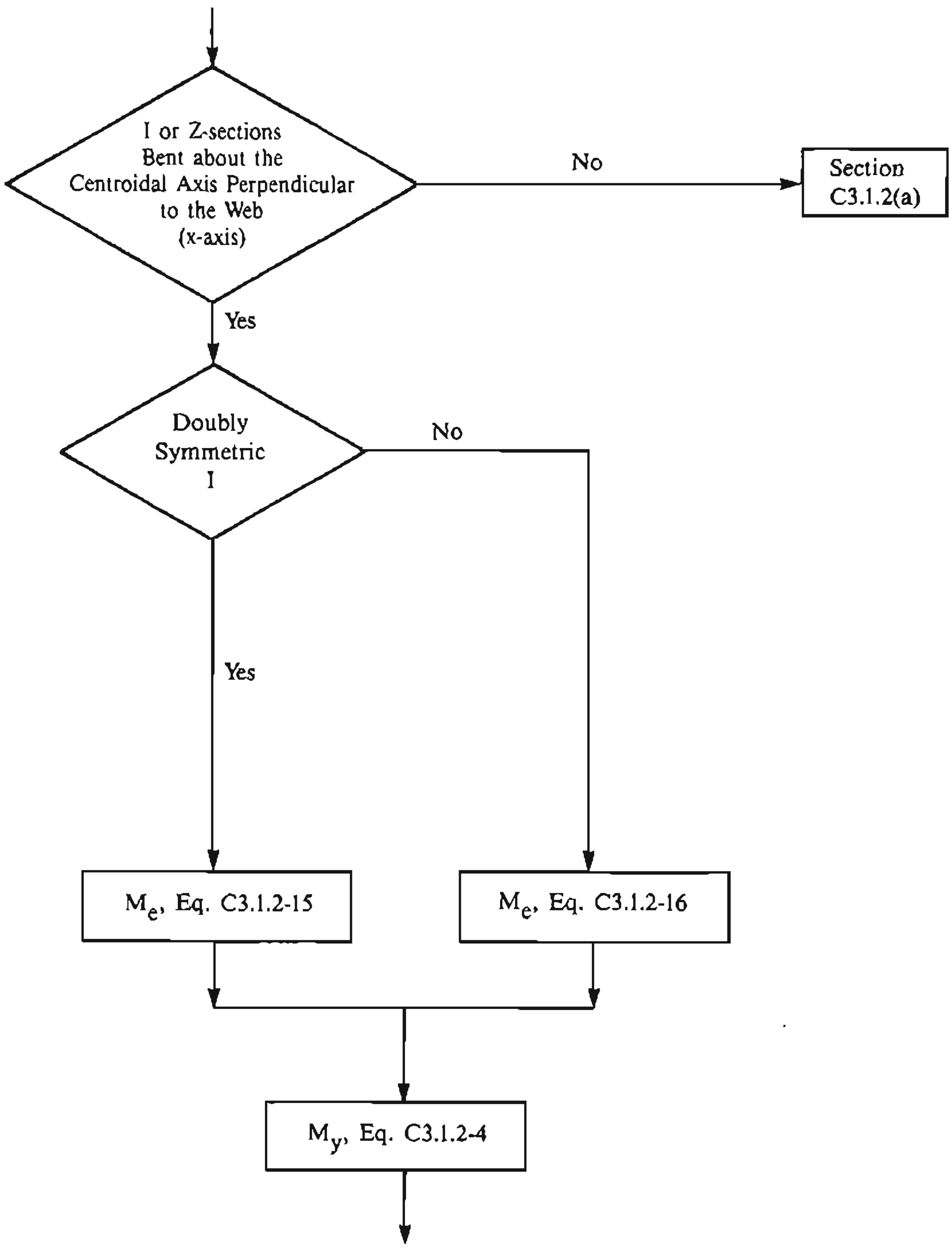
C3.1.2 Lateral Buckling Strength (continued)



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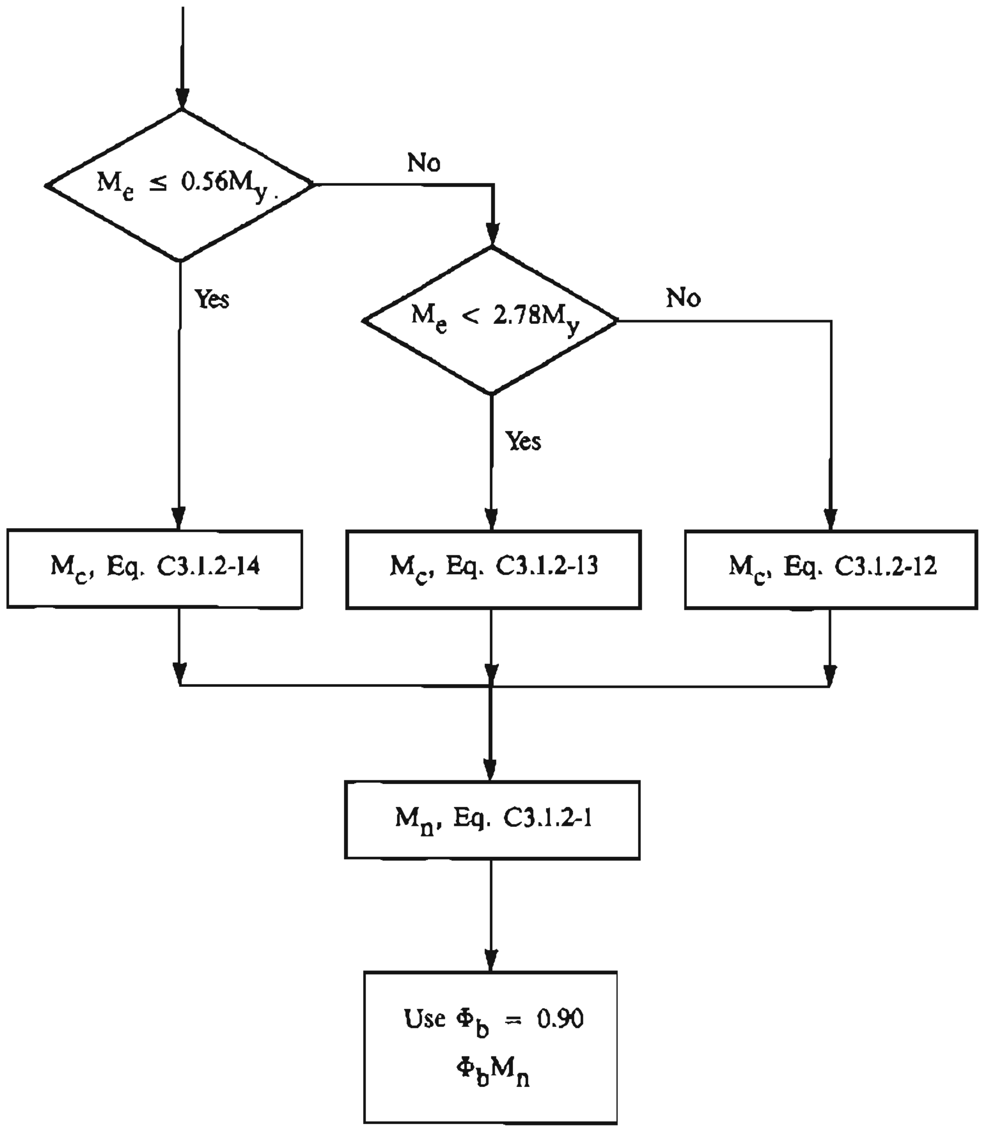
C3.1.2 Lateral Buckling Strength (continued)



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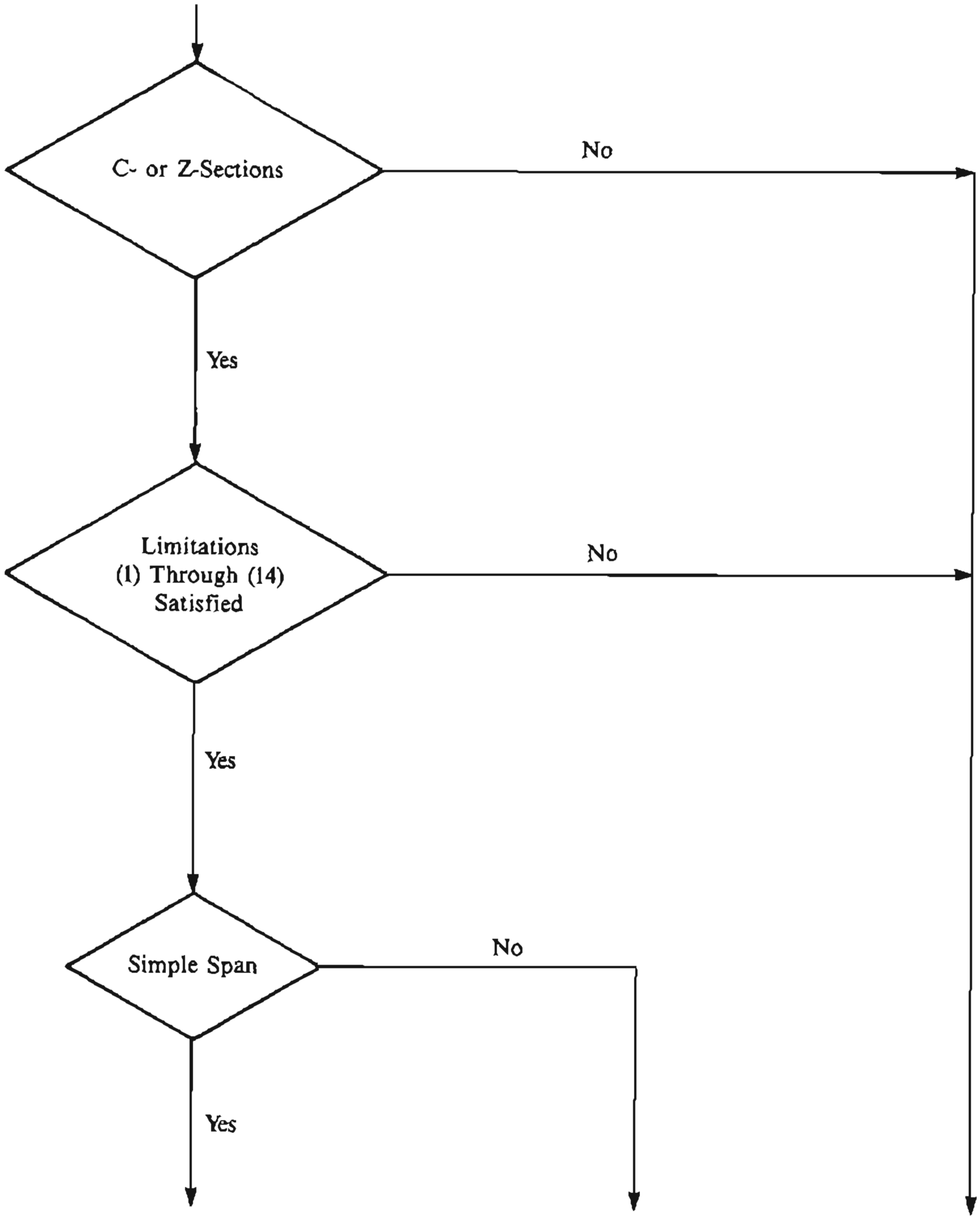
C3.1.3 Beams Having One Flange Through-Fastened to Deck or Sheathing



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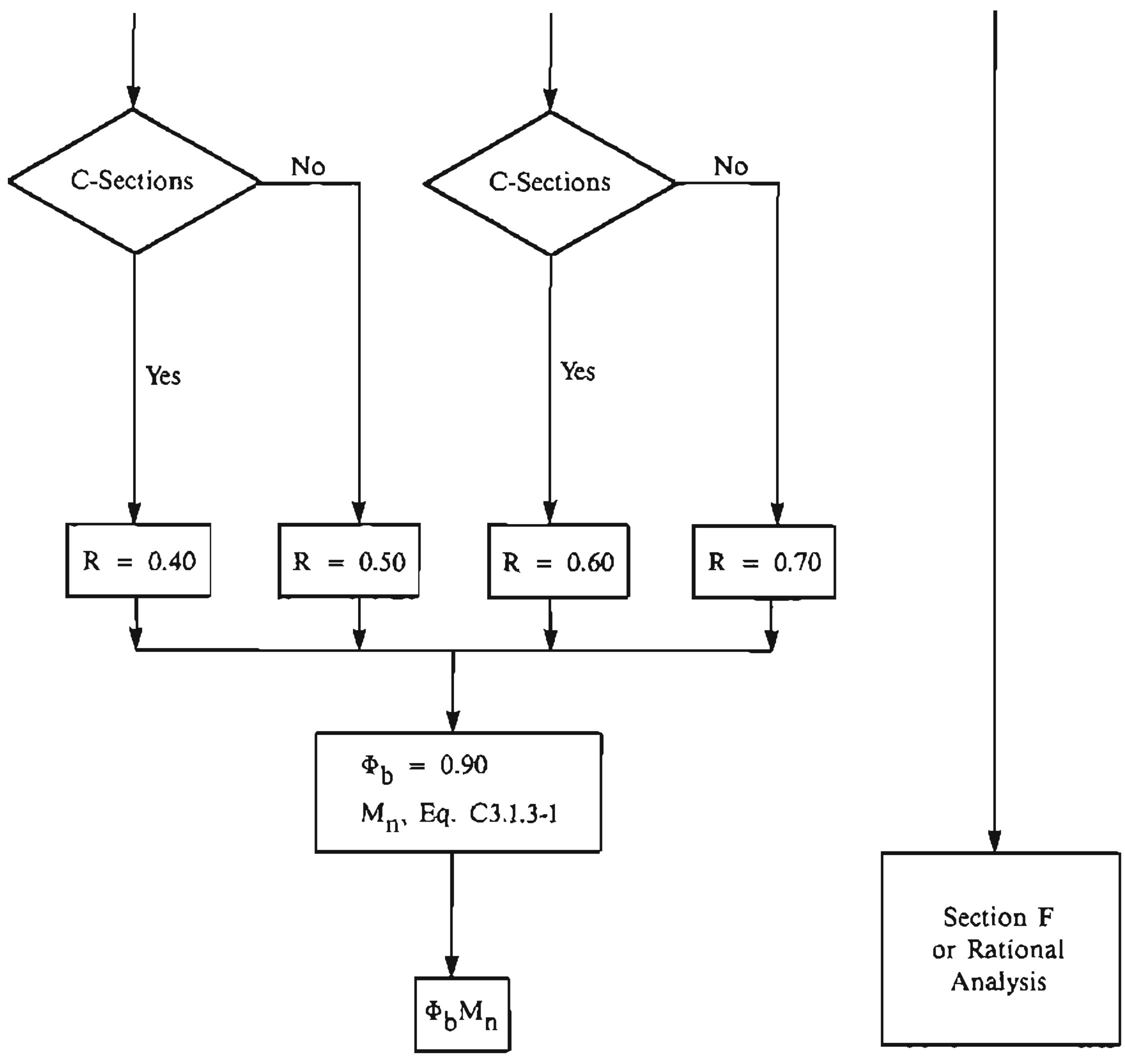
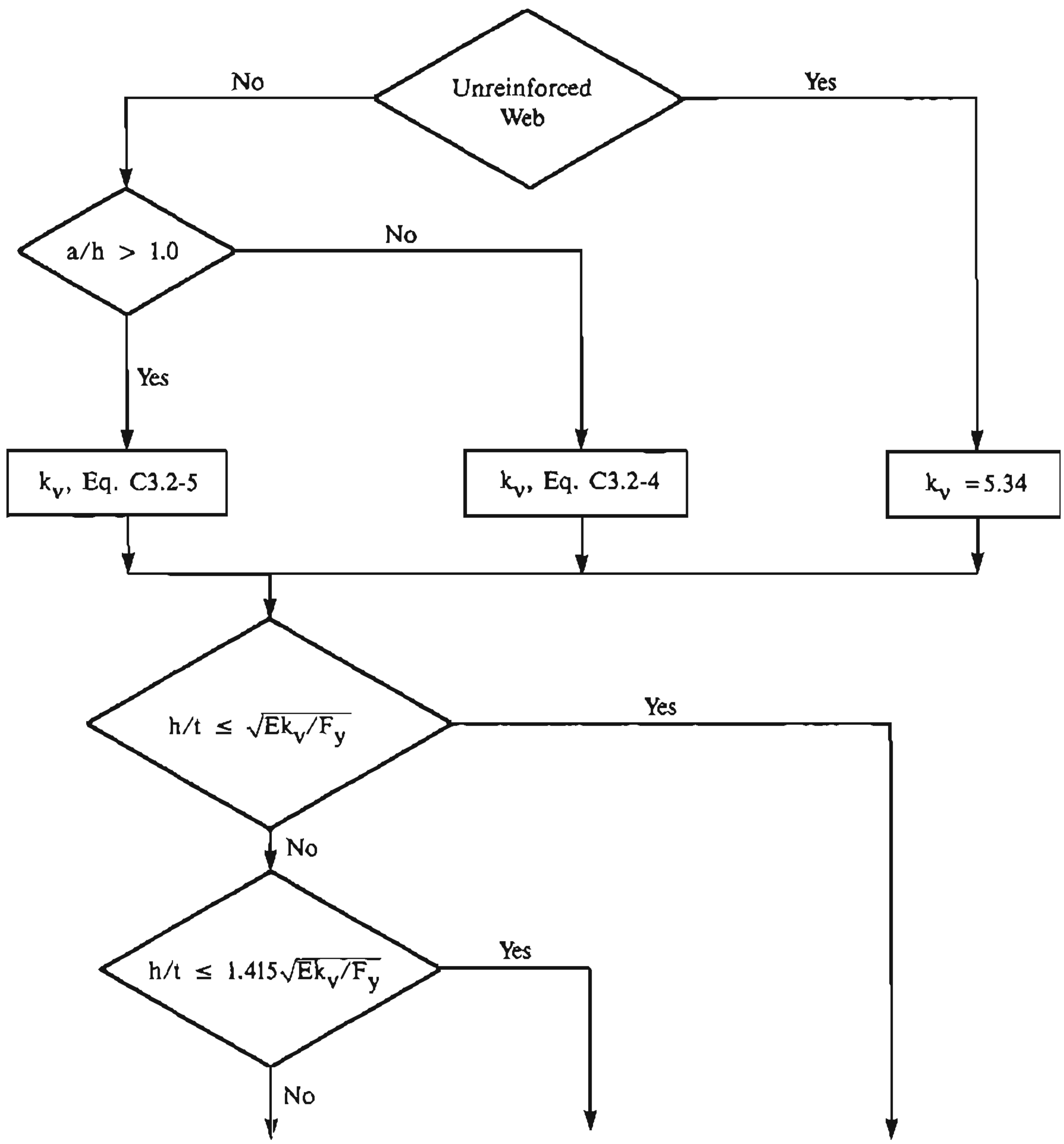
C3.1.3 Beams Having One Flange Through-Fastened to Deck or Sheathing



C3.2 Strength for Shear Only



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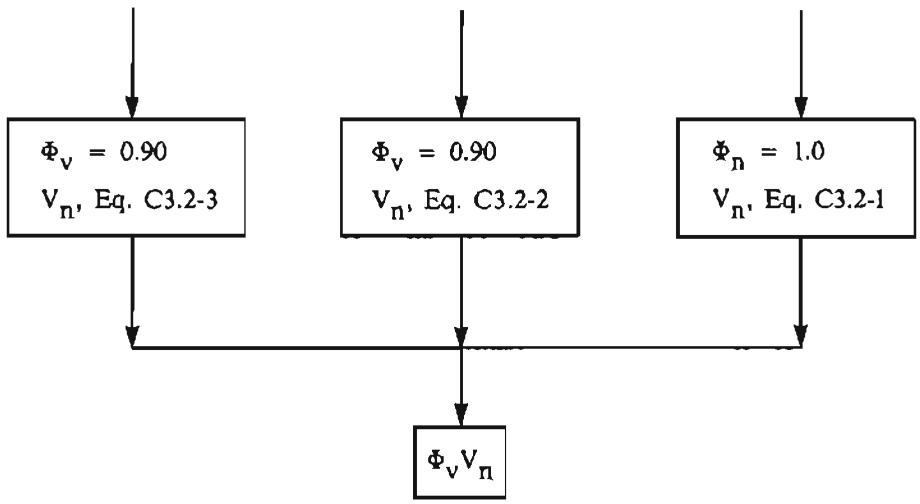
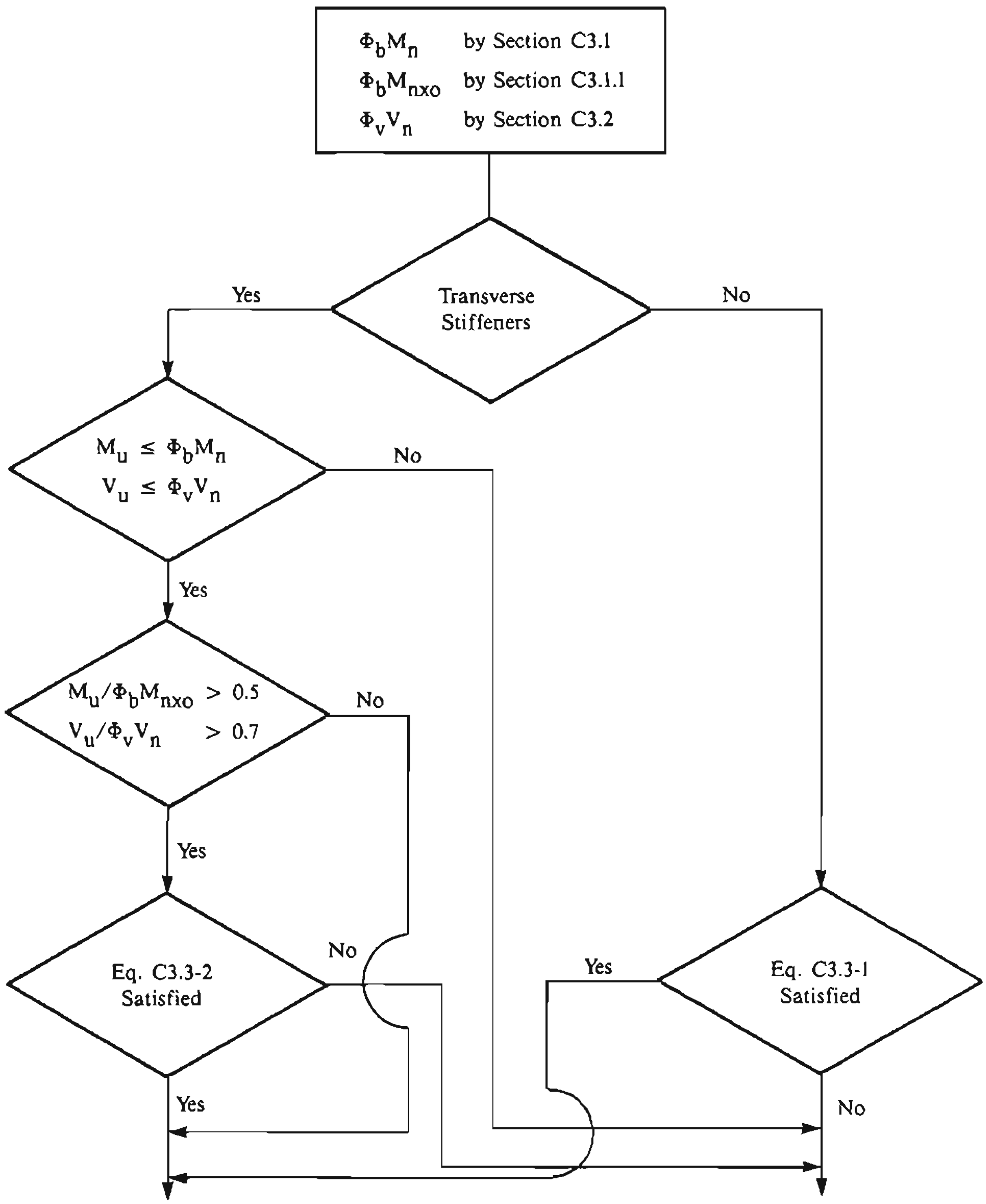
C3.2 Strength for Shear Only (continued)



C3.3 Strength for Combined Bending and Shear



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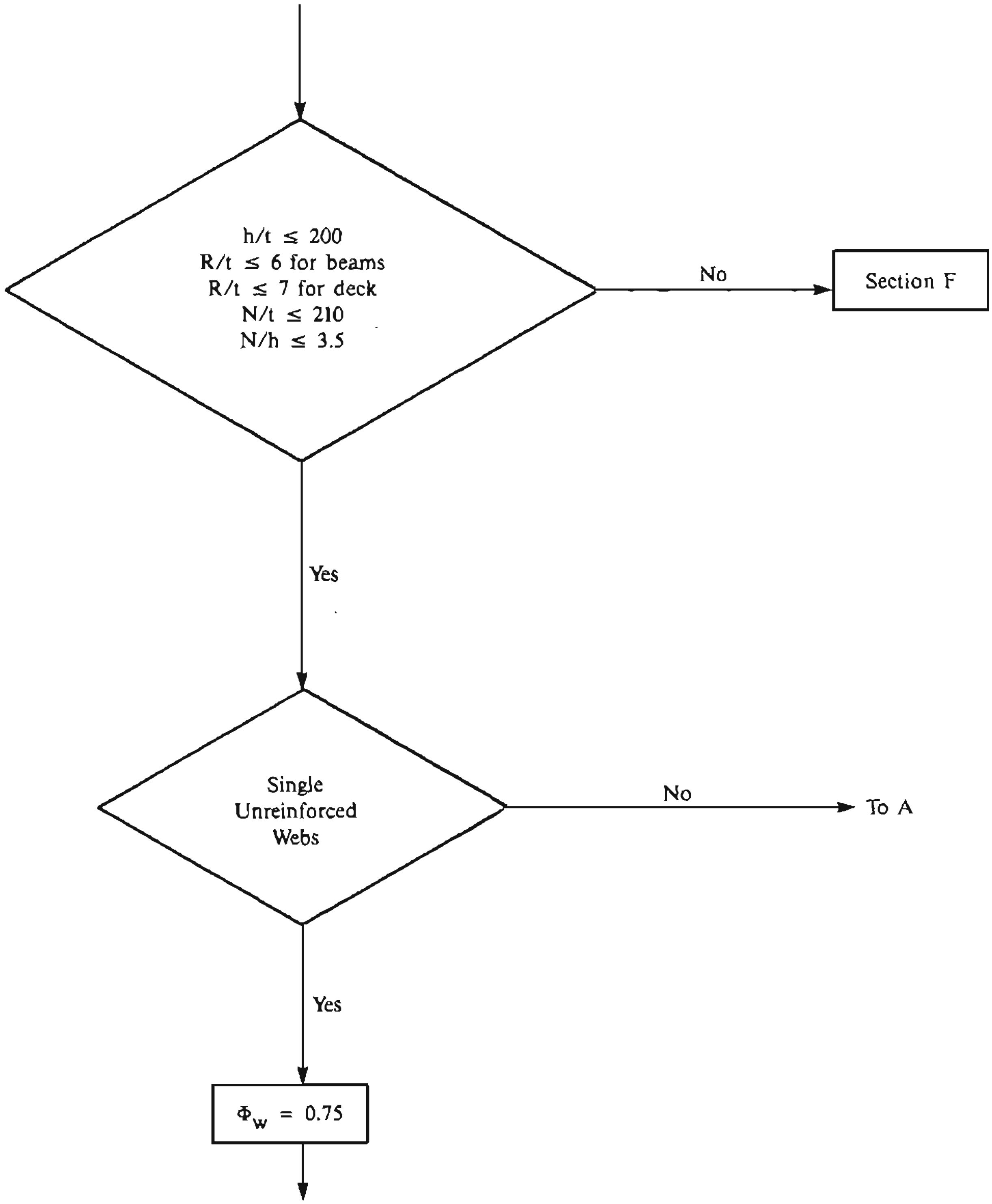
C3.3 Strength for Combined Bending and Shear (continued)

C3.4 Web Crippling Strength



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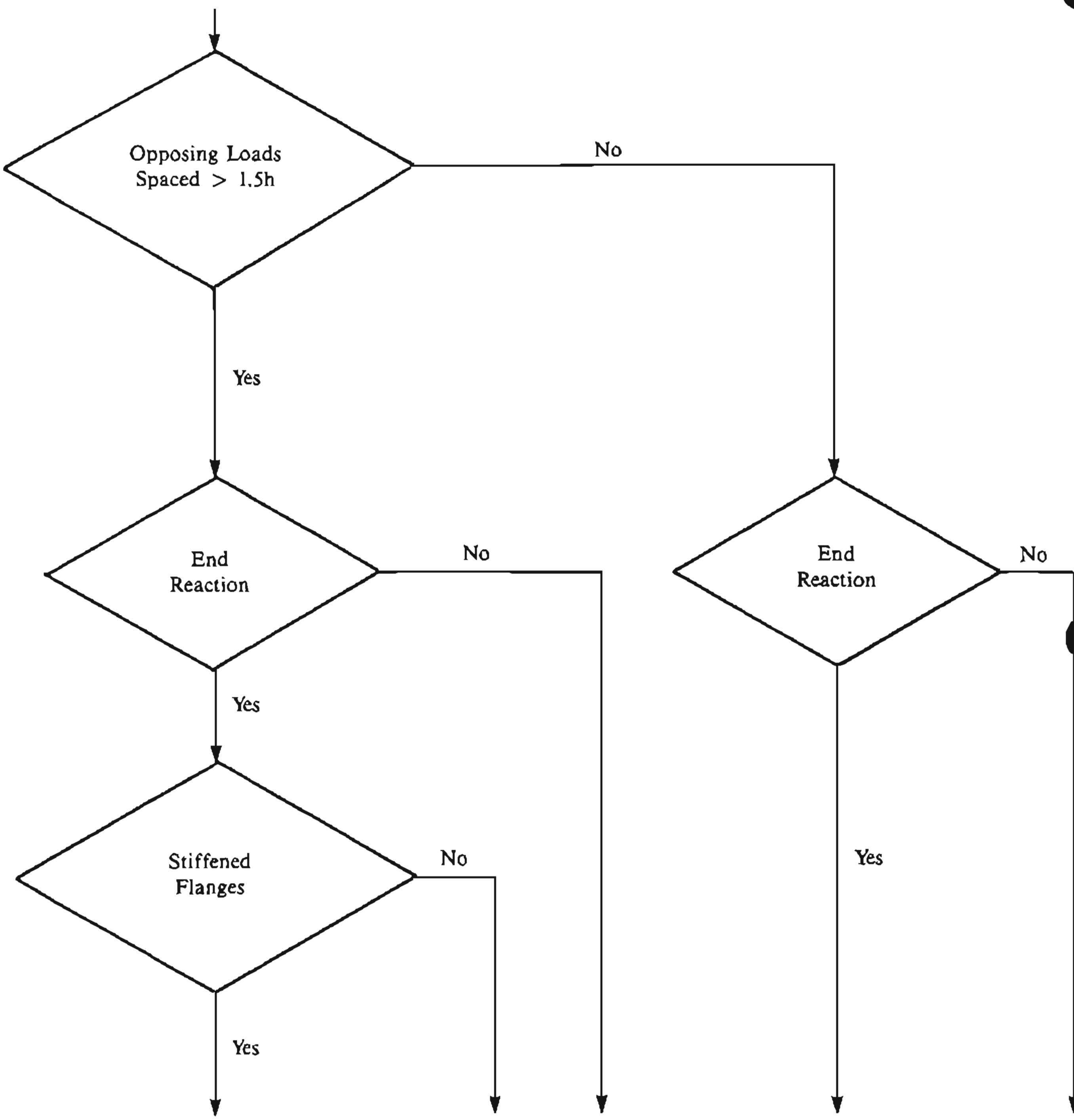
C3.4 Web Crippling Strength (continued)



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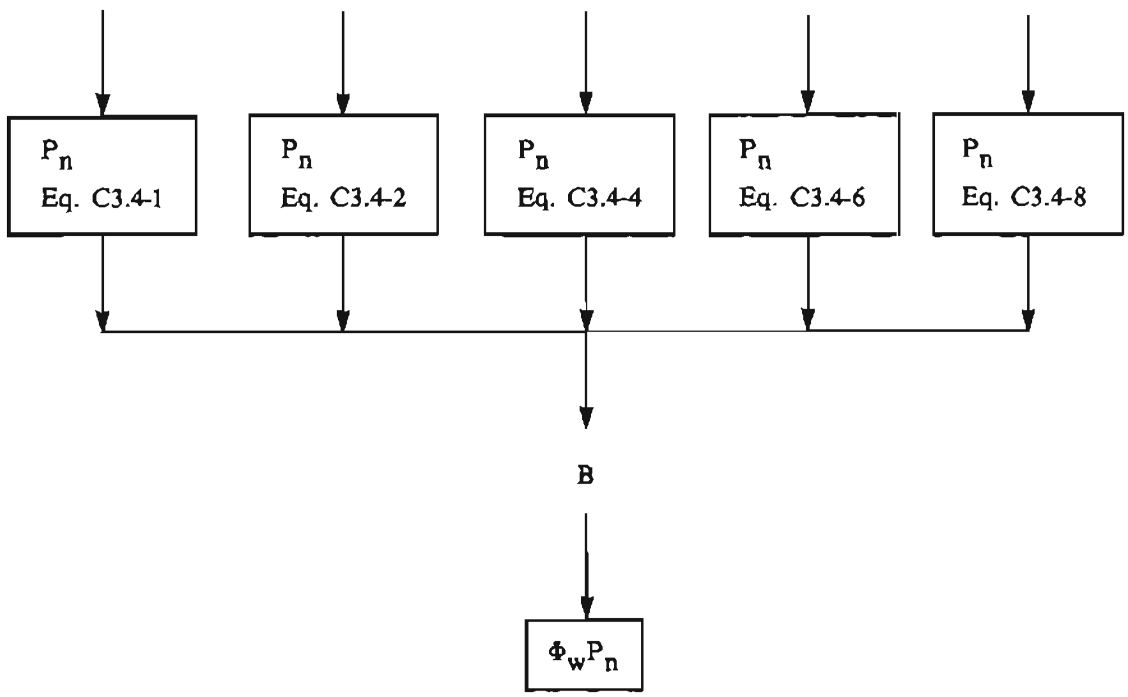

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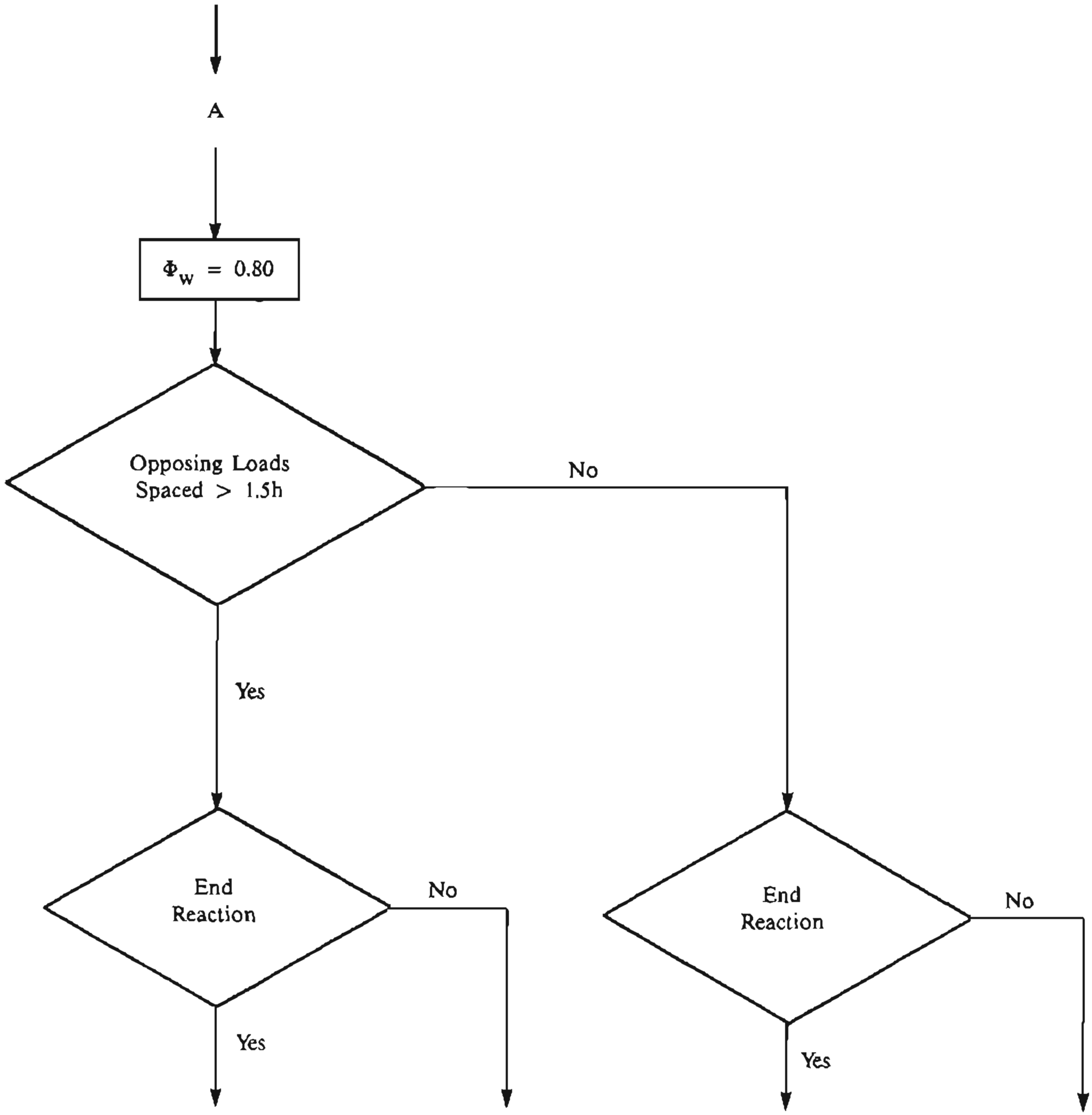
C3.4 Web Crippling Strength (continued)



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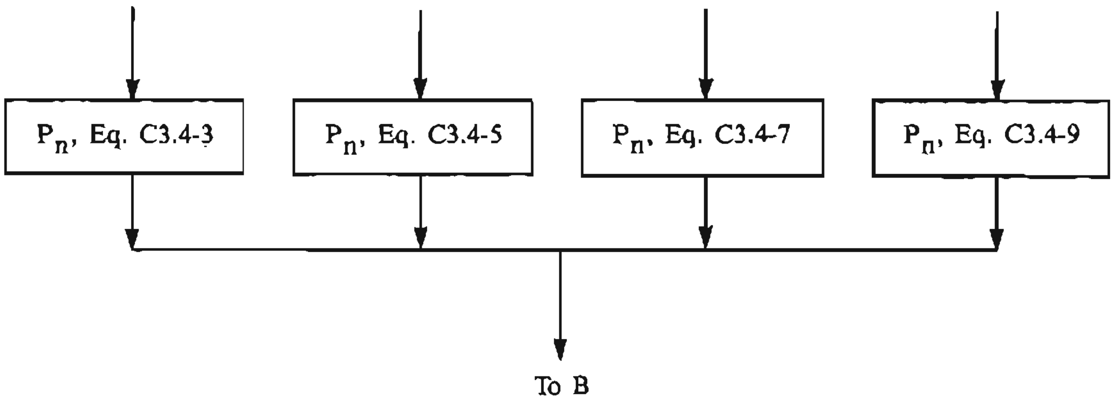
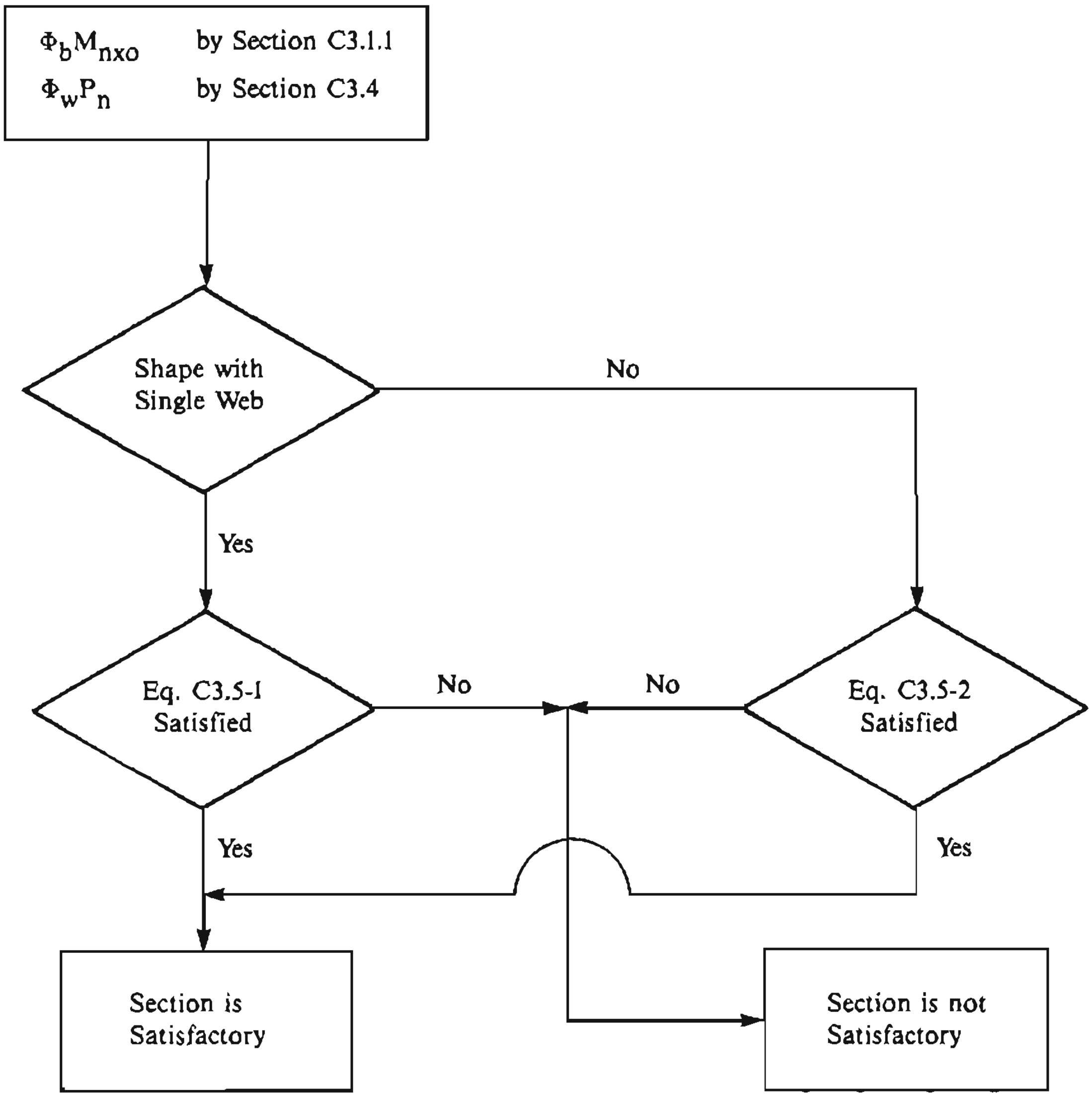
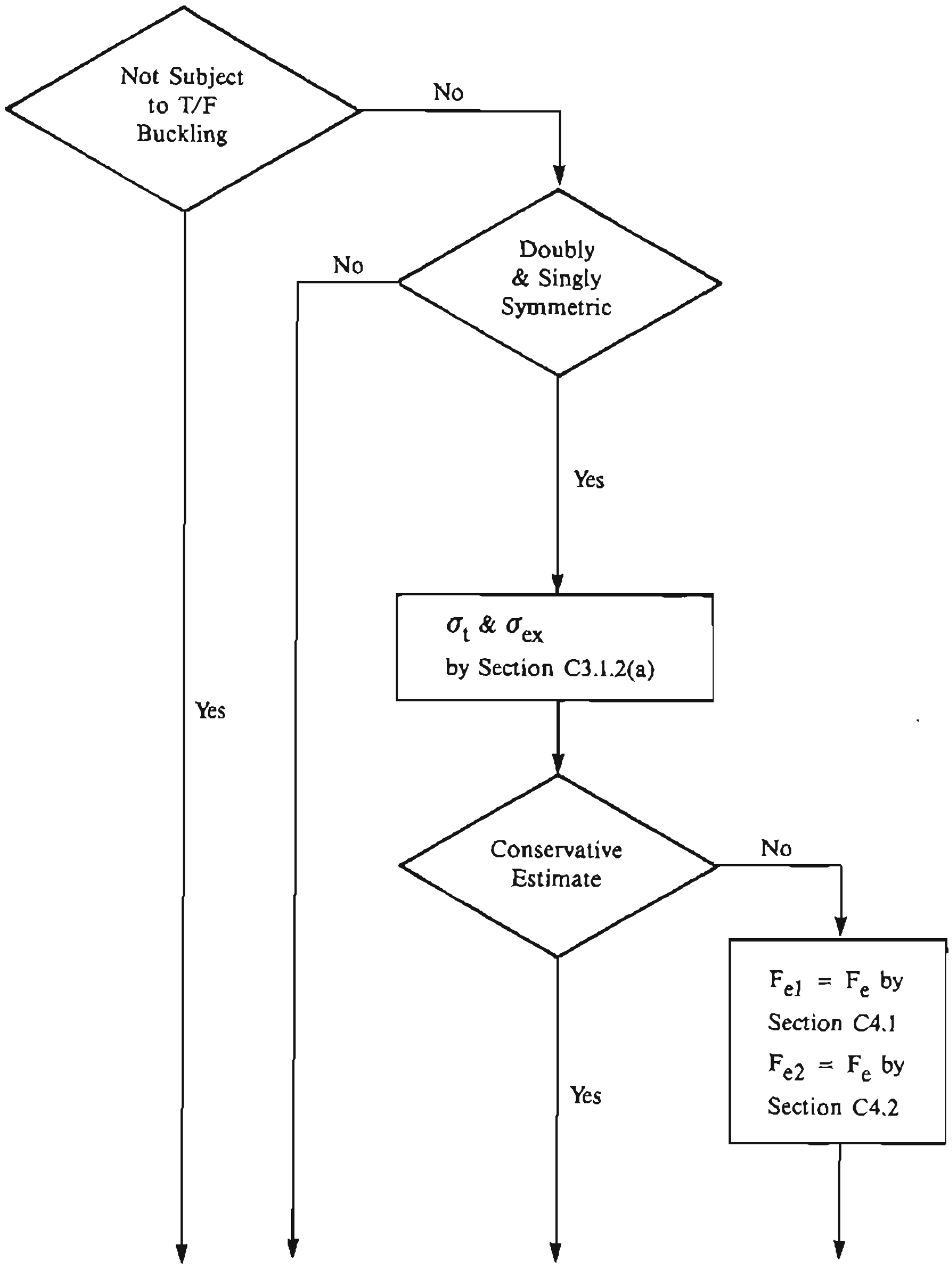
C3.4 Web Crippling Strength (continued)



C3.5 Combined Bending and Web Crippling Strength



C4 Concentrically Loaded Compression Members



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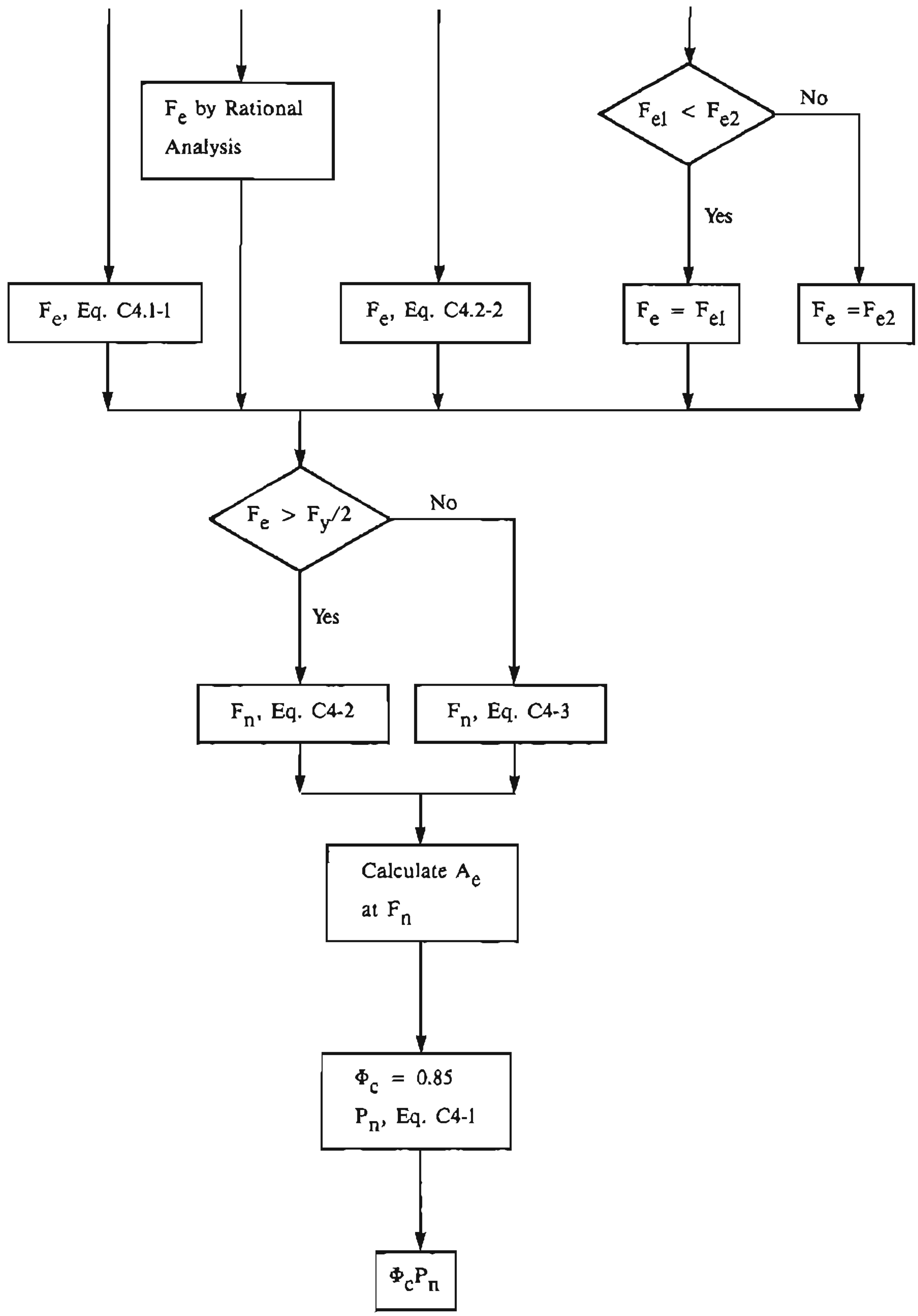
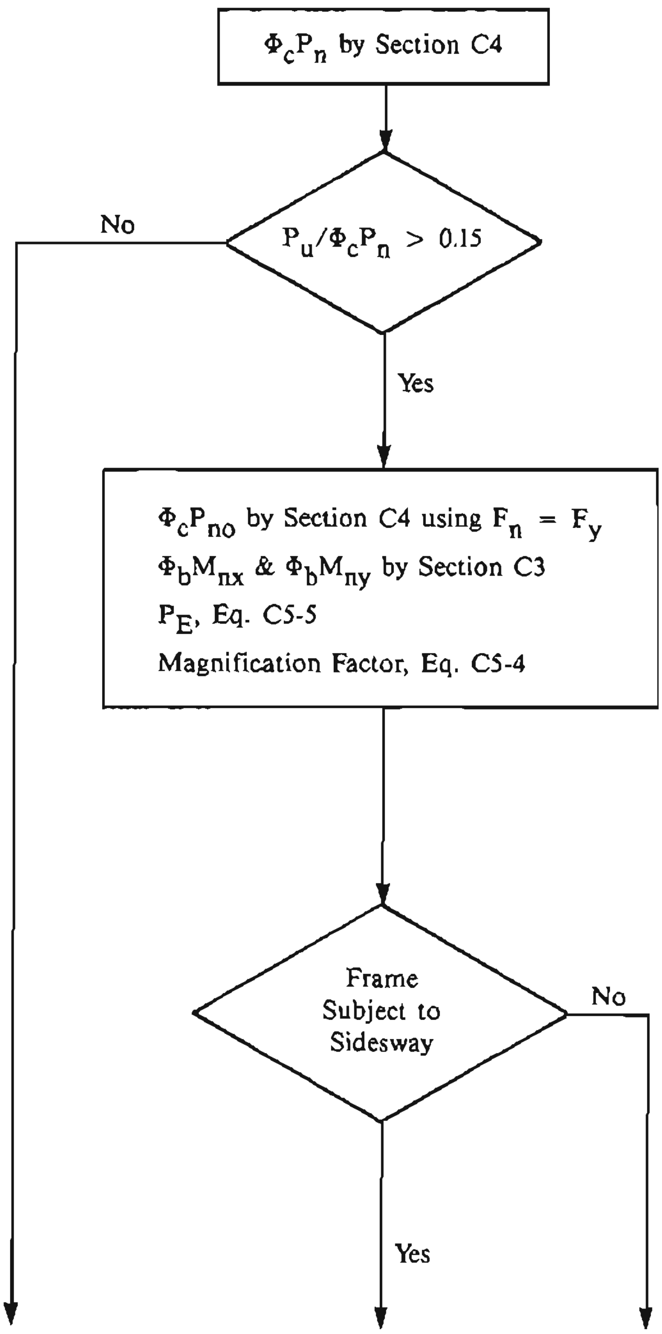
C4 Concentrically Loaded Compression Members (continued)



C5 Combined Axial Load and Bending



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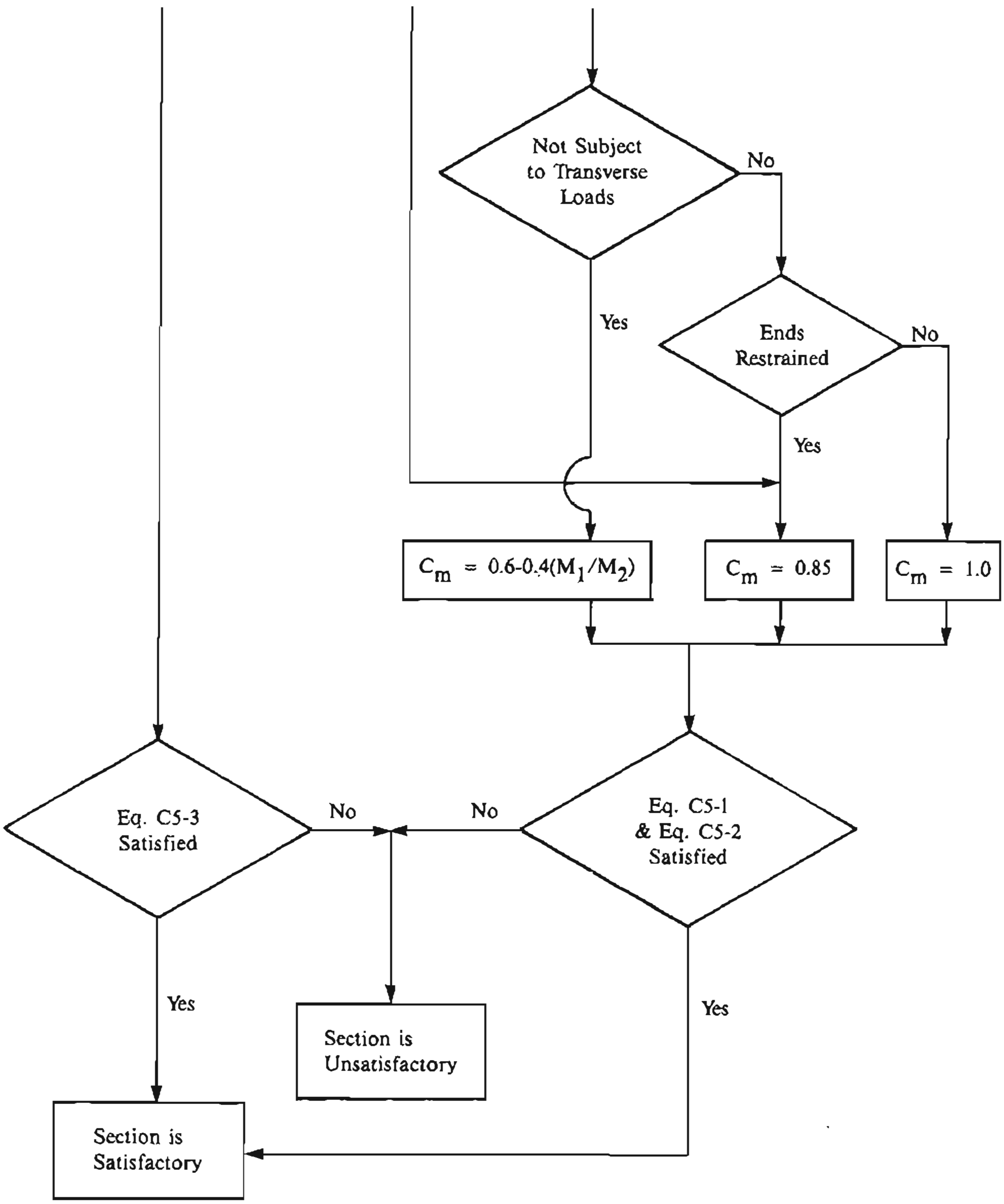
C5 Combined Axial Load and Bending (continued)



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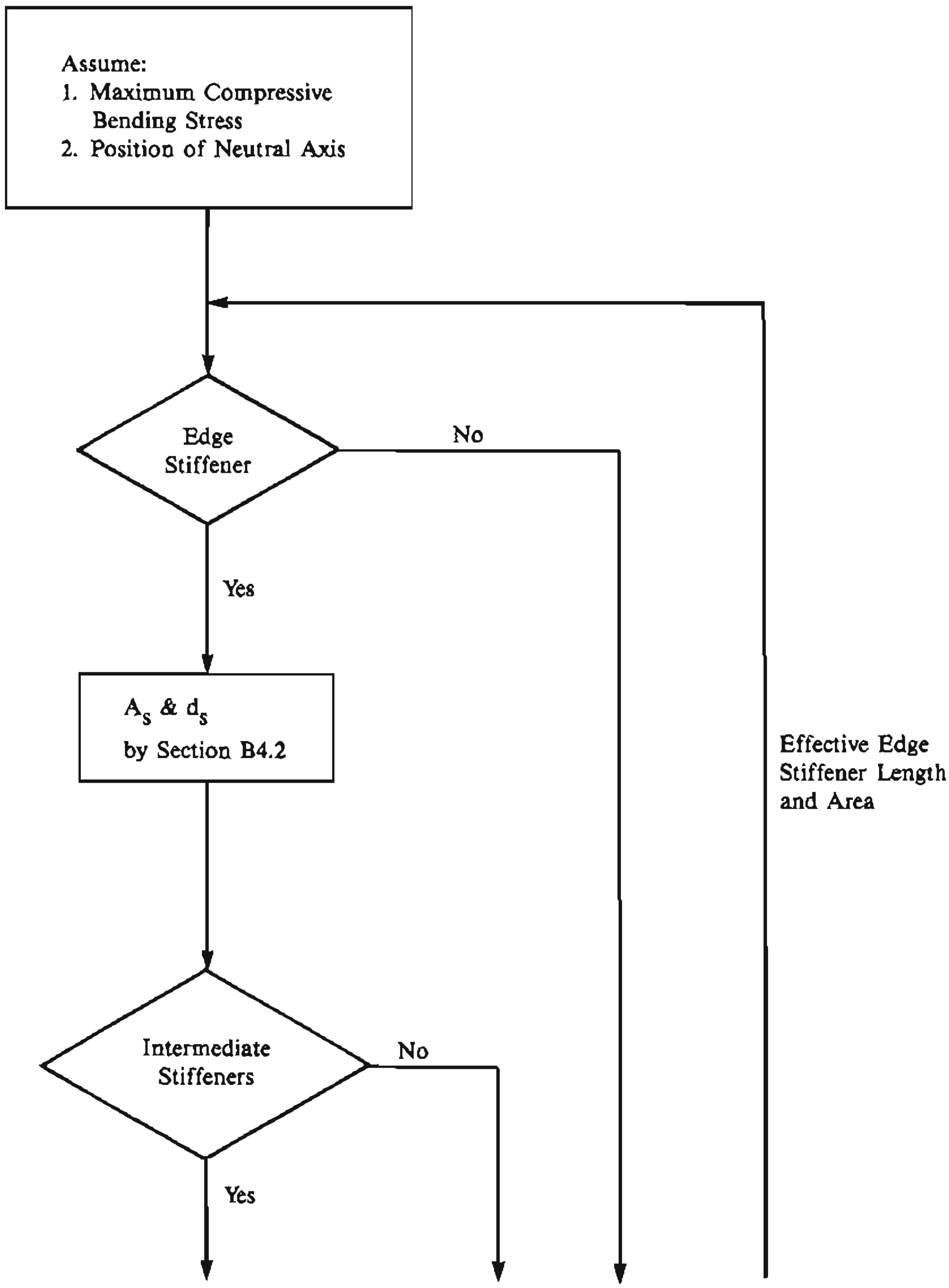
Moment Capacity of Flexural Members



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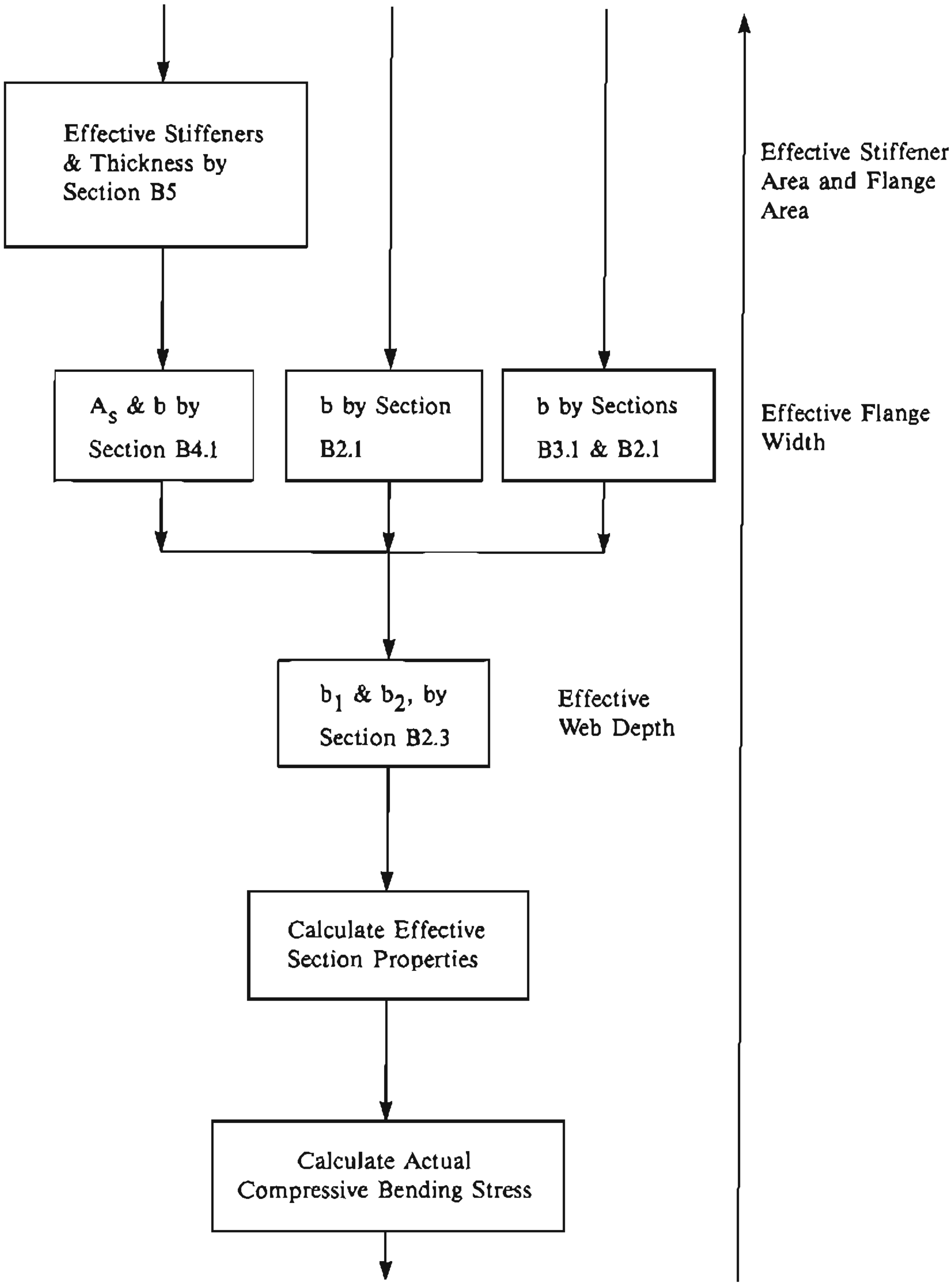
Moment Capacity of Flexural Members (continued)



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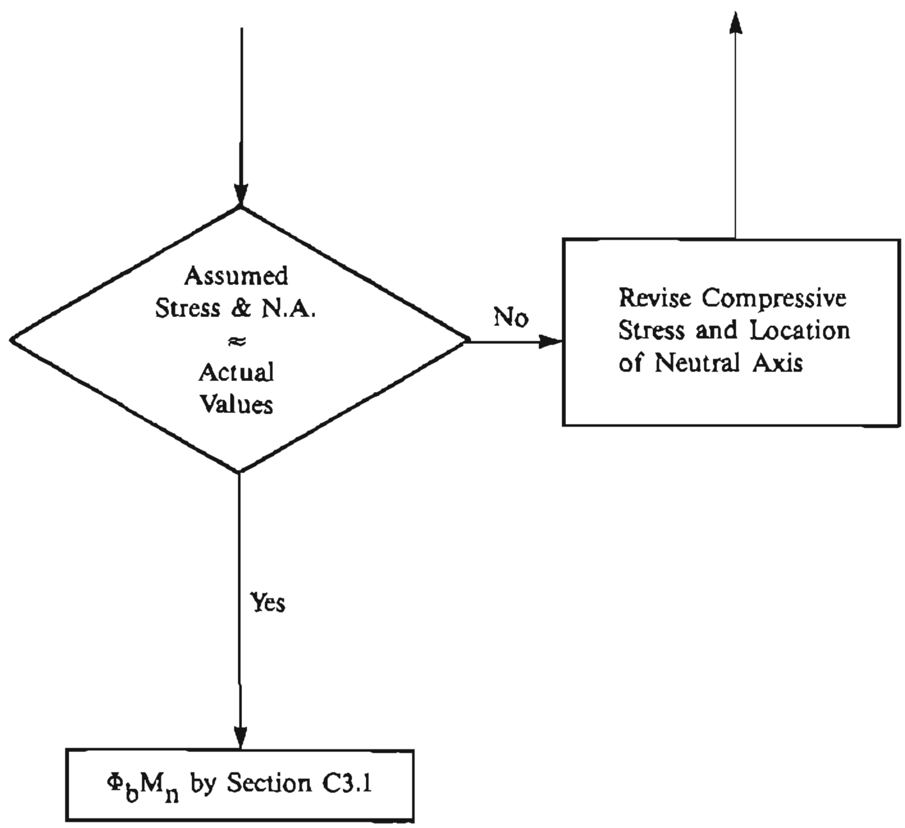
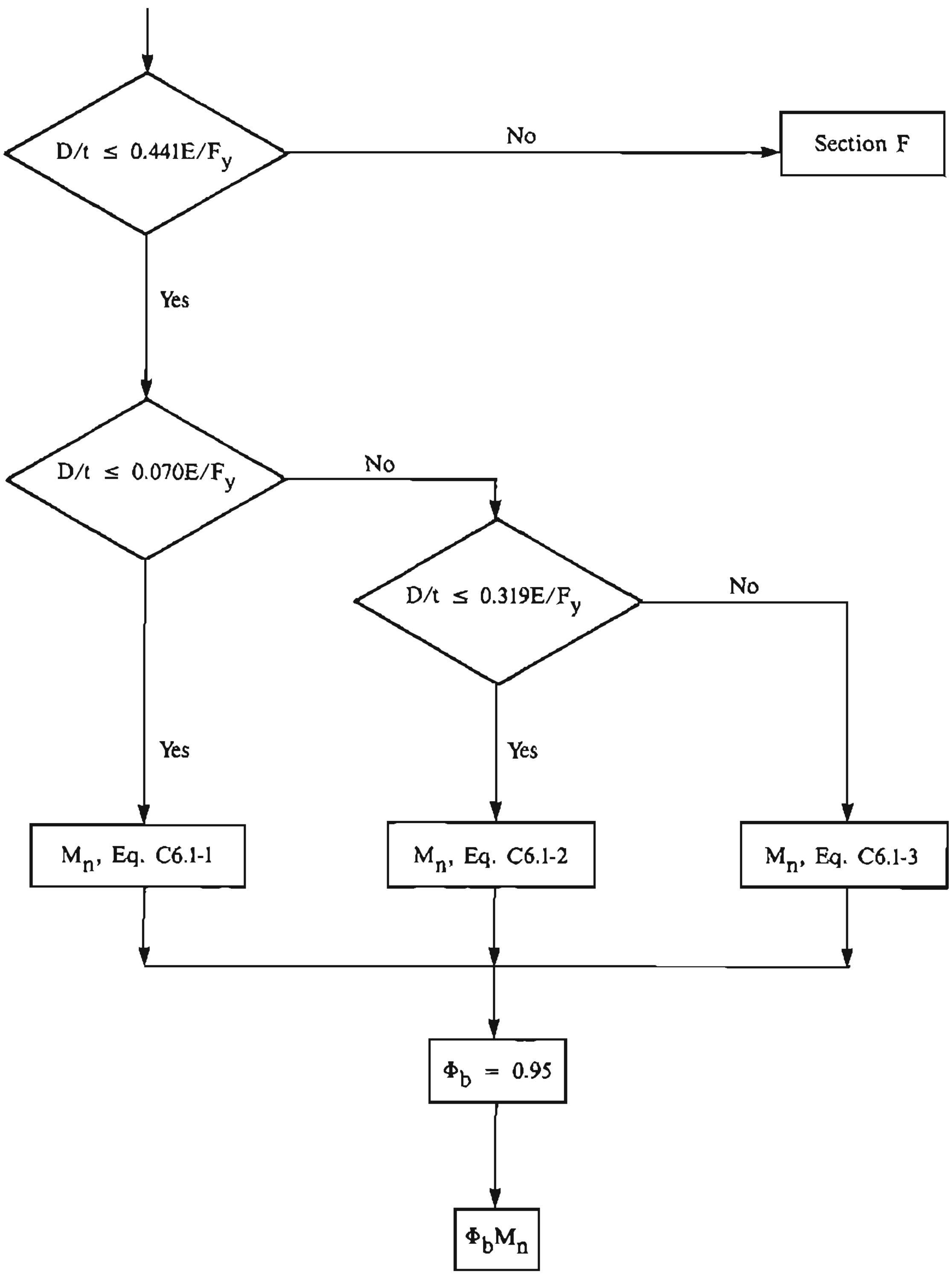
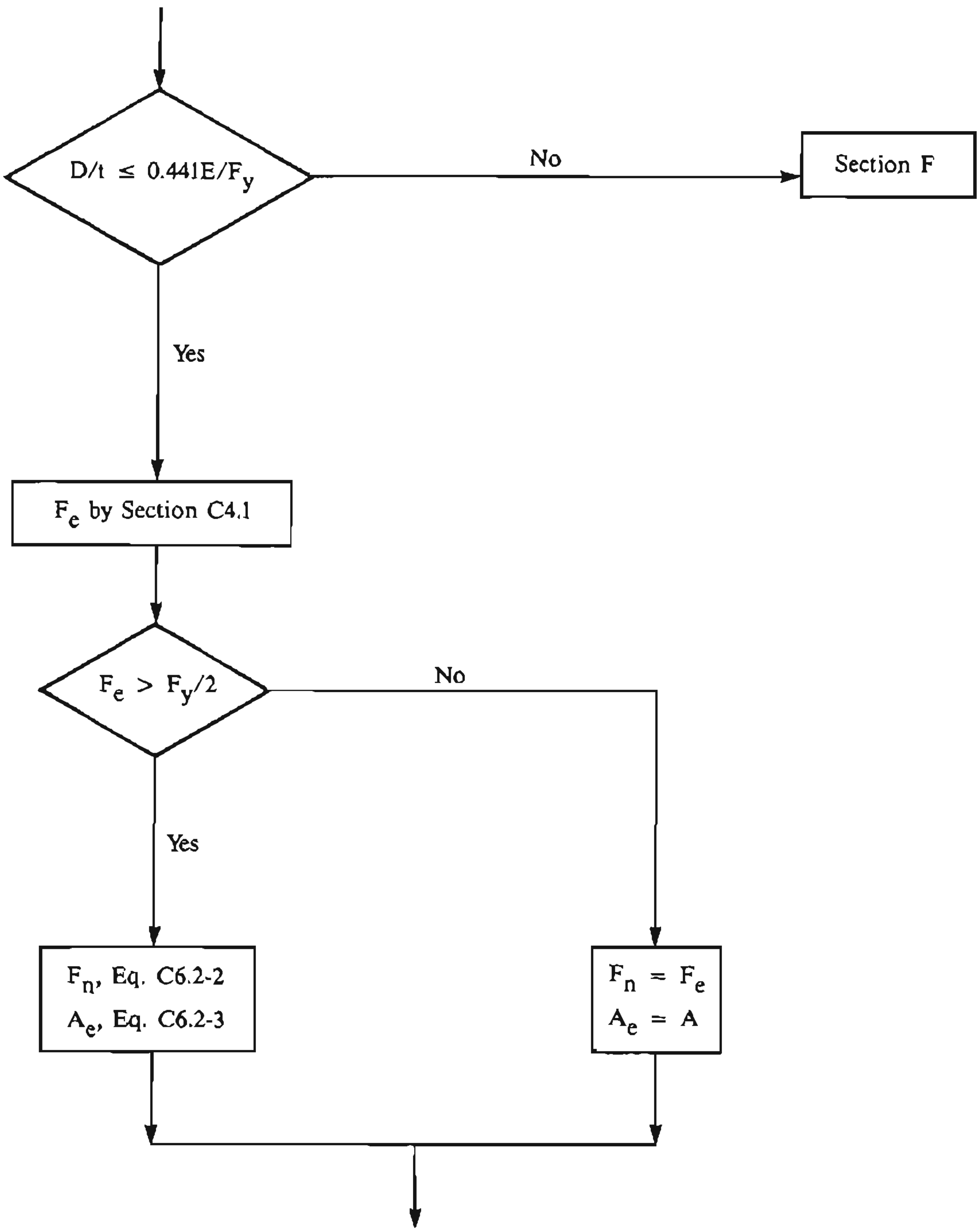
Moment Capacity of Flexural Members (continued)



C6.1 Bending - Cylindrical Tubular Members



C6.2 Compression - Cylindrical Tubular Members



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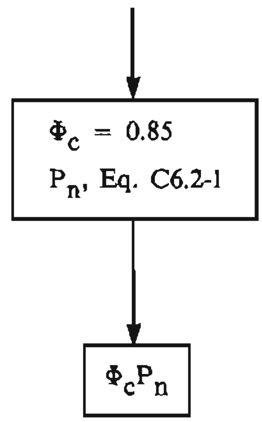
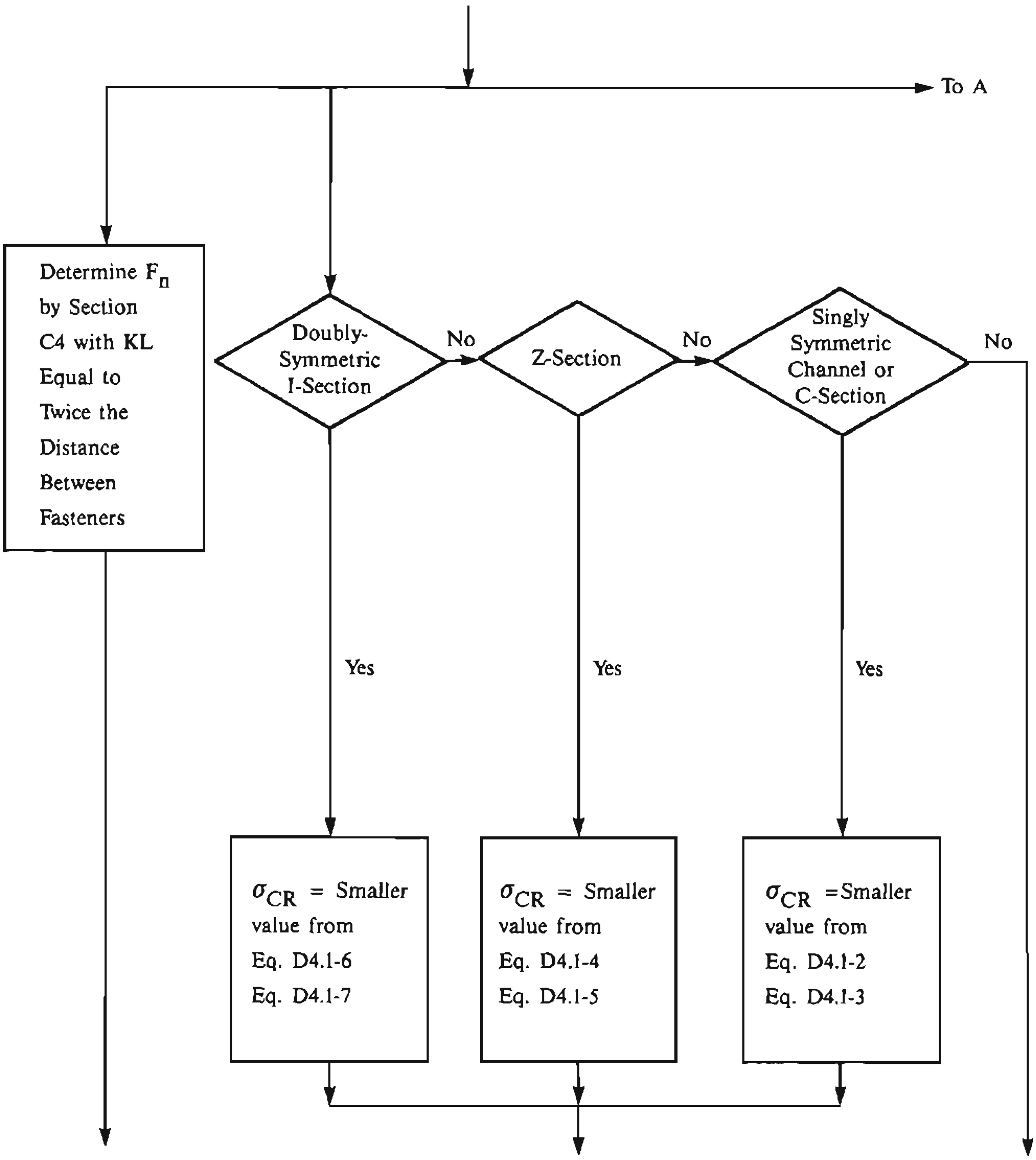
C6.2 Compression - Cylindrical Tubular Members (continued)



D4.1 Capacity of a Wall Stud



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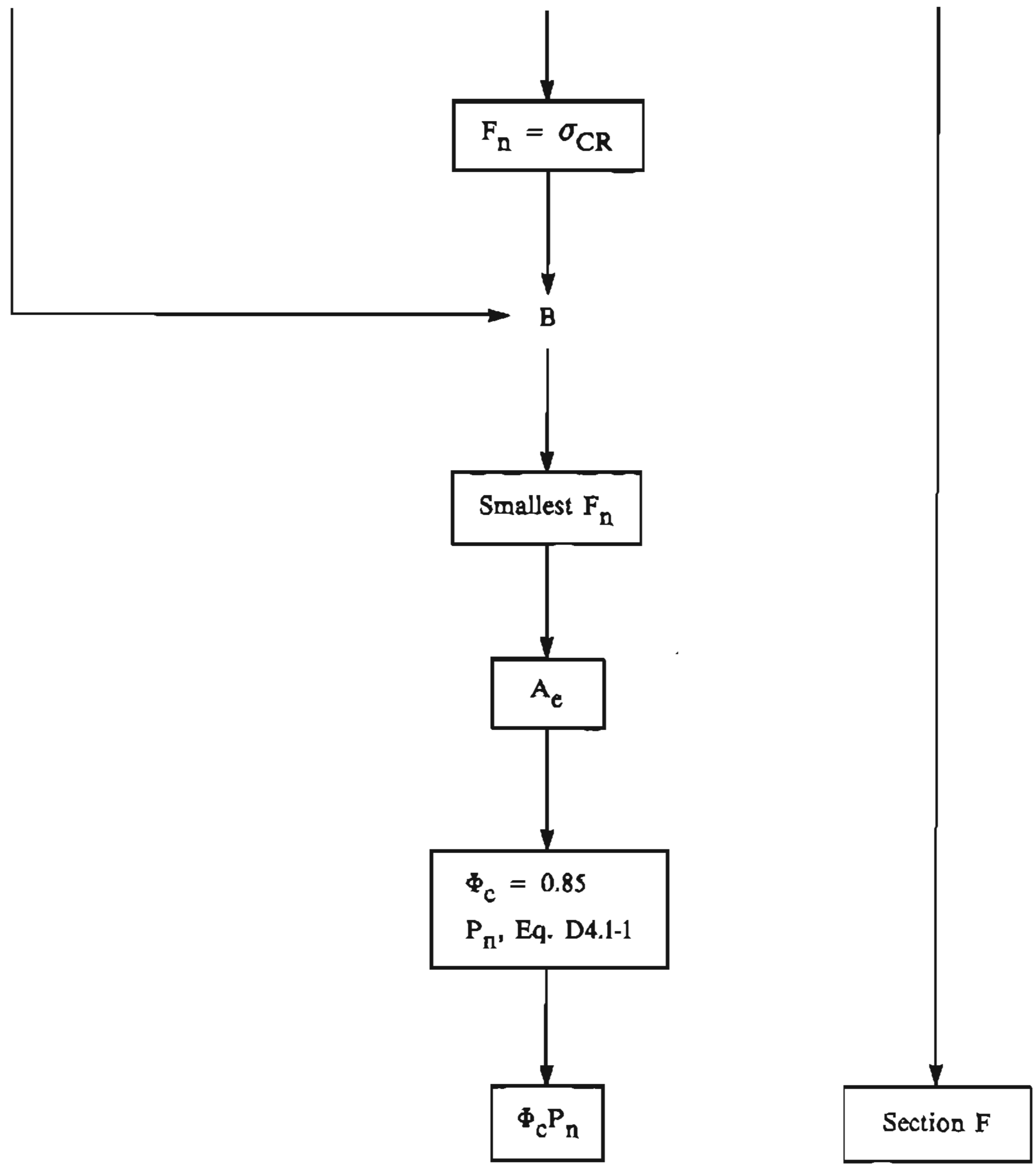
D4.1 Capacity of a Wall Stud (continued)



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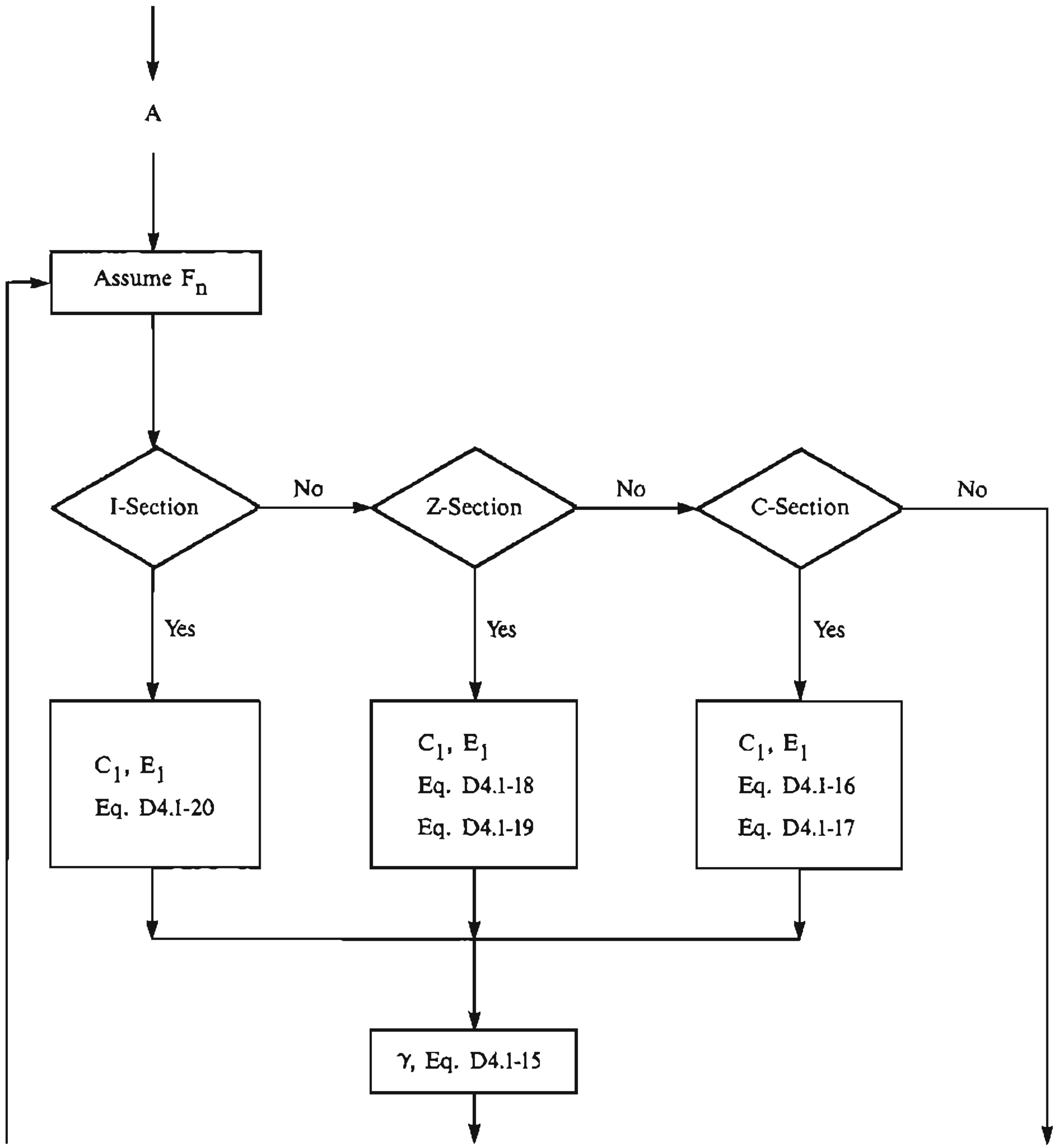
D4.1 Capacity of a Wall Stud (continued)



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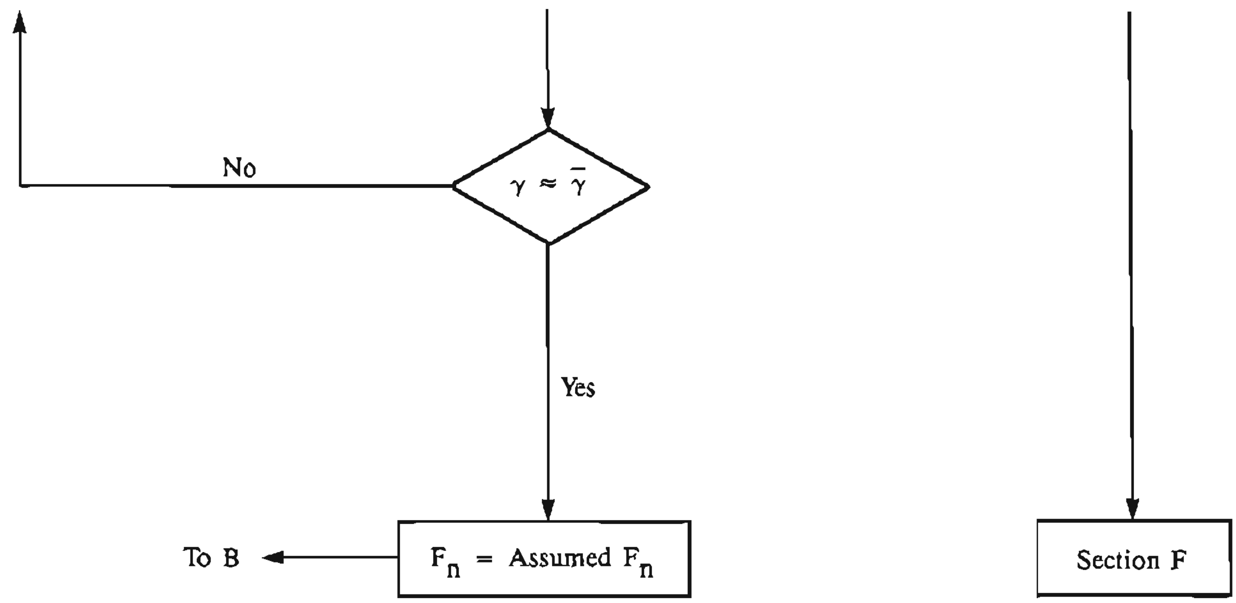
D4.1 Capacity of a Wall Stud (continued)



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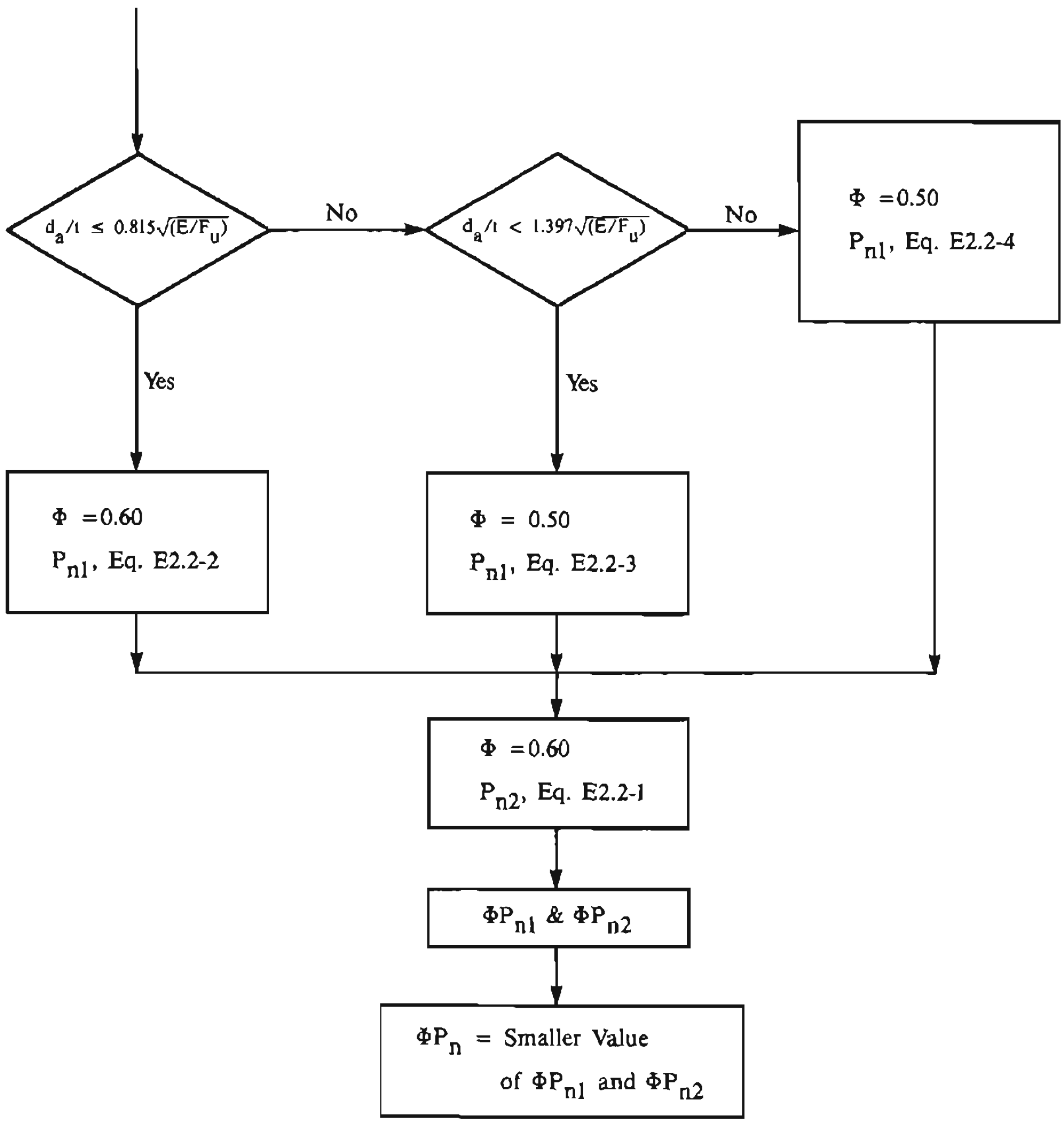
E2.2 Arc Spot Welds



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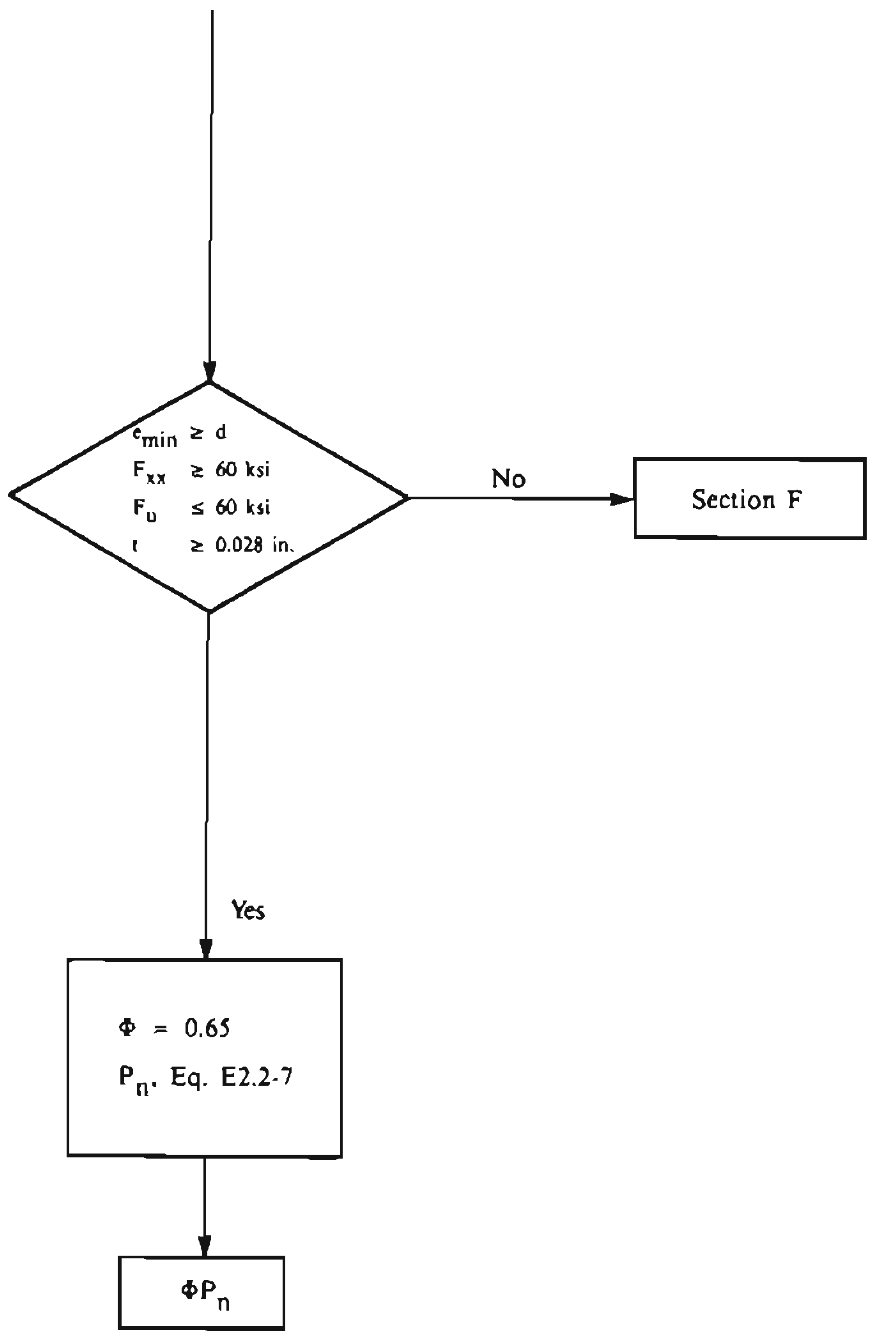
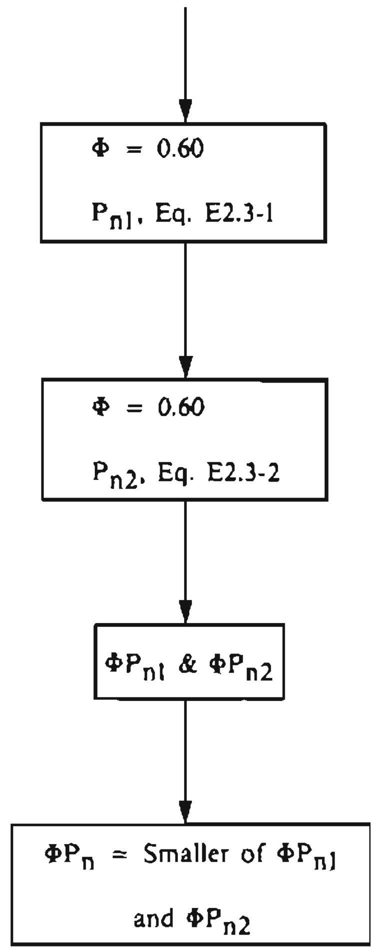
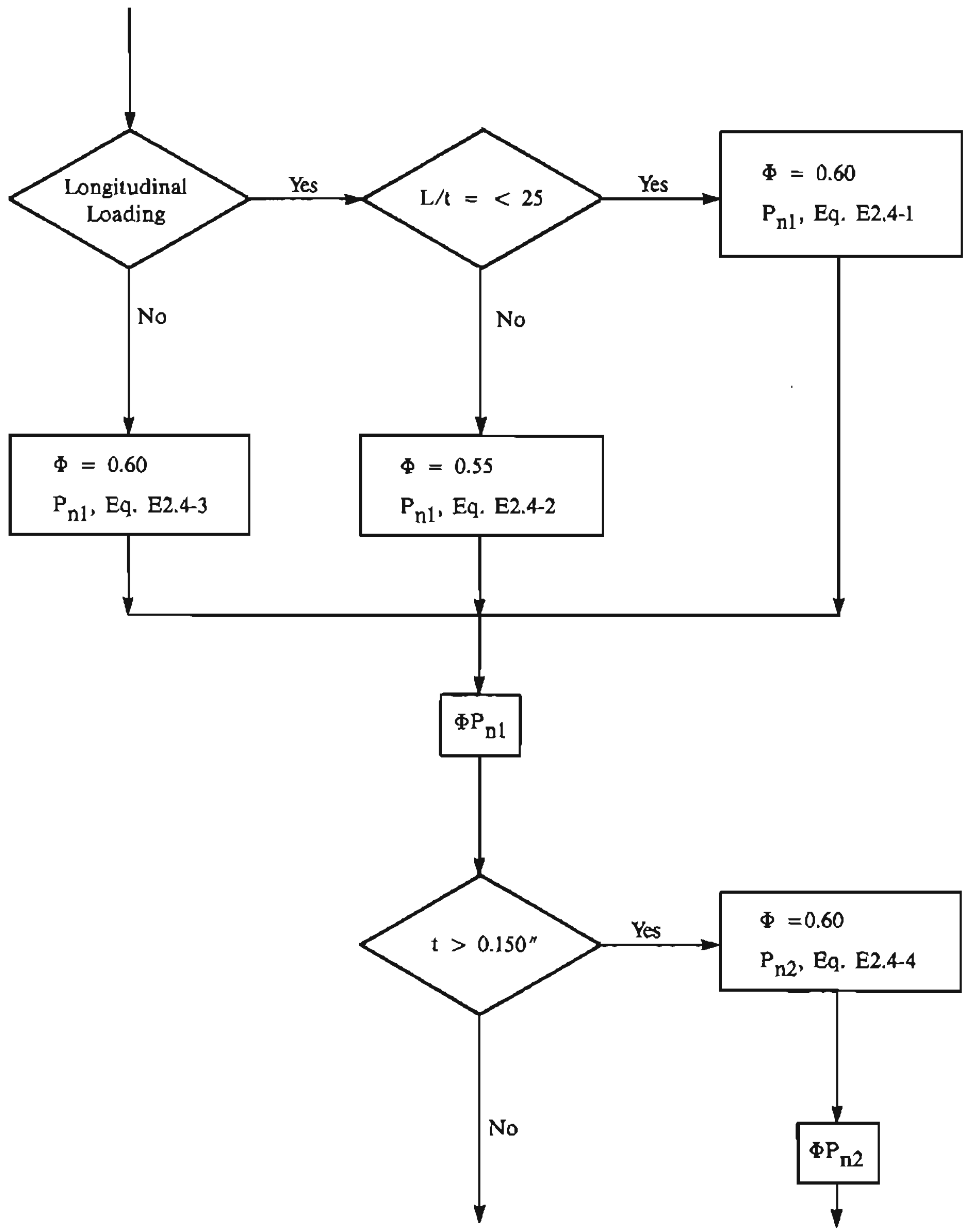
E2.2 Arc Spot Welds (continued)



E2.3 Arc Seam Welds



E2.4 Fillet Welds



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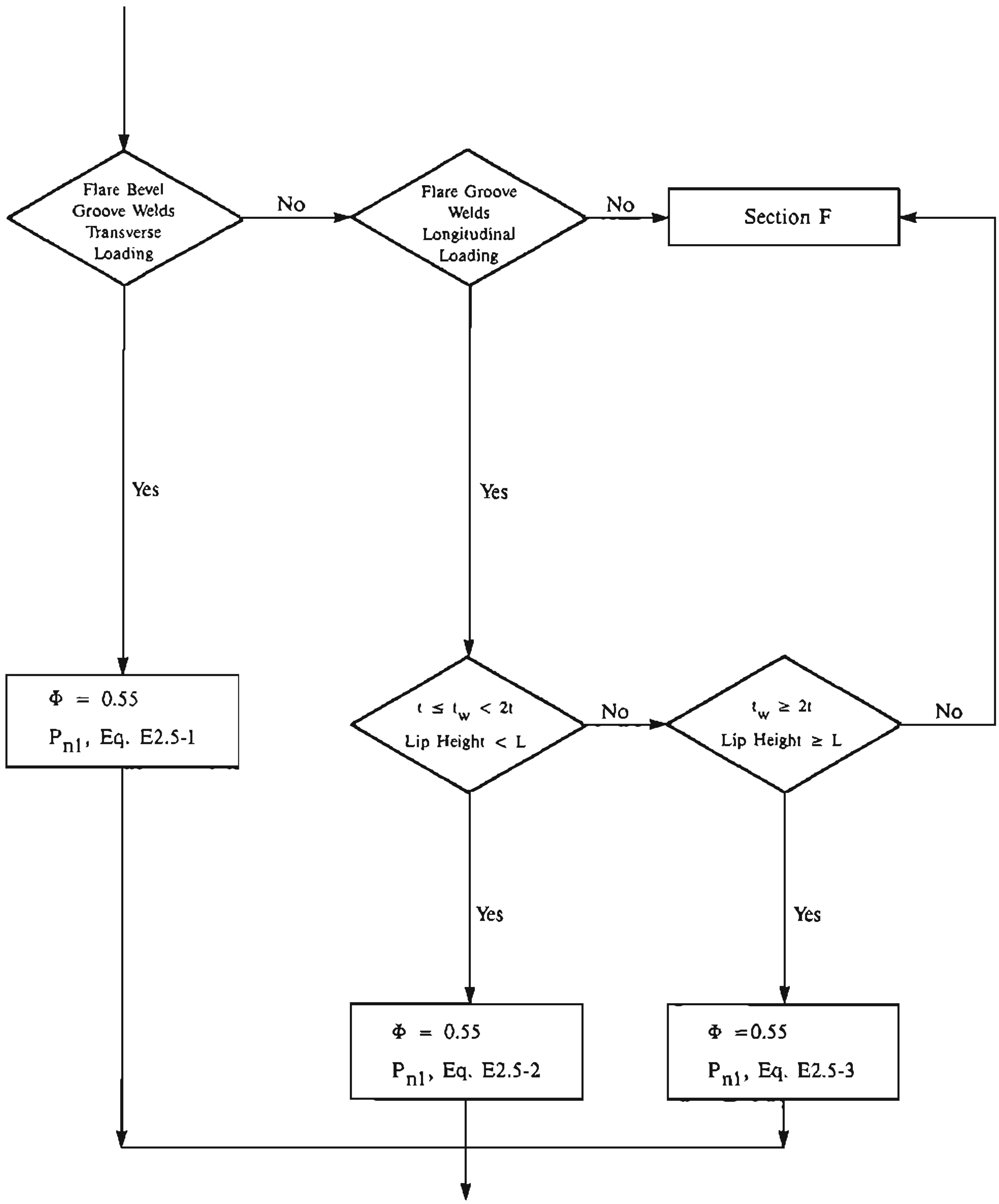
E2.4 Fillet Welds (continued)

$\Phi P_n = \Phi P_{n1}$

ΦP_n = Smaller of ΦP_{n1} and ΦP_{n2}

E2.5 Flare Groove Welds



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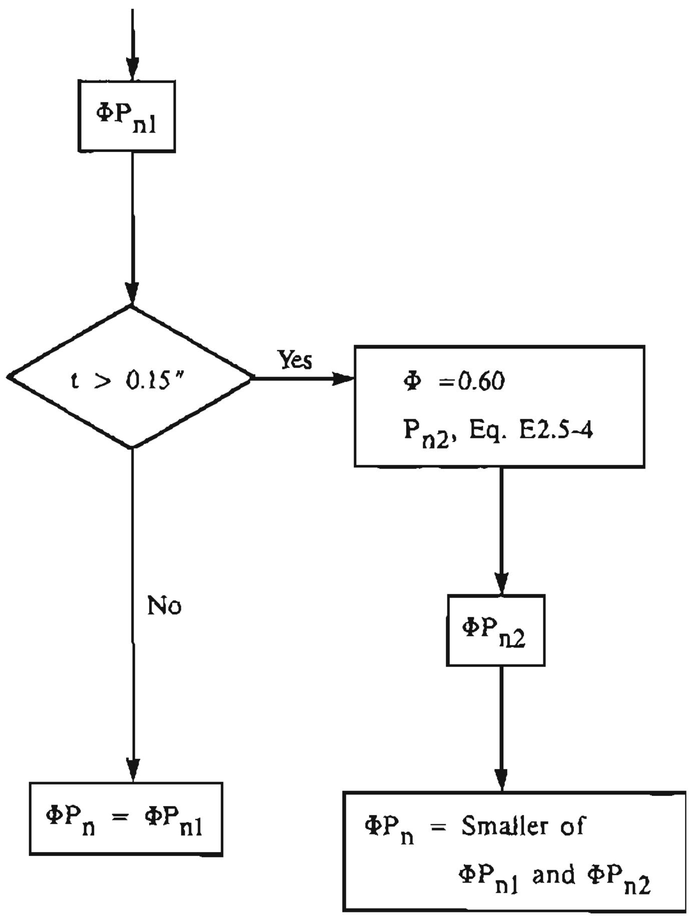
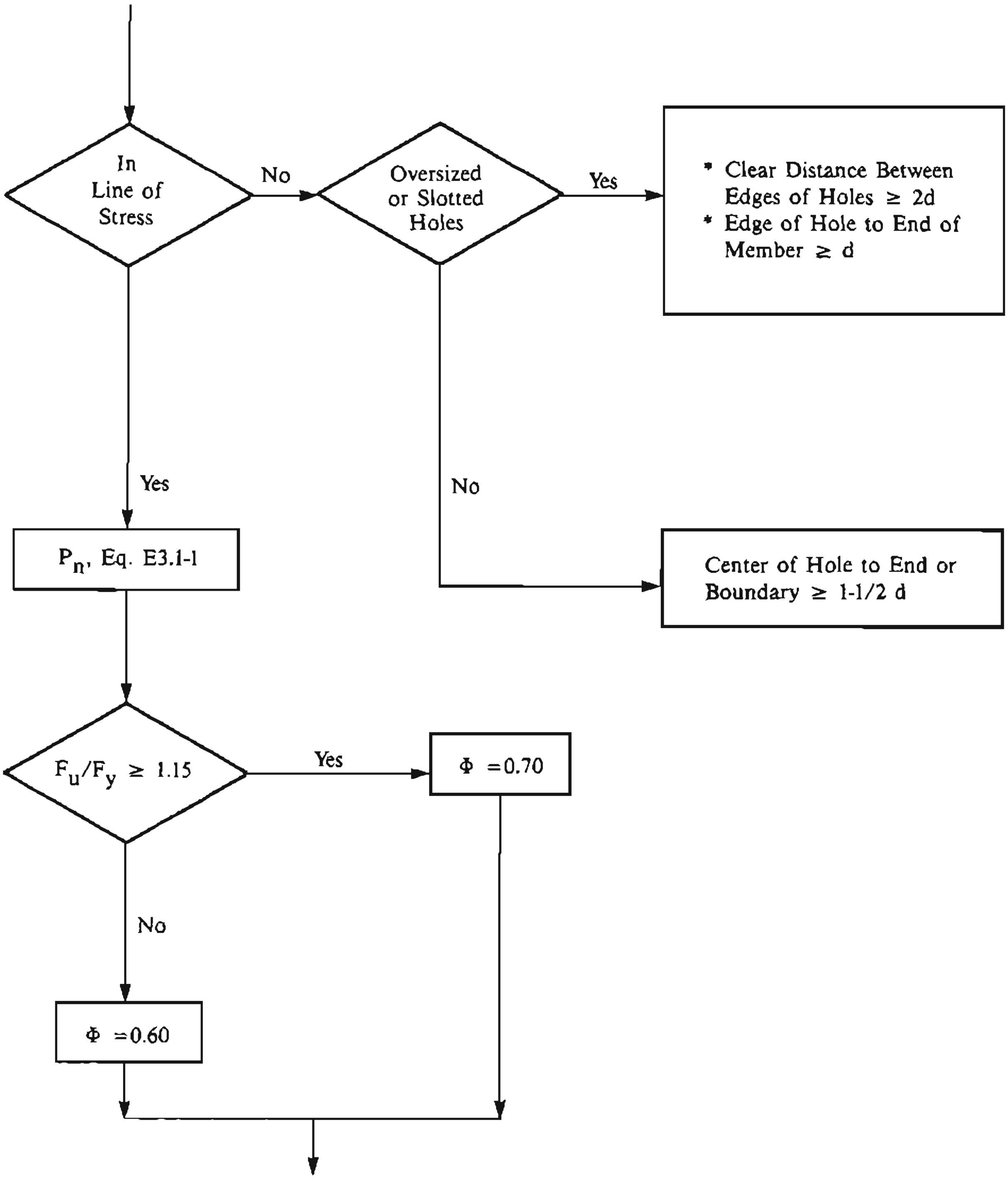
E2.5 Flare Groove Welds (continued)



E3.1 Spacing and Edge Distance



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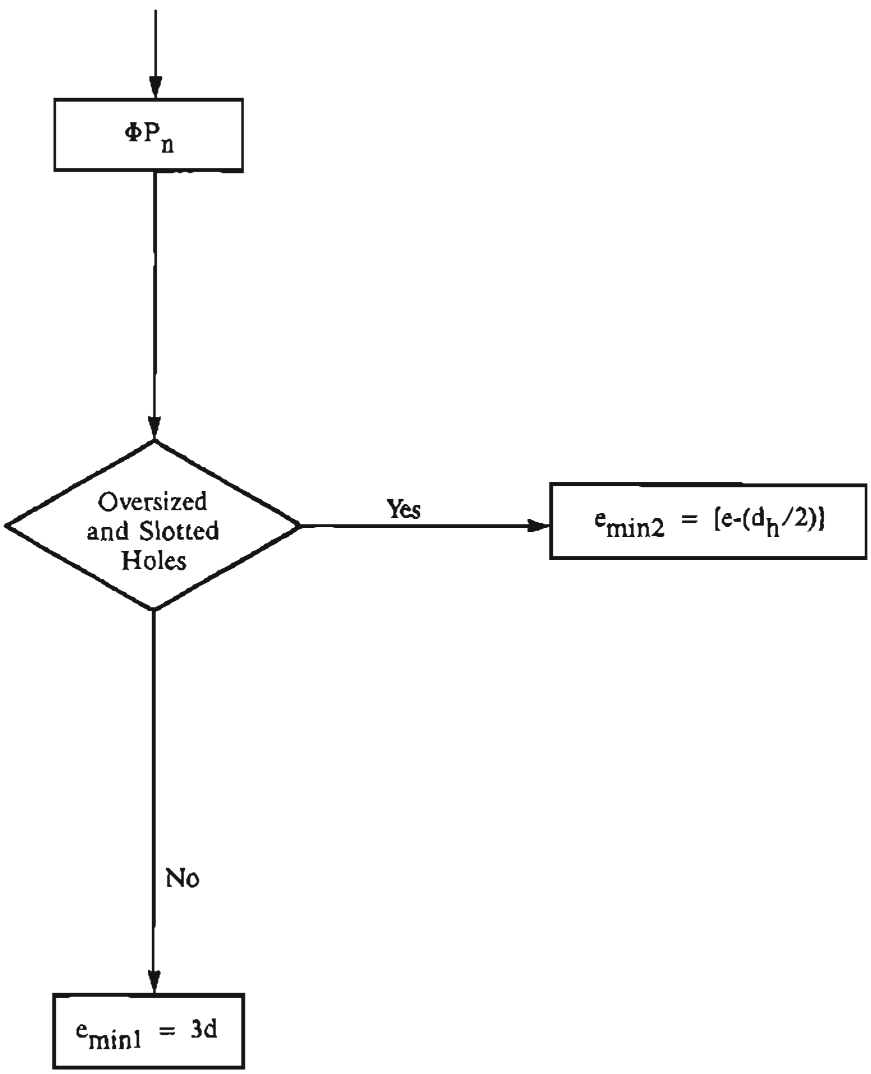
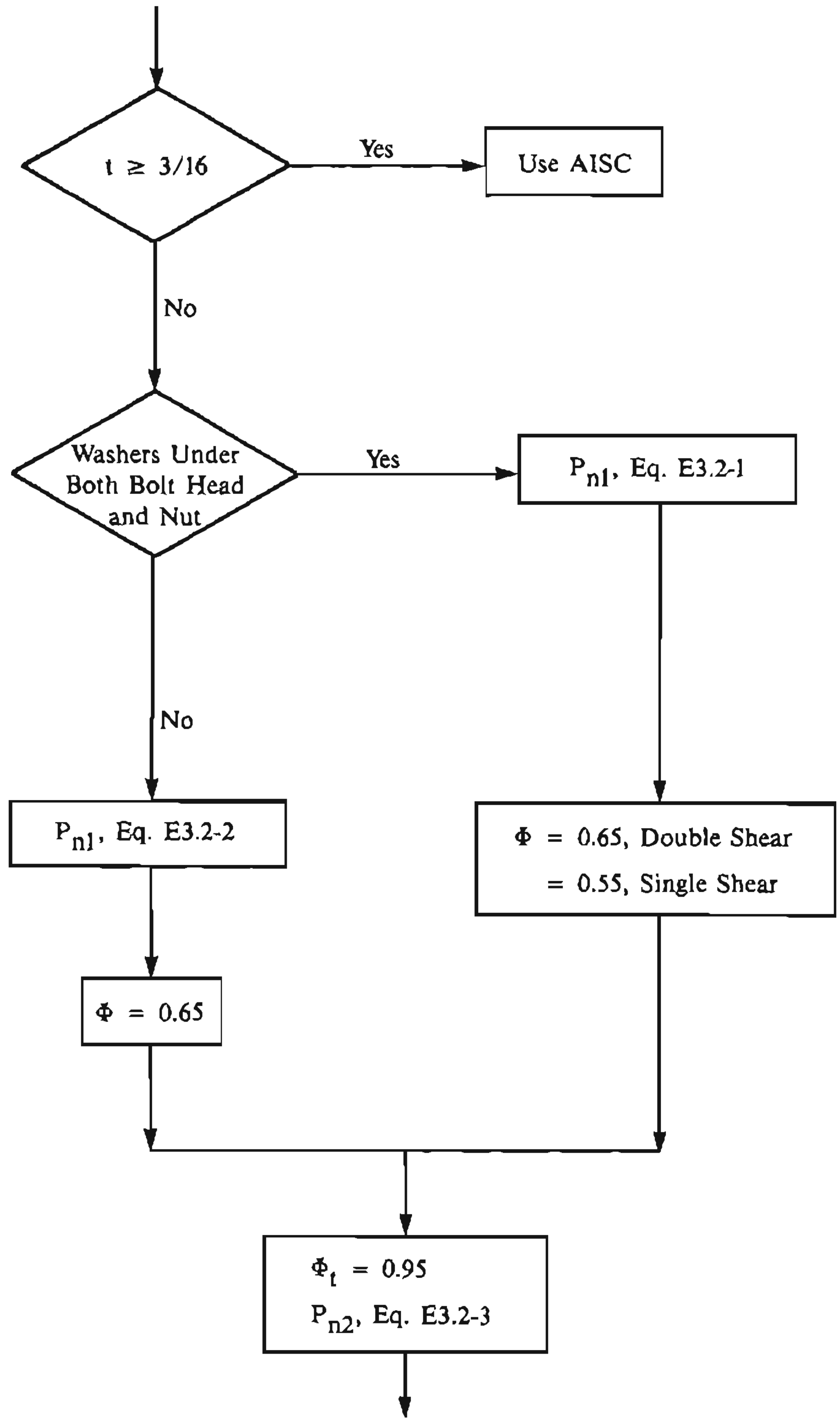
E3.1 Spacing and Edge Distance (continued)



E3.2 Tension in Connected Part



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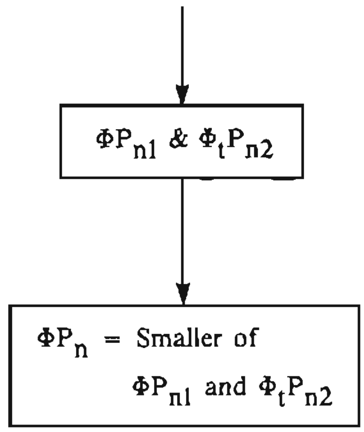
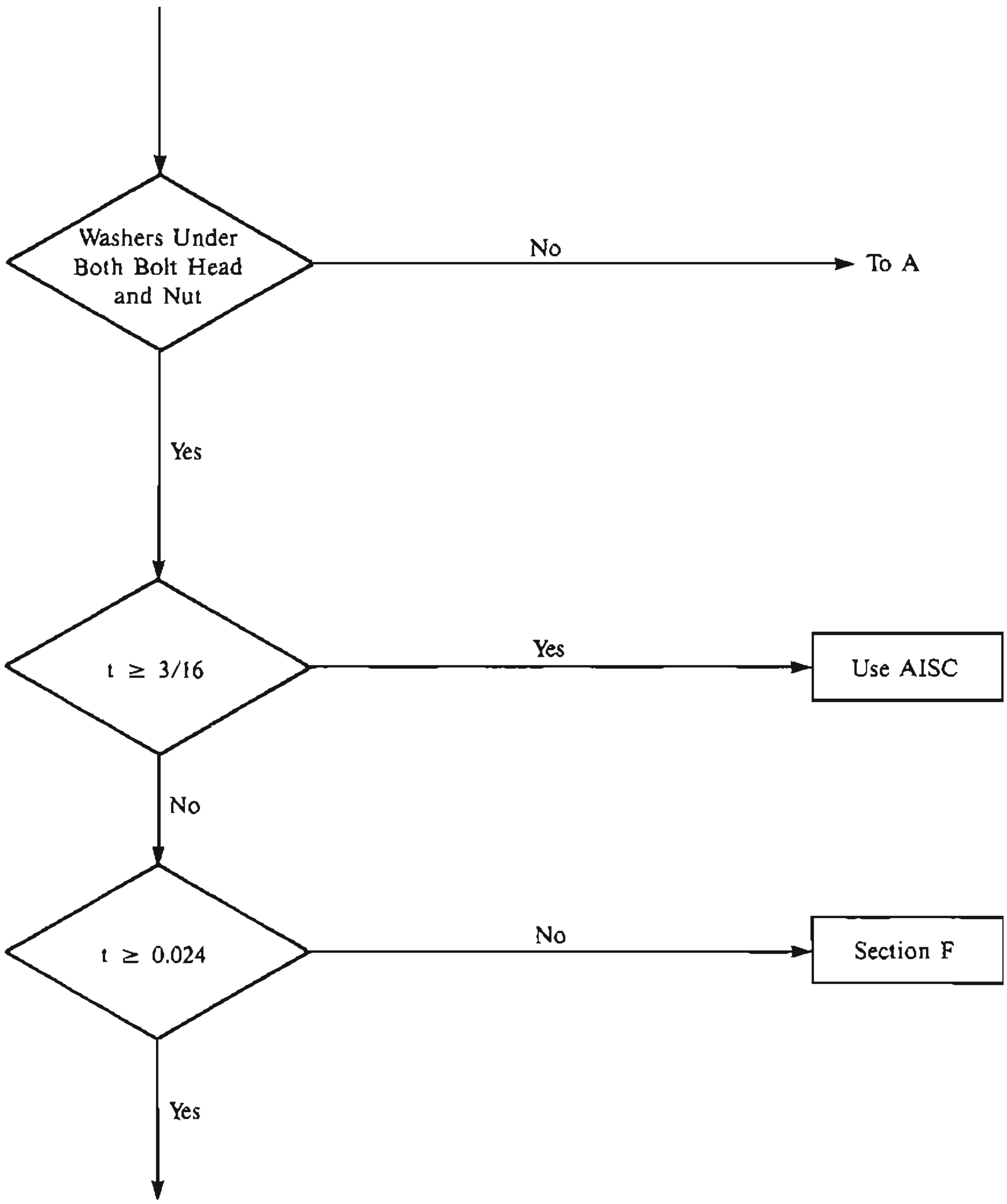
E3.2 Tension in Connected Part (continued)



E3.3 Bearing



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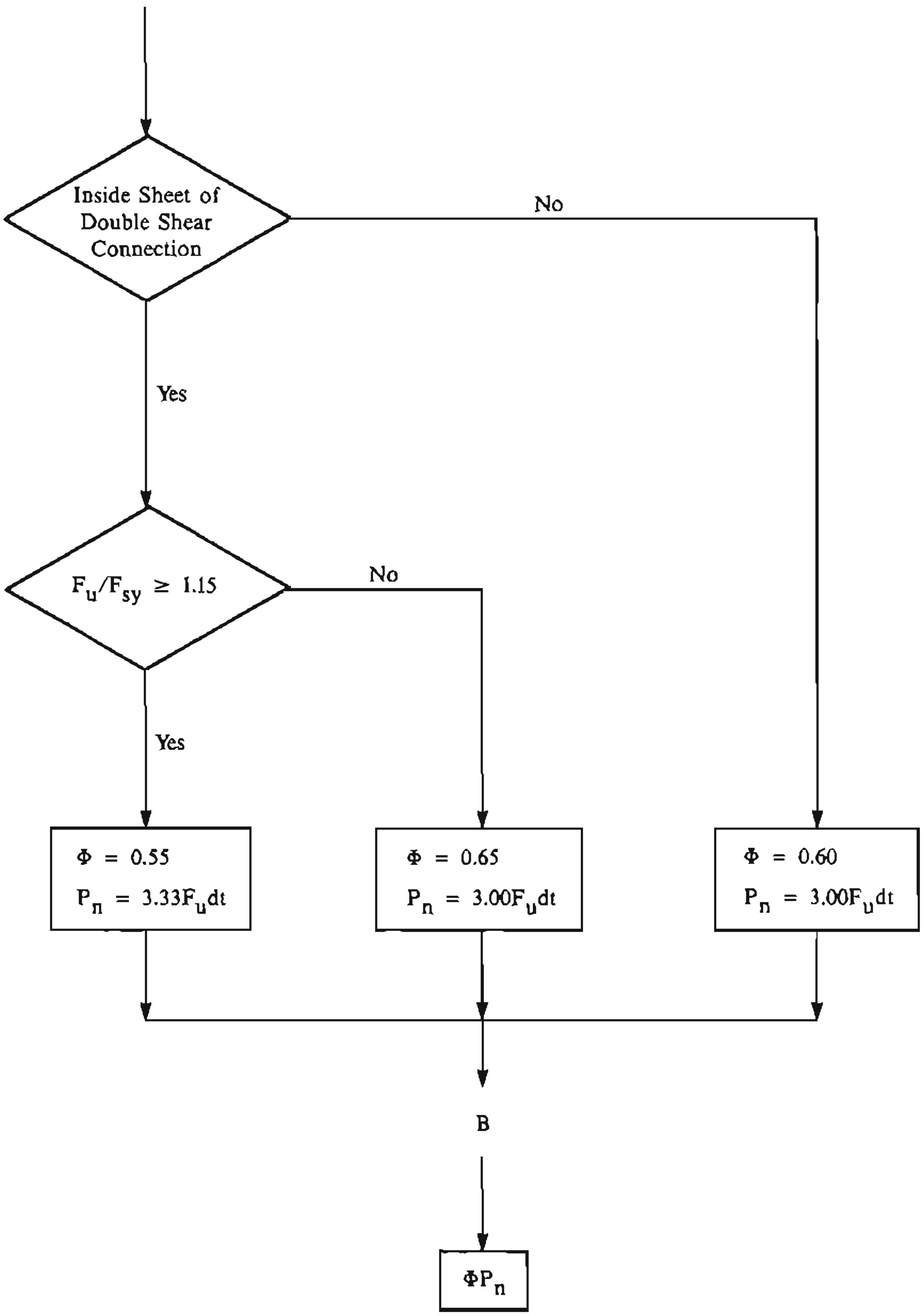
E3.3 Bearing (continued)



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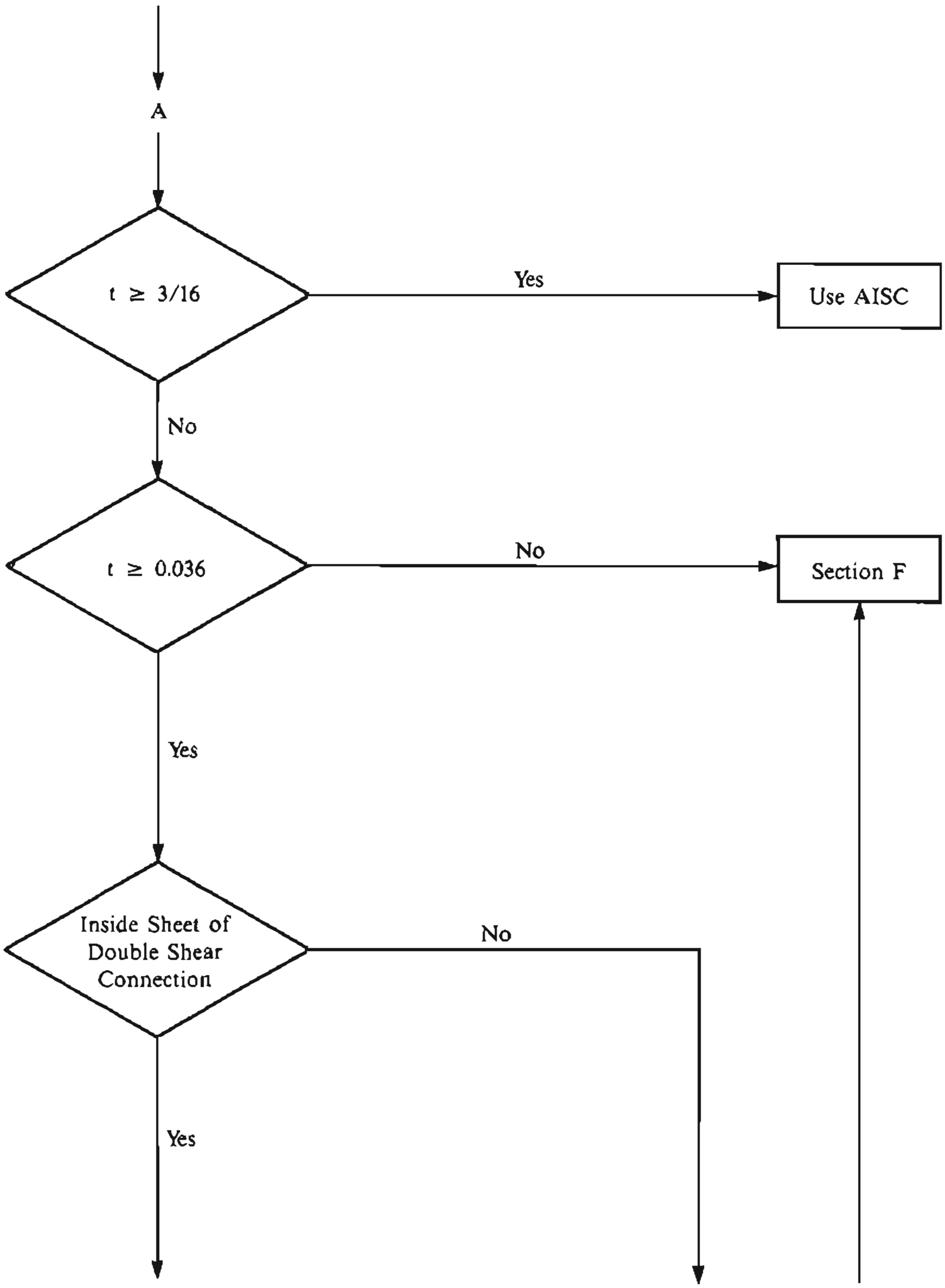
E3.3 Bearing (continued)



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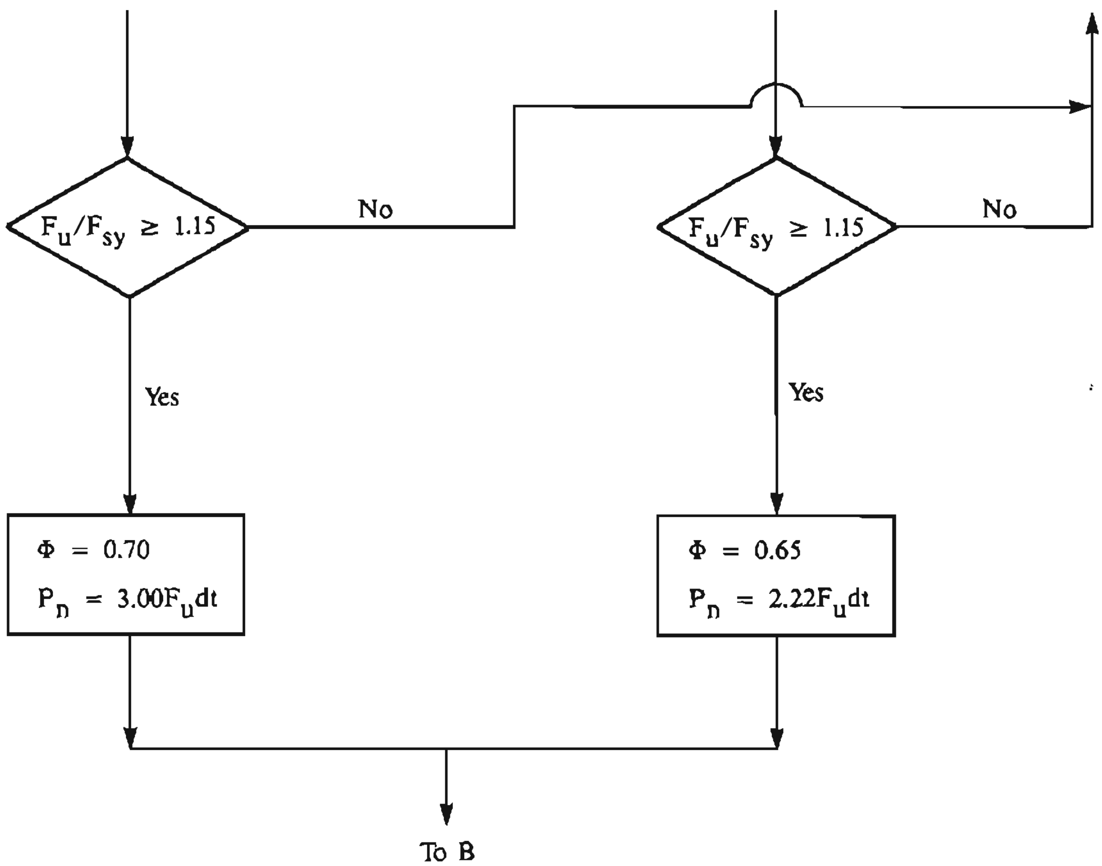
E3.3 Bearing (continued)



TEST PROCEDURES

TEST PROCEDURES

FOR USE WITH THE
MARCH 16, 1991 EDITION OF THE

LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

LRFD Cold-Formed Steel Design Manual-Part VII

AMERICAN IRON AND STEEL INSTITUTE
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PREFACE

This document, *Part VII of the LRFD Cold-Formed Steel Design Manual*, contains test procedures applicable to various provisions of the *Specification*. It consists of three parts: Test Method for Rotational-Lateral Stiffness of Beam-to-Panel Assemblies, Test Method for Stub Columns, and Standard Methods for Determination of Uniform and Local Ductility.

American Iron and Steel Institute
December 1991

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FOR USE WITH THE
MARCH 16, 1991, EDITION OF THE LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

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STANDARD METHODS FOR DETERMINATION OF UNIFORM AND LOCAL DUCTILITY

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ROTATIONAL-LATERAL STIFFNESS TEST METHOD FOR BEAM-TO-PANEL ASSEMBLIES

1. Scope

1.1 The purpose of this test is to determine the rotational-lateral stiffness of beam-to-panel assemblies. This test method is used primarily in determining the strength of beams connected to panels as part of a structural assembly.* The unattached "free" flange of the beam is restrained from lateral displacements and twisting by the bending stiffness of the beam elements, the connection between the "attached" flange of the beam and the panel, and the bending stiffness of the panel.

1.2 This test method applies to structural subassemblies consisting of panel, beam, and joint components, or of the joint between a wall, floor, ceiling, or roof panel and the supporting beam (purlin, girt, joist, stud).

1.3 This test method is also used to establish a limit of the displacements for avoiding joint failure.

1.4 The combined stiffness of the assembly determined by this method, K, consists of: (a) the lateral stiffness of the beam, K_a, which is a function of the geometry of the beam and geometric details of the beam-to-panel connection, (b) the local stiffness of the joint components in the immediate vicinity of the connection, K_b, which is affected by the type of fasteners, the fastener spacing used, and the geometry of the elements connected, and (c) the bending stiffness of the panel, K_c, which is a function of the moment of inertia of the panel, the beam spacing, and the beam location (edge vs interior). The latter stiffness shall be taken into account by theoretical analysis or by using the alternate test procedure described in Section 10.

1.5 For specific geometric conditions the design engineer may require duplicate testing using a new specimen with the beam orientation, or the force direction, reversed.

2. Description of Terms

2.1 *Subassembly*—A subassembly is a representative portion of a larger structural assembly consisting of a wall, floor, ceiling, or roof panel with one beam connected to the panel either continuously or at regular intervals (Figure 1).

2.2 *Panel*—The panel used in the subassembly may be made of any structural material, for example: aluminum, reinforced concrete, fiberboard, gypsum board, plastic, plywood, steel, etc. (Figure 1).

2.3 *Beam*—A beam may have an open or a closed cross section. One flange of the beam is connected to the panel, and is called the "attached" flange. The other is the "unattached" flange (Figure 1).

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* AISI *Specification*, Section C.3.1.3.

2.4 *Joint or Connection*—A joint or connection includes the local area around a mechanical fastener, weld, or adhesively bonded area that connects the beam with the panel. The local area also includes filler material such as insulation located between the panel and the beam flange.

2.5 *Lateral Load*—The total lateral load, P (in kips), is applied to the unattached flange of the beam (Figure 2) in a plane parallel to that of the original panel position.

2.6 *Lateral Deflection*—The lateral deflection (Figure 2) is the lateral displacement, D (in inches), of the unattached flange due to the lateral load, P.

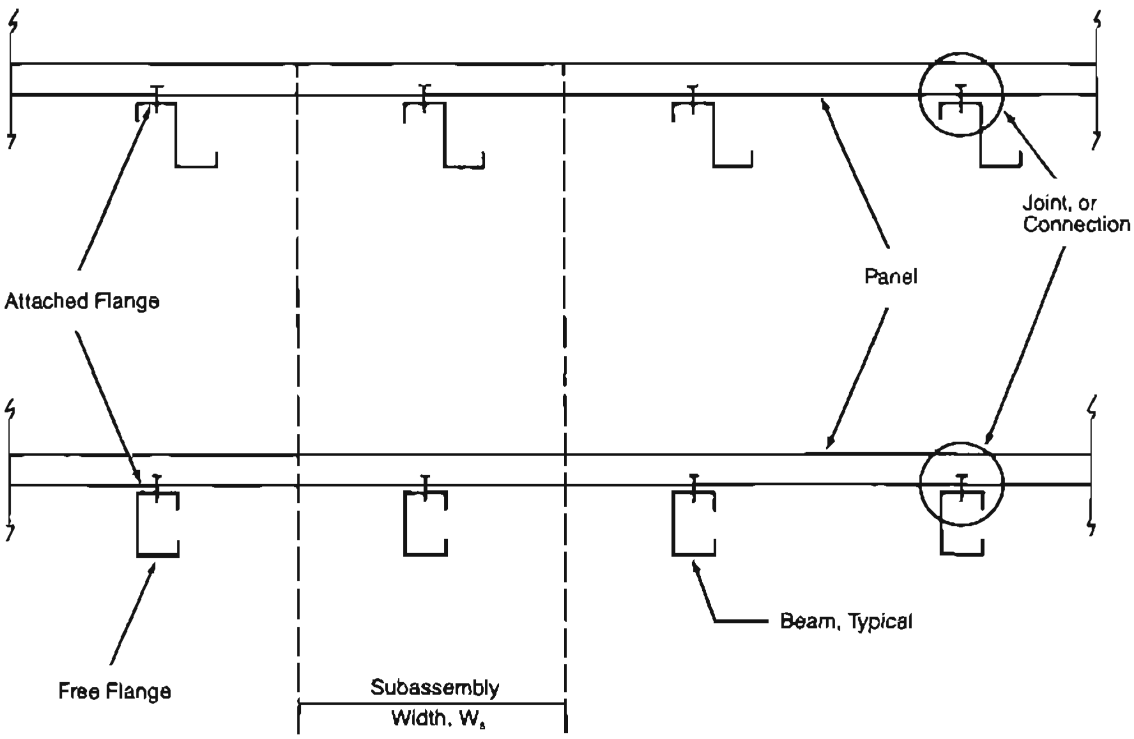
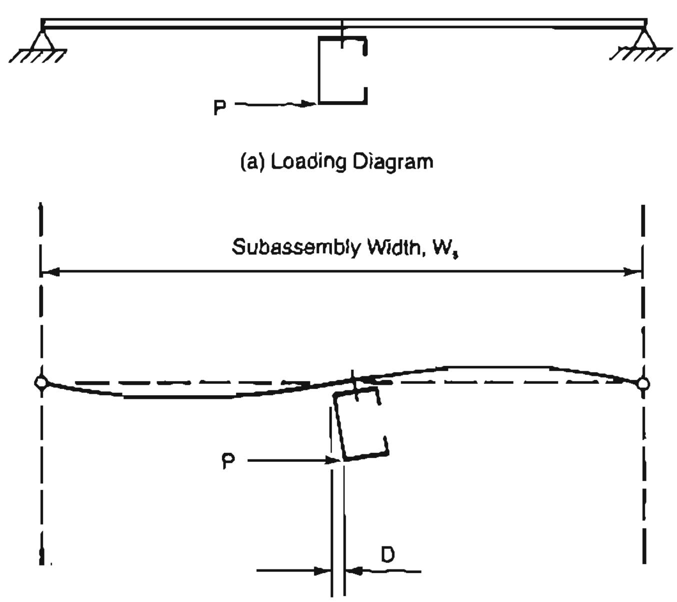


Figure 1 Wall, Floor, Ceiling, or Roof Assembly



(b) Deflected Subassembly

Figure 2 Loaded and Deflected Subassembly

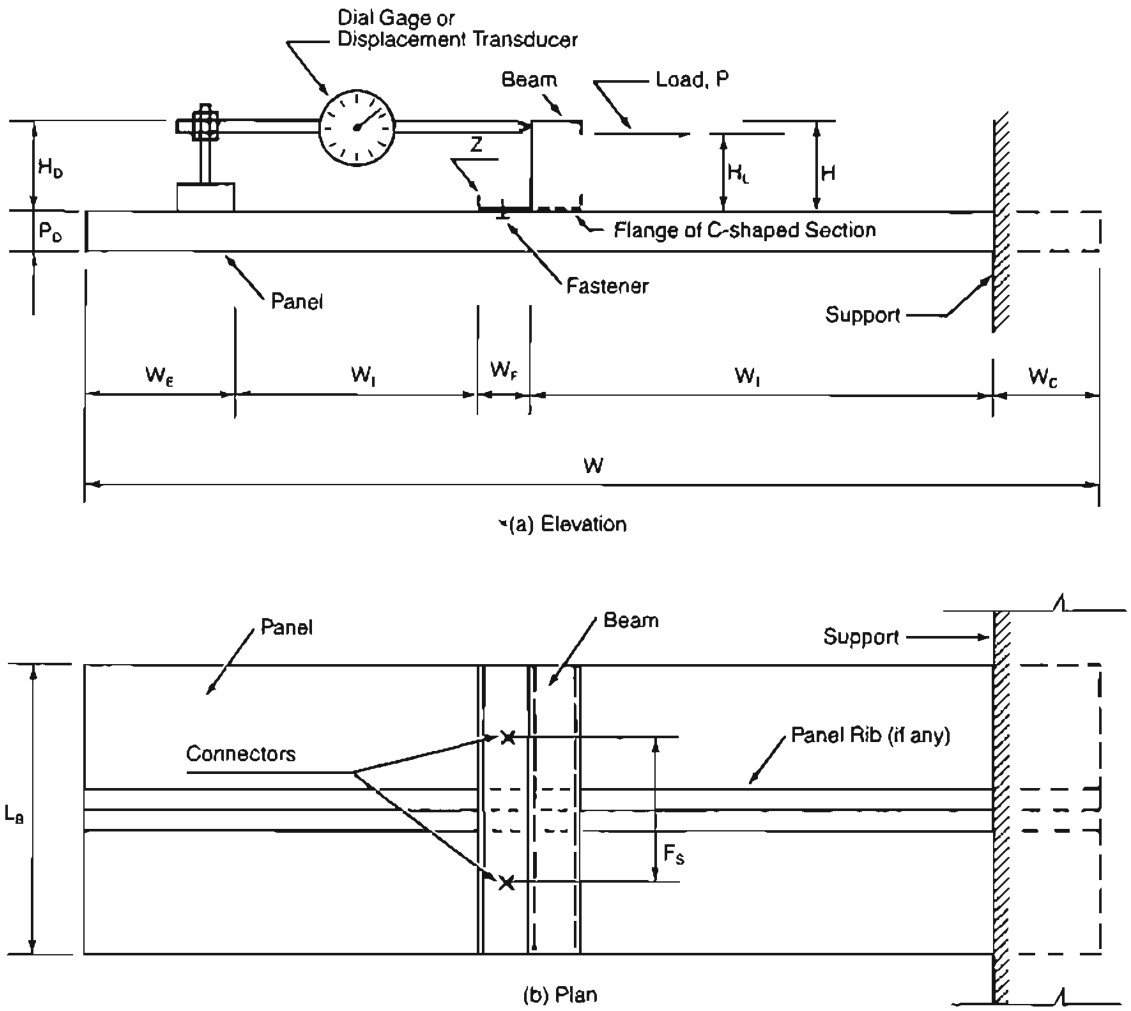


Figure 3 Test Specimen and Horizontal Test Setup

2.7 *Rotational-Lateral Stiffness*—The rotational-lateral stiffness, K, is equal to the total lateral load applied on the unattached flange of the test beam, divided by the length dimension of the beam, L_B (Figure 3b), and divided by the lateral deflection of the unattached flange of the beam at that load level. Thus, the units of K are: kips of lateral load per inch of beam length per inch of deflection, or k/in./in.

3. Materials

3.1 Components of the test specimen(s) shall be measured, and the component suppliers shall be identified.

3.2 Physical and material properties of the panel and beam shall be determined according to the latest edition of Specification ASTM E370 or other applicable standards.

4. Test Specimens

4.1 The overall panel width, W (Figure 3), of the specimen shall be such that the dial-gage support and the specimen support are each separated from the beam by a distance, W_I, not less than the largest of the following distances: (a) 1.5 times the overall panel depth P_D, (b) the overall width of the attached beam flange W_F, and (c) the fastener spacing along the flange of the beam, F_S. For ribbed panels, W_I shall also exceed two times the width of the attached flat of the panel.

4.2 The clamped width of the specimen, W_C, shall be at least equal to two times the panel depth, but not less than 2 inches.

4.3 The end dimension, W_E, shall be long enough to conveniently attach a dial gage or an extensometer to the end of the panel.

4.4 The minimum overall panel width shall be equal to:

$$W = W_E + 2W_1 + W_F + W_C \qquad (1)$$

4.5 The minimum beam and panel length, L_B, of the test specimen shall not be less than the largest of (a) two times the maximum connector spacing, F_S, used in actual field installations, or (b) the nominal coverage width of the panel. The specimen shall contain at least two fasteners in each line of connections along the beam.

4.6 Each specimen shall be assembled under the supervision of a representative of the testing laboratory, either at the manufacturer's facilities or at the testing laboratory.

4.7 Each specimen shall be assembled from new material; i.e., materials not used in previous test specimens, and in accordance with manufacturer's specifications.

4.8 The fabrication and field installation procedures specified for the overall assembly, and the tools used, shall also be used in the specimen construction as much as possible.

4.9 Drilled or punched pilot holes in the panels or beams shall be the same as those used in field installations.

5. Test Setup

5.1 The test specimens may be tested in a horizontal or vertical position (Figure 3 and Figure 4, respectively). The zero-load readings of the deflection-measuring device(s) shall be recorded.

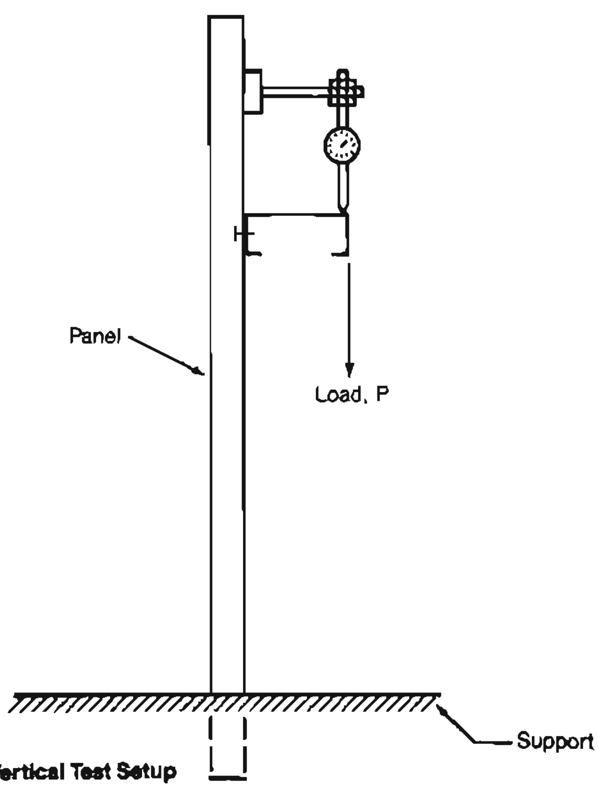


Figure 4 Vertical Test Setup

5.2 The clamped end of the panel shall be the only support of the test specimen.

5.3 When the test specimen panel is a hollow-core, corrugated, or trapezoidal panel, voids of the clamped regions shall be filled with filler materials such as wood, gypsum, or similar filler materials to ensure that the clamped overall depth of the panel is reasonably maintained. For foam-filled sandwich panels, if necessary, the filler material over the distance W_c may be replaced with wood, gypsum, or similar filler materials.

5.4 Loads applied to the unattached flange shall be introduced as close as possible to the extreme fiber of the beam, or at the intersection of the outer faces of the unattached flange and the web.

5.5 If the beam does not have a flat face perpendicular to the panel at the locations where the load is to be applied and the lateral displacement is to be measured, brackets are to be mechanically attached to the beam web to provide a flat surface. Figure 5 shows a typical application of a load bracket and/or dial gage bracket. The attachment of either bracket shall be accomplished such that the bracket does not stiffen the beam, or reduce its distortion.

5.6 The total lateral load applied, P, shall be distributed over several locations, if necessary, to reduce variations in the lateral deflection along the length of the unattached flange.

5.7 The load application shall be accomplished by chain or wire, and the necessary precautions shall be taken to ensure that the direction of the applied load remains essentially parallel to the original plane of the panel (Figure 5).

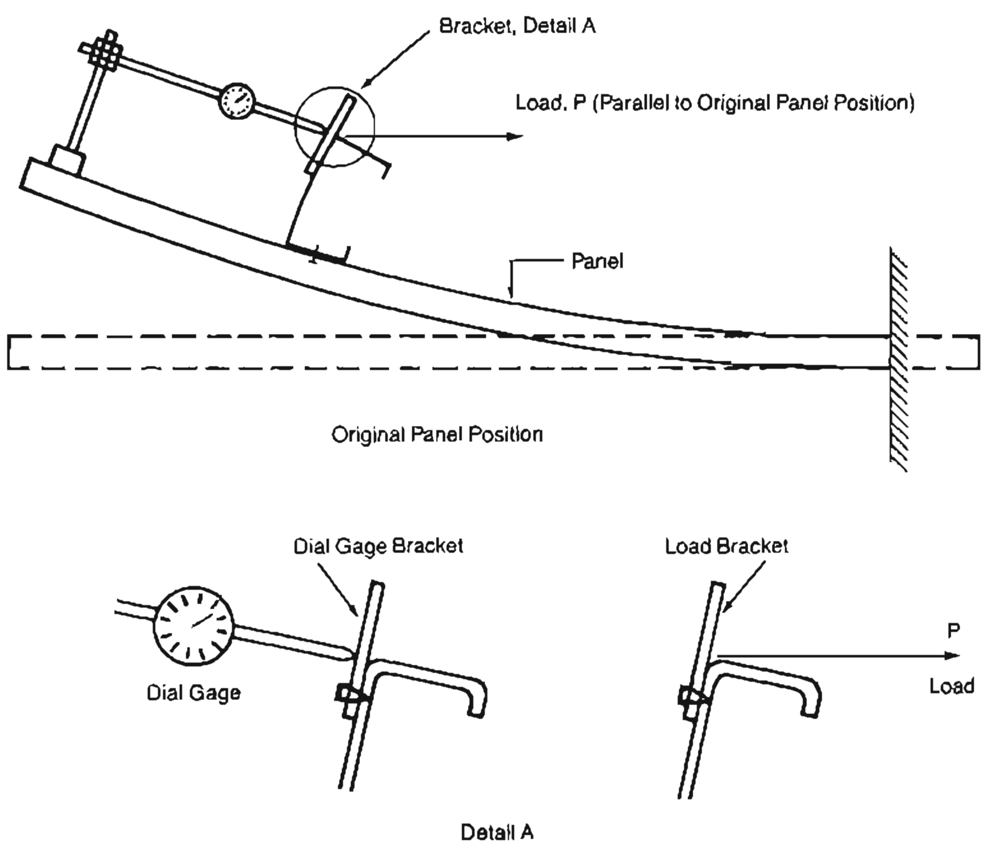


Figure 5 Dial Gage and Load Bracket

5.8 One or more dial gages or displacement transducers shall be used to measure the lateral displacements during loading. The gages shall be arranged symmetrically about the mid-width point, and have graduations at not greater than 0.001-inch intervals.

6. Test Procedures

6.1 The dial-gage height, H_D, and load height, H_L, as shown in Figure 3, shall be arranged such as to equal as close as possible the overall beam depth, H. Prior to loading the test specimen, the dimensions H_D and H_L, and the dial-gage readings shall be recorded.

6.2 No preload is to be used. The load shall be applied in a direction which is critical for the intended use of the results.

6.3 The applied load shall be increased in five or more equal increments to the maximum expected value, in order to produce deflection increments of not more than 5 percent of the beam depth.

6.4 If the specimen includes fiberglass insulation or other non-metallic elements in the joint between panel and beam, the load shall be held at each increment for 5 minutes before reading the lateral movement.

6.5 After each load increment is added, and the deflection has stabilized, the load and lateral movement of the unattached flange shall be measured and recorded.

6.6 A test shall be terminated at failure (fastener pullout, fastener failure, panel buckling, panel failure, beam failure, etc.) and the mode of failure recorded, unless the design engineer has determined that the application of the rotational-lateral stiffness, K, occurs at lower load or displacement levels and that the test may be terminated earlier.

7. Number of Tests

7.1 The minimum number of tests for one set of parameters shall be three. For parametric studies using multiple values of one or more parameters a smaller number of tests may be used.

7.2 If used as part of a series of at least three tests, one test is sufficient for a specific condition of an all-metallic mechanically-fastened specimen using the same basic components, but using unique geometrical or physical-property differences such as fastener spacings, different beam or panel yield strengths, etc.

7.3 Three tests are required for any specific condition of welded or adhesively-bonded specimens, or for specimens using non-metallic materials.

7.4 When the rotational-lateral stiffness for three or more panel or beam thicknesses with otherwise identical parameters is to be determined, at least two specimens each with the minimum and the maximum thickness shall be tested. For a ratio of maximum-to-minimum thicknesses greater than 2.5, additional specimens with intermediate thicknesses must be tested. One test of every thickness may be used in accordance with Section 7.2.

7.5 When the rotational-lateral stiffness for a range of screw spacings is to be determined, the minimum number of specimens shall be as follows: For a ratio of maximum-to-minimum screw spacings equal to or less than 2, at least two specimens each with the minimum and the maximum screw spacing shall be tested. For a range of five or more different screw spacings, or for a ratio of maximum-to-minimum screw spacings greater than 2, additional specimens with intermediate spacings must be tested. One test of every screw spacing may be used in accordance with Section 7.2

7.6 Where the rotational-lateral stiffness for a range of other panel parameters—such as yield or ultimate strength, changes in geometry, etc.—are to be determined, a number of tests similar to the requirements under Sections 7.2 through 7.5 shall be performed.

7.7 For unsymmetric or staggered fastener arrays and/or beams unsymmetric about a plane parallel to the web, duplicate tests may be required by the design engineer using new specimens with the beam orientation, or the force direction, reversed.

8. Test Evaluation Procedure

8.1 Typical load-displacement curves (P vs. D) obtained from the tests are as shown in Figure 6. For multiple tests of one set of test parameters, the curve resulting in the lowest value of K_t, as defined in Section 8.2, shall be used for the test evaluation procedure.*

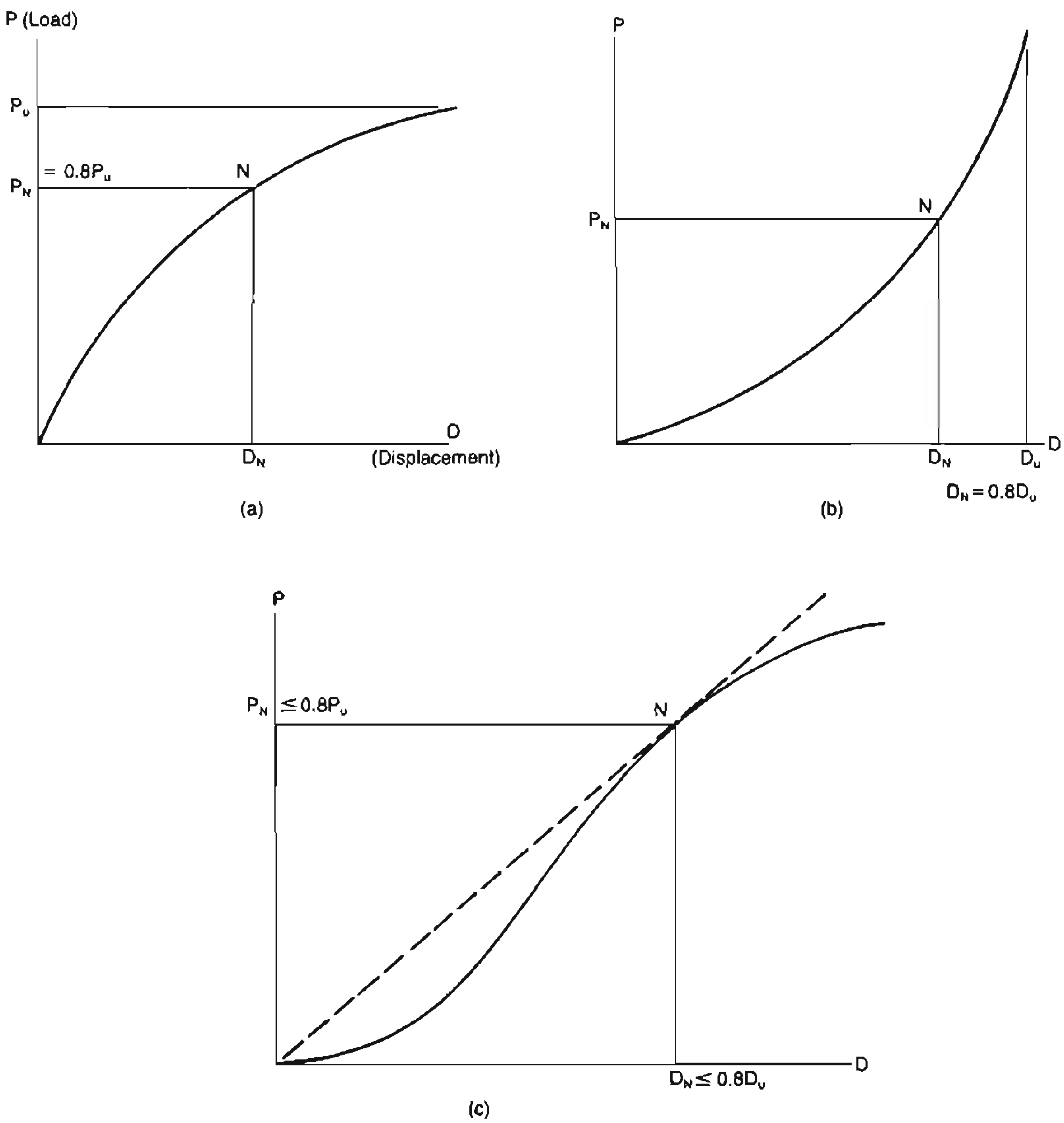


Figure 6 Typical Load-displacement Curves

*The test stiffness, K_t, includes the stiffness effects of the beam, K_a, and the beam-to-panel connection, K_b, but excludes the effects of the bending stiffness of the panel, K_c, and follows the relationship $K_t = (1/K_a + 1/K_b)^{-1}$.

8.2 The test stiffness, K_t, at any load level is determined by

$$K_t = P/D/L_B \tag{2}$$

8.3 The nominal test stiffness. K_N, shall be determined by

$$K_N = P_N/D_N/L_B \tag{3}$$

where P_N and D_N shall be determined for a point, N, such that either P_N shall be equal to 0.8 times the ultimate load, P_u, for load-displacement curves as shown in Figure 6(a), or the displacement D_N shall be equal to 0.8 times the ultimate displacement, D_u, for load-displacement curves as shown in Figure 6(b), or by a tangent drawn from the origin to the P-D curve as shown in Figure 6(c), resulting in $P_N \le 0.8P_u$ and $D_N \le 0.8D_u$.

8.4 When the design engineer specifies in advance a desired maximum lateral displacement limit of D_{NL}, the test may be discontinued when D_{NL} is reached, and K_N may be determined from P_N at D_{NL}, as long as the limits under Section 8.3 are observed and D_{NL} is not exceeded in actual design applications.

8.5 Where either H_D or H_L are not equal to the overall beam height, H, K_t and K_N shall be corrected by the factor H_DH_L/H^2.

8.6 In addition, K_t and K_N shall be adjusted by the stiffness contributions of the panel, K_c, derived from the linear-elastic displacement analysis representing the actual design applications, unless such an analysis shows that these contributions are insignificant. Alternately, the panel stiffness may be included by using the alternate test method under Section 10.

8.7 For subassemblies such as shown in Figure 2, the applied lateral test loads cause a bending moment distribution in the panel similar to that shown in Figure 7, and a lateral displacement of the unattached flange of the beam, D_c, equal to

$$D_c = PH_L^2W_S/(12EI) \tag{4}$$

where W_S is the width of the subassembly (Figure 2 and Figure 7), E is the modulus of elasticity of the panel material, and I is the effective moment of inertia of the panel cross section (obtained from deflection determination calculations for cold-formed metal deck panels).
The panel stiffness is equal to

$$K_c = 1/D_c \tag{5}$$

8.8 The overall rotational-lateral stiffness of the subassembly shall be determined as

$$K = (1/K_t + 1/K_c)^{-1} \tag{7}$$

8.9 When tests covering ranges of parameters (thickness, yield strengths, screw spacings, etc.) are conducted according to Section 7, a linear interpolation may be used to determine intermediate K values.

9. Test Report

9.1 The test report shall consist of a description of all specimen components, including drawings defining the actual and nominal geometry, material specifications, material properties test results describing the actual physical properties of each component, and the sources of supply. Differences between the actual and the nominal dimensions and material properties shall be noted in the report.

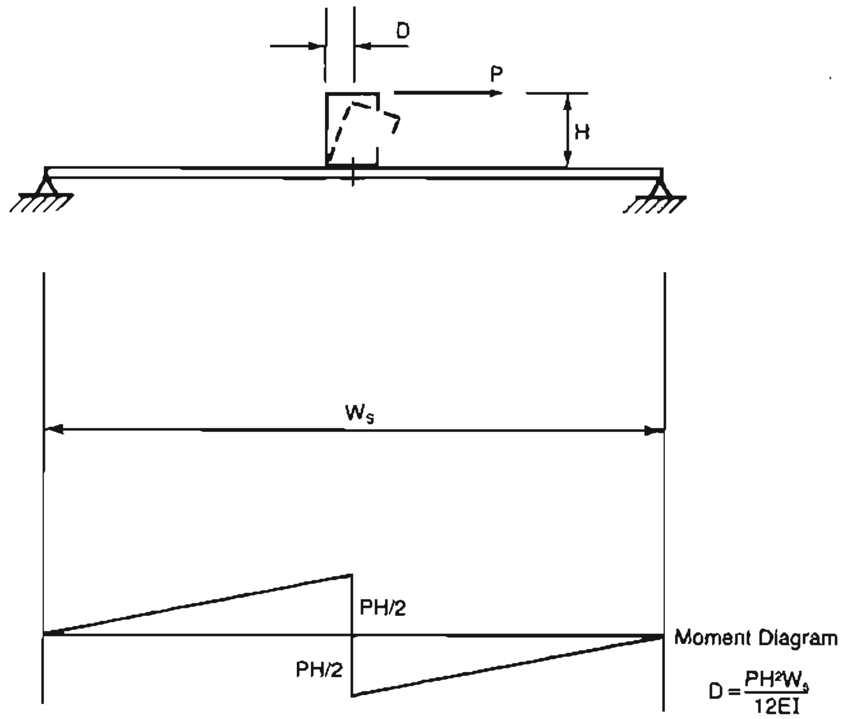


Figure 7 Bending Moment Diagram for Panel with an Interior Beam

9.2 In addition, the test report shall contain a sketch or photograph of the test setup, the latest calibration date and accuracy of the equipment used, the signature of the person responsible for the tests, and a tabulation of all raw and evaluated test data.

9.3 All graphs resulting from the test evaluation procedure shall be included in the test report.

9.4 A summary statement, or tabulation, shall be included in the summary of the report to define the actual and nominal rotational-lateral stiffness derived from the tests conducted, including all limitations.

10. Alternate Rotational-Lateral Stiffness Test*

10.1 To include the panel-stiffness contribution in the test, rather than by linear-elastic analysis, the design engineer may request a test specimen and setup as shown in Figure 8 and Figure 9, respectively.

10.2 The test specimens shall be as described under Section 4 except as follows.

10.2.1 The minimum overall panel width of the specimen, W (Figure 8), shall be

$$W = W_E + W_1 + W_C \tag{6}$$

10.2.2 The minimum end dimension, W_E, shall equal the width of the attached beam flange plus 4 inches to allow the development of local deformation patterns around the fasteners as they would develop in a real structure.

*This method is conservative as compared to the basic methods which analytically account for the stiffness of the panel.

10.2.3 For specimens representing interior-beam subassemblies, as shown in Figures 1 and 2, the dimension W_I of the test specimen (Figure 8) shall be equal to 1/12 of the subassembly width, W_S (Figures 1 and 2), to assure that the overall rotational-lateral stiffness contribution of the test-specimen panel is the same as that of the subassembly.

10.2.4 For other subassembly conditions, W_I shall be determined to represent the actual conditions.

10.3 The test-setup shall be as described under Section 5 except as follows.

10.3.1 The clamped support as shown in Figures 8 and 9 shall be sufficiently rigid to minimize the rotation and translation of the test specimen at the support.

10.3.2 The lateral-displacement measuring device shall be located on a support fixed relative to the clamped support of the test panel, as shown in Figure 9.

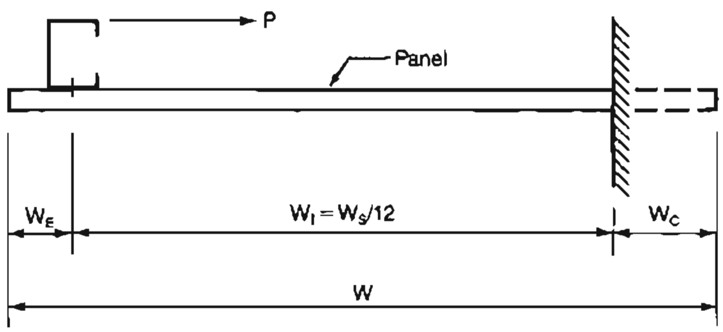


Figure 8 Panel Width for Alternate Test Procedure

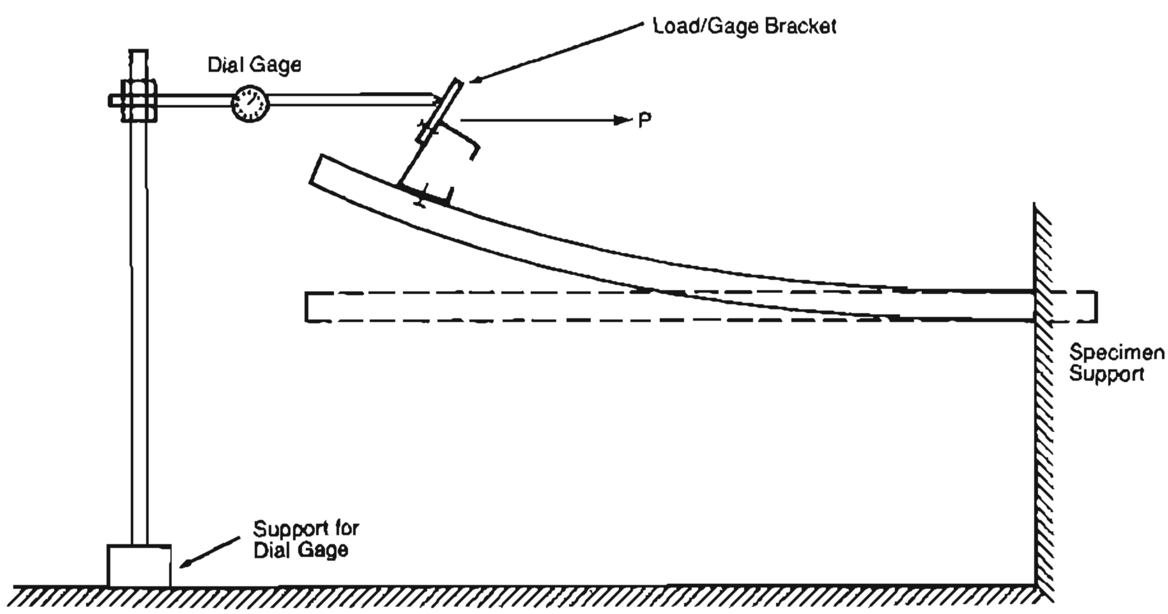


Figure 9 Test Setup For Alternate Test

10.4 Test procedures shall be the same as described under Section 6.

10.5 The number of tests shall be determined as described in Section 7.

10.6 The test-evaluation procedure shall follow the underlying principles used to develop Section 8. The test stiffness at any load level shall be determined according to Equation 2 and the nominal test stiffness shall be determined according to Equation 3. No further adjustments are needed.

10.7 For other interior-beam spacings, for exterior-beam conditions, or for other geometrical conditions, the measured displacements shall be adjusted by a linear-elastic analysis to represent the actual field conditions, unless such an analysis shows that these displacements and their effect on K are insignificant.

STUB-COLUMN TEST METHOD[1] FOR EFFECTIVE AREA OF COLD-FORMED STEEL COLUMNS

1. Scope

1.1 This test method covers the determination of the effective cross-sectional area of cold-formed steel columns. It primarily considers the effects of local buckling and residual stresses and applies to solid or perforated columns that have holes (or hole patterns) in the flat and/or curved elements of the cross section (1).[2]

1.2 The effective area is used to determine the allowable axial loads of cold-formed column sections in accordance with the *AISI Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members*, hereafter called AISI *Specification*.

1.3 The effective area is a variable section property of columns. It reflects the effects of local buckling in relatively thin area elements caused by axial stresses, or loads. When the axial load is zero, the effective area is equal to the gross cross-sectional area; however, when an axial load is applied, the effective area may be less than the gross area. In such a case, the effective area will reduce with increasing load.

1.4 Local buckling reduces the axial load-carrying capacity that would otherwise be limited only by general yielding or overall column buckling. The amount of the reduction depends on the width-to-thickness ratio of the flat elements of the column cross section, the yield strength of the steel sheet from which the column is formed, and the size and frequency of holes or hole patterns, if present.

2. Applicable Documents

2.1 ASTM Standards:
A370—Tensile Test Method For Steel Sheets
E4—Verification of Testing Machines

2.2 *AISI Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members, 1991 Edition.*

[1]This test and evaluation method will be proposed to the appropriate ASTM Committee for review and adoption.
[2]Numbers in parentheses refer to references at the end of this test method.

3. Terminology

3.1 ASTM Definitions Standards:
E6—Definitions of Terms Relating to Methods of Mechanical Testing.
E380—Standard for Metric Practice.

3.2 Description of terms specific to this standard:
Elements = Straight or curved portions of the cross section of a column or stub column.
Local Buckling = The local buckling mode of a flat element of a column cross section, which influences the overall column-buckling behavior.
Overall Buckling = Buckling of a column as a function of its overall length.
Stub-Column = An axial compression member of the same cross section and material as the column for which the strength needs to be determined, but of sufficiently short length to preclude overall column buckling, if possible.

3.3 Symbols:

A = the gross cross-sectional area of a column without holes or perforations, or the minimum gross cross-sectional area of a column with holes or perforations.

A_a = the average of all gross cross-sectional areas of the stub columns in a test unit, or the average of gross cross-sectional areas of a stub column.

A_e = the effective cross-sectional area of a stub column at a load less than the ultimate test load, or the effective area of a full-length column.

A_{ei} = the effective cross-sectional area of a stub column at load P_i.

A_{eu} = the nominal effective cross-sectional area at ultimate load adjusted to the nominal thickness and the minimum specified yield strength.

A_{eua} = the average effective cross-sectional area of a test unit of stub columns at the ultimate axial load.

A_{eu1} = the effective cross-sectional area of a stub column with parameters of Test Unit 1 at ultimate load.

A_{eu2} = the effective cross-sectional area of a stub column with parameters of Test Unit 2 at ultimate load.

A_1 = the minimum gross cross-sectional area of a stub column with parameters of Test Unit 1 at ultimate load.

A_2 = the minimum gross cross-sectional area of a stub column with parameters of Test Unit 2 at ultimate load.

D = the axial shortening of a stub column at load P.

D_i = the axial shortening of a stub column at load P_i.

D_u = the axial shortening of a stub column at load P_u.

f = the average axial stress assumed to be uniformly distributed over the effective cross-sectional area, A_e.

f_i = the average axial stress assumed to be uniformly distributed over the effective cross-sectional area, A_{ei} at load P_i.

f_o = the average axial stress assumed to be uniformly distributed over the effective cross-sectional area, A_e, above which the section is not fully effective.

F_n = the nominal ultimate stress, assumed to be uniformly distributed over the effective cross section of a column as calculated from Section C4 of the AISI *Specification*, at which flexural, torsional, torsional-flexural, or local buckling, and/or yielding, may occur.

F_u = the ultimate stress, assumed to be uniformly distributed, at which local failure occurs in a tested stub column.

F_y = the minimum specified elastic limit or yield stress of column or stub-column material.

F_{ya} = the average elastic limit or yield stress of the sheet steel for a given test unit.

F_{yi} = the individual elastic limit or yield stress of the sheet-steel specimens in a test unit.

i = load-displacement-reading number for a particular stub-column test (load displacement D_i at load P_i).

j = total number of load-displacement readings taken for a particular stub-column test.

L = the length of the stub-column test specimen.

Lp = the pitch of a repeating pattern of perforations along the longitudinal column axis.

n = the ratio of the effective cross-sectional area at the ultimate load to the full cross-sectional area, A_{eu}/A.

P = the applied axial compression force (column load).

P_i = the applied load at load-increment i.

P_n = the nominal failure load of a column.

P_u = the ultimate stub-column load at which local failure occurs.

P_{ua} = the average of all ultimate stub-column loads within a test unit.

r = the minimum radius of gyration of the cross-sectional area, A.

t = the nominal base-steel thickness exclusive of coating.

t_a = the average of all base-steel thicknesses within a test unit, exclusive of coating.

W = the greatest overall width of the cross section including corner(s).

4. Significance

4.1 This test method provides requirements for testing, and equations to determine, the effective area of a cold-formed column section at ultimate load, A_{eu}, and the load- or stress-dependent effective area, A_e. These properties are used in the AISI *Specification* to determine the ultimate and less-than-ultimate column strengths. The ultimate column strength, P_u, is the product of the minimum specified yield stress, F_y, or the buckling stress F_n, and the corresponding effective cross-sectional area at that stress, A_{eu}. At an applied column strength of P less than P_u, the corresponding effective cross-sectional area shall be A_e.

4.2 The test method also provides a means to observe, measure, and account for local buckling deformations when the appearance of a column section under stress must be determined.

4.3 An inherent assumption of the test method is that true stub-column behavior (which considers local buckling effects only) is achieved when overall column-buckling effects are eliminated. For this condition the ultimate test load on a stub column, P_u, equals the product of the effective cross-sectional area at ultimate load, A_{eu}, times the stress that causes local buckling, or times the yield stress of the virgin steel sheet. In case overall buckling cannot be avoided because of geometrical constraints, the critical column-buckling stress must be used.

4.4 The determination of A_e may be conducted by either one of the two following methods:

(1) The basic, more simple, and conservative method:
This method is embodied in the main part of this document and is based on the measured test loads of stub columns and their measured and tested physical and mechanical properties.

(2) An alternate and less conservative method:
This method is based on the shortening of stub columns which occurs during testing. Also, this evaluation method requires more calculations. The results of this method lead to more accurate results for A_e, and to higher allowable axial loads at lower-than-ultimate stress levels. The evaluation procedure for this method is described in Appendix A.

5. Apparatus

5.1 The tests shall be conducted on a testing machine that complies with the requirements of ASTM E4.

5.2 Linear displacement devices for measuring lateral displacements shall have a 0.001-inch least-reading capability.

5.3 Measuring devices for determination of the actual geometry of a test specimen shall have a 0.001-inch least-reading capability.

5.4 If axial shortening is recorded, the measuring device shall have a 0.0001-inch least-reading capability.

6. Test Unit

6.1 A test unit shall include a minimum of three identical stub-column specimens and a minimum of two corresponding sheet-type tensile specimens.

6.2 The specimens within a unit shall represent one type of cold-formed steel section with the same specified geometrical, physical, and chemical properties. The specimens may be taken from the same column or from different production runs provided the source of the specimens is properly identified and recorded.

6.3 If stub-column specimens are taken from different production runs, at least two corresponding sheet-type specimens must be taken and tested from each production run.

6.4 The stub-column test specimens shall be used to determine:

(1) The actual geometry of each specimen.

(2) The ultimate stub-column test load.

(3) Axial shortenings at each load level if the alternate test-evaluation method described in Appendix A is used.

(4) Lateral displacements of the specimen at locations of interest (if desired).

6.5 The tensile test specimens shall be used to define the yield stress of each stub-column specimen acording to the requirements described in ASTM A370.

6.6 For each test specimen and test unit, the measured geometrical and tested physical properties of the individual specimens shall meet the requirements stated by the fabricator and material producer, respectively.

6.7 If the average area, thickness, or yield strength of a test unit varies by more than 20 percent from the respective nominal or specified-minimum value, the test unit is considered to be non-representative of the column section, and further evaluations of the effective area are considered to be invalid.

7. Stub-Column Specimens

The stub-column specimens shall meet length and end-flatness requirements as follows, depending on whether or not unconnected or welded endplates are used.

7.1 Stub-Column Length—The length requirements of the stub-column test specimen, L, as shown in Figures 1 and 2, are that it be (1) sufficiently short to eliminate overall column buckling effects, and (2) sufficiently long to minimize the end effects during loading, which means that its center portion be representative of the repetitive hole pattern in the full column.

7.1.1 To eliminate overall column-buckling effects, the stub-column length shall not exceed twenty times the minimum radius of gyration, r, of the cross section, A, except where necessary to meet the requirements of Sections 7.1.2 through 7.1.5.

7.1.2 For unperforated columns (Figure 1a) the stub-column length shall not be less than three times the greatest overall width of the cross section, W.

7.1.3 For perforated columns in which the pitch (gage length) of the perforation pattern, L_p, for a single hole or a group of holes, is smaller than, or equal to, the greatest overall width, W, of the cross section (Figures 1b and 1g), or for a single hole pattern with a gage length larger than the greatest overall width (Figure 1c), the specimen length shall not be less than three times the greatest overall width of the cross section, W. For widely spaced hole patterns (Figure 1c) the significant hole or hole pattern shall be located at or near the midlength of the stub column.

7.1.4 For perforated columns in which the pitch of the perforation pattern, L_p, is greater than the widest side, W, of the cross section (Figures 1d, 1e, 1f, and 1h), the specimen length shall not be less than three times the pitch of the perforation pattern.

7.1.5 For perforated sections in which the specimen end planes must pass through the normal perforation pattern (Figure 1i), a special section (Figure 1j) may be fabricated to obtain full cross-sectional surfaces at the specimen ends.

7.2 Stub-Column End Surface Preparation—The end planes of the stub-column test specimens shall be carefully cut to a flatness tolerance of plus or minus 0.002 inches. When the required flatness can be achieved, welding of the stub-column ends to the endplates is not required. However, when this flatness cannot be achieved, steel endplates shall be continuously welded to both ends of the specimen so that there shall be no gap between the ends of the stub column and the endplates.

7.3 Stub-Column Specimen Source—Stub-column test specimens may be cut from the commercially fabricated column product. Alternatively, stub columns may be specially

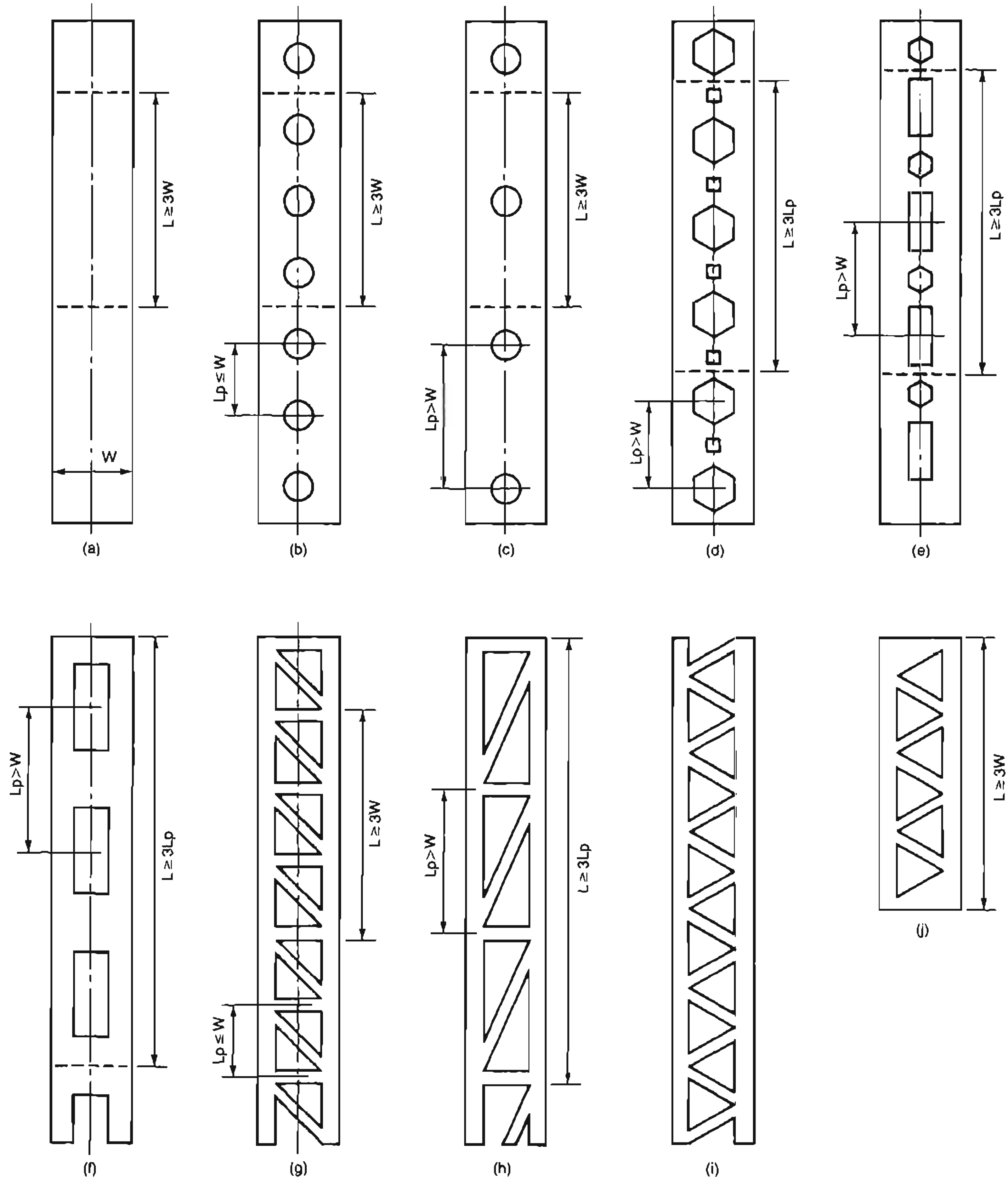


Figure 1 Hypothetical Perforation Patterns And Suggested Stub Column Lengths

NOTES: (1) Perforations shown are in a flat portion of a member with width W
(2) L = Length of Stub Column
(3) L_p = Pitch Length of Perforation Pattern

fabricated provided care is taken not to exceed the cold work of forming expected in the commercial product; however, subsequent proof tests using specimens from commerically produced columns are recommended.

7.4 Tensile Specimen Source—Longitudinal tensile specimens shall be cut from the center of the widest flat of a formed section from which the stub-column specimens have been taken. If perforations are large and frequent in all flats of the formed section, the tensile specimens may be taken from the sheet or coil material used for the fabrication of the stub-column specimens. The tensile specimens shall not be taken from parts of a previously tested stub column.

7.5 Endplate Requirements—Steel endplates shall be at least 0.5 inch thick and have a flatness tolerance of plus or minus 0.002 inches.

8. Stub-Column Test Procedure

8.1 Vertical alignment of the stub column is essential to ensure that the applied load is uniformly distributed over the specimen end surfaces. Care should also be taken to center the specimen on the axis of the test machine.

8.1.1 Steel endplates shall be used to transfer the test loads uniformly into the stub columns (Figure 2).

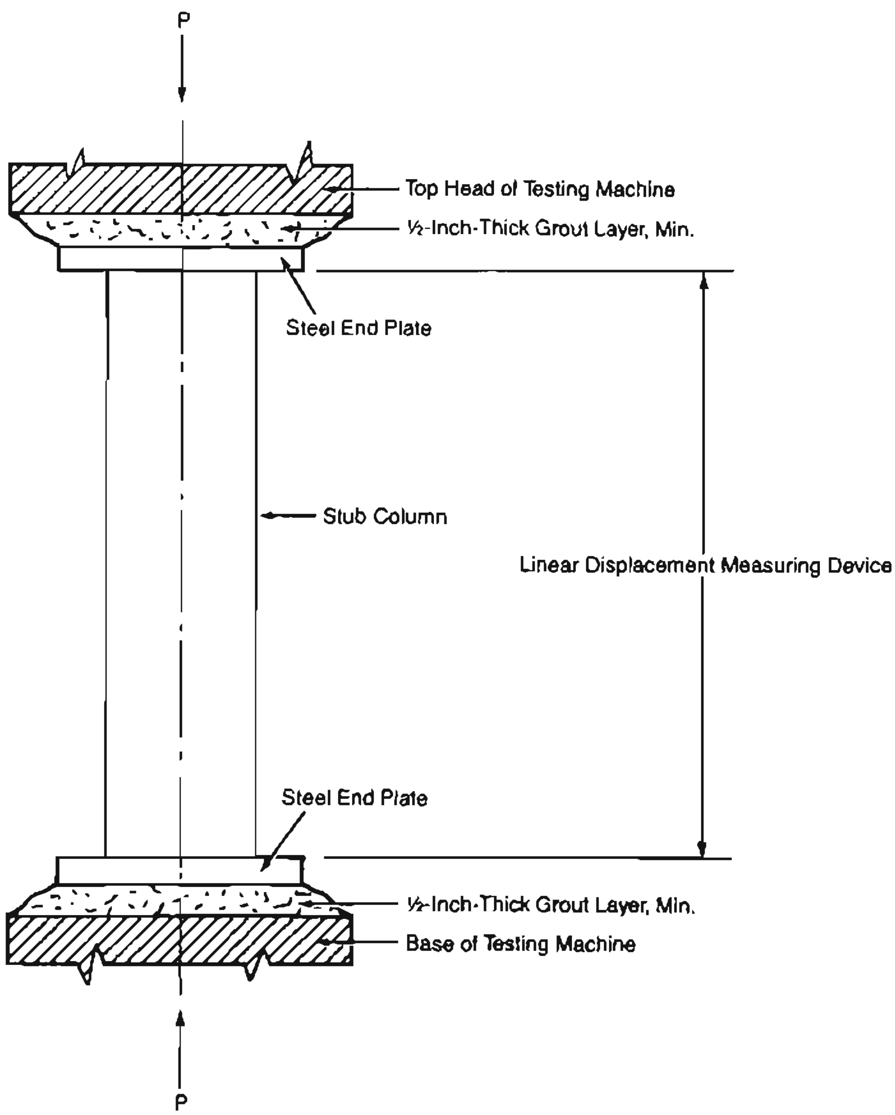


Figure 2 Test Setup

8.1.2 A ½-inch-thick layer of grout, similar to gypsum-based concrete capping compound used for fast setting, shall be placed between the stub-column endplates and the machine heads to facilitate aligning the test specimen (Figure 2).

8.2 When an axial compression load is applied to the test specimen as a result of grout expansion during curing, or if a small preload is purposely applied to ensure proper contact between the stub-column endplates and the machine heads, the load shall be treated as part of the applied test load.

8.3 The load increments applied during the test shall not exceed 10 percent of the estimated ultimate test load.

8.4 The maximum loading rate between load increments shall not exceed a corresponding applied stress rate of 3 kips per square inch of cross-sectional area per minute.

8.5 When axial shortening values are recorded, the following procedures shall be required:

(1) The change in the vertical distance between the inside surfaces of the endplates (Figure 2) shall be measured to the nearest 0.0001-inch at each load increment for each specimen.

(2) The load increments applied during the test shall be the same for each specimen within a test unit, with a variation not to exceed one percent.

9. Calculations

9.1 For a given test unit, all individual ultimate loads, P_u, derived from the stub-column tests shall be used to calculate the average ultimate load, P_{ua}. Similarly, all individual yield strengths, F_{yi}, derived from the tensile tests of the same unit shall be used to calculate the average yield stress of the same test unit, F_{ya}.

9.2 The effective areas A_{eua}, A_{eu}, and A_e shall be calculated as specified in Sections 9.3 through 9.6; however, the final value of these effective areas shall not exceed that of the minimum gross cross-sectional area, A.

9.3 For tests in which the length of the stub column does not exceed twenty times the minimum radius of gyration of the cross section, r, the average effective area at the ultimate load, A_{eua}, for a given test unit shall be calculated as

$$A_{eua} = P_{ua}/F_{ya}$$

9.4 For tests in which the length of the stub column exceeds twenty times the minimum radius of gyration of the cross section, the average effective area at the ultimate load shall be determined by iteration of the following equations:

$$A_{eua} = A_a - (A_a - P_{ua}/F_n)/(F_n/F_{ya})^n$$

where A_a is the average minimum gross area of the stub columns in the test unit, and F_n is the flexural or torsional-flexural buckling stress derived from Section C4 of the AISI Specification with K = 0.5 (using the average cross-sectional properties of the test unit). The exponent n is determined as follows:

$$n = A_{eua}/A_a$$

Assuming an initial value for n equal to less than 1.0, A_{eua} can be calculated from the first equation. Using this A_{eua} in the second equation will provide a new value for n. Repeating this process will lead to convergence of the above equations and an acceptable value of A_{eua} for one specific test unit.

9.5 The value of A_{eua} for a specific test unit shall be adjusted to A_{eu}, which is the effective cross-sectional area of a column at ultimate load with a nominal cross section of A and a specified minimum yield strength of F_y. The adjustment shall be performed in one or two steps as follows.

9.5.1 If the average area of the stub columns in the test unit, A_a, or the average base steel thickness, t_a, are diferent from the nominal area or thickness, respectively, the effective cross-sectional area at ultimate load shall be calculated as follows:

$$A_{eu} = A_{eua}(A/A_a)$$

or

$$A_{eu} = A_{eua}(t/t_a)$$

9.5.2 If the average yield strength of all stub columns in a test unit, F_{ya}, is different from the nominal yield strength, F_y, the effective cross-sectional area at ultimate load shall be the lower of the two values calculated as follows:

$$A_{eu} = A[1 - (1\text{-}A_{eua}/A)(F_y/F_{ya})]$$

or

$$A_{eu} = A_{eua}(F_{ya}/F_y)^{0.4}$$

9.5.3. If the average area and the minimum specified yield strength are different from the nominal values of a test unit, A_{eu} derived from the equation in Section 9.5.1 shall be used as A_{eua} in the equations of Section 9.5.2, which will lead to an acceptable value of A_{eu}.

9.6 The effective area at any working stress level, A_e, may be determined by

$$A_e = A - (A\text{-}A_{eu})(f/F_y)^n$$

9.7 For a series of sections, such as in a parameter study during which only one parameter (thickness, depth, width, yield strength, etc.) is changed, interpolations between test units, or extrapolations beyond test units, shall be acceptable as described in Appendix B.

9.8 Extrapolations beyond 20 percent of the extreme parameters tested shall not be permitted.

10. Report

10.1 Documentation—The report shall include a complete record of the sources and locations of all stub-column and tensile-test specimens and shall describe whether the specimens were taken from one or several columns, one or several production runs, coil stock, or other sources.

10.2 The documentation shall include all measurements taken for each stub-column test specimen, including (1) cross-section dimensions, (2) uncoated sheet thickness, (3) longitudinal yield strength, (4) end preparation procedure, (5) applicable material specification, and (6) test and evaluation procedure used.

10.3 The determination of the selected stub-column length shall be fully documented with appropriate calculations.

10.4 A description of the test setup—including the endplates, the grout layer used for alignment, and the instrumentation used to measure lateral displacements and axial shortening—shall be included.

10.5 The report shall include the load increments, rate of loading, and intermediate and ultimate loads for each stub column tested.

10.6 The report shall include complete calculations and results of the effective area, A_{eu}, for each test unit and calculations of A_e, if requested.

11. Precision

11.1 The following criteria shall be used to judge the acceptability of the test results.

11.1.1 Repeatability—Individual stub-column test results shall be considered suspect if they differ by more than 10 percent from the mean value for a test unit with at least three specimens.

11.1.2 Reproducability—The results of tests on stub-columns conducted at two or more laboratories should agree within ten (10) percent when adjusted for differences in cross sectional dimensions and yield strength.

REFERENCES

(1) T. Pekoz, "Development of a Unified Approach to the Design of Cold-Formed Steel Members, Committee of Sheet Steel Producers, American Iron and Steel Institute, 1000 16th Street, NW, Washington, DC 20036, 1986.

APPENDIX A

Use Of Axial Shortening Measurements In Design

A-1 Axial shortening measurements as part of thin-walled cold-formed steel stub-column tests may be used as an alternative method of determining the effective area of a column, A_e, at a certain design load or stress. This method provides a more accurate and less conservative alternative to design engineers to determine the effective area of a column section, A_e.
A-2 The calculations by this method shall be made separately for each stub-column specimen within a test unit. This shall result in a total of j calculations as a result of a total of j load-displacement tests for each test unit.
A-3 For a given specimen the effective area at ultimate load, A_{eu}, shall be calculated from Section 9.3 or 9.4 letting $A_{eua} = A_{eu}$, $A_a = A$, $F_{ya} = F_y$, and $P_{ua} = P_u$.

A-3.1 Calculations at each load-displacement reading, i, shall be conducted according to the following procedure; however, at zero load, the effective area, A_e, shall be equal to the minimum gross cross-sectional area, A. This provides results for the effective area at each load point:

(1) Starting with the lowest load-displacement reading, the effective area, A_i, and the assumed uniformly distributed stress f_i, shall be calculated for each reading, i, from:

and $$A_{ei} = \frac{P_i D_u}{F_y D_i}$$

$$f_i = F_y D_i / D_u$$

where D_i an D_u are the axial shortening at loads P_i and P_u, respectively.

(2) If A_{ei} calculated is greater than A, A_{ei} shall be set equal to A.

(3) If A_{ei} calculated is less than A, A_{ei} shall be as calculated, and f_o, the stress above which the section is not fully effective, shall be set equal to f_{i-1}, as calculated for the previous load-displacement reading.

A.3.2 For specimens within a test unit, the lowest A_{ei} values shall be used for further evaluations.

A-4 For any load that causes a stress f higher than f_o, an exponential equation may be developed as follows.:

$$A_e = A[1-(1-A_{eu}/A)(f-f_o)/F_y - f_o)]^b$$

where $$b = \frac{\sum_{i=1}^{j} (X_i)(Y_i) - (a)\sum_{i=1}^{j} (X_i)}{\sum_{i=1}^{j} (X_i)^2}$$

and $$X = \ln[(f_i - f_o)/(F_y - f_o)]$$

$$Y_i = \ln(1 - A'_{ei}/A)$$

$$a = \ln(1 - A_{eu}/A)$$

and ln designates the natural logarithm.

A-5 If the effective areas for a section with specified dimensions and minimum yield strength are desired, which are different from the tested specimens, the Aeu and Aei values calculated under Section A-3 shall be normalized to the specified parameters according to Section 9.5 before the curve-fitting procedure of Section A-4 is employed.

A-6 All calculations pertaining to this procedure shall be included in the report, as discussed in Section 10.

APPENDIX B

Parametric Studies

B-1 For parametric studies intended to develop the effective area for a series of sections with the same basic cross section (either C, U, H, or any other shape) and the same hole pattern, but with one or more changing parameters, the required number of test units may be less than the sum of all sections with different geometries and yield strengths.

B-1.1 For a series of sections with three different values for one parameter only (dimension or nominal yield strength), at least two test units shall be chosen to include the minimum and the maximum value of the changing parameter. For the third value, A_{eu} may be interpolated according to Section B-2.

B-1.2 If more than three different values for one parameter are included in a series of sections, additional units with intermediate values shall be tested such that the ratio of the changing values in adjacent units is not greater than 1.5 or be less than 0.67. For intermediate values of the changing parameter, A_{eu} may be interpolated according to Section B-2.

B-1.3 For a series of sections with the same basic cross section that includes different values for several parameters (dimensions and/or yield strength), an appropriate factorial of test units shall be established by the responsible professional engineer in accordance with the guidelines for changes in an individual parameter, and in compliance with responsible code authorities. Interpolations and extrapolations may be made as mutually agreeable, following the general guidelines set forth in Section B-2 for changes of one parameter only.

B-1.4 For a section that falls outside a series of tested members with the same basic cross section, A_{eu} may be extrapolated provided the changing parameter does not exceed a value of 20 percent below or above the respective minimum or maximum values tested in the series.

B-2 Interpolations and extrapolations are allowed as part of a parametric study, and as defined under B-1.

B-2.1 For a section with a thickness different from the thicknesses tested, but with identical overall nominal cross-sectional dimensions and minimum specified yield strength, A_{eu} for a thickness t and an area A may be calculated provided t does not exceed the limits described under Section B-1.2 and B-1.4. Under these conditions, A_{eu} may be determined by interpolation or extrapolation from the results of the nearest two test units with thicknesses t_1 and t_2, respectively:

$$A_{eu} = A[A_{eu1}/A_1 + (A_{eu2}/A_2 - A_{eu1}/A_1)(t_1-t)/(t_1-t_2)]$$

where A_1 and A_2 are the minimum gross cross-sectional areas, and A_{eu1} and A_{eu2} are the nominal effective cross-sectional areas for Test Units 1 and 2, respectively.

B-2.2. For a section with a yield strength different from the yield strengths tested, but with identical cross-sectional dimensions, A_{eu} for a yield strength F_y may be calculated provided F_y does not exceed the limits described under Section B-1.2 and B-1.4. Under these conditions, A_{eu} may be determined by interpolation or extrapolation from the results of the nearest two test units with yield strengths F_{y1} and F_{y2}, and with effective areas A_{eu1} and A_{eu2}, respectively:

$$A_{eu} = A[A_{eu1}/A_1 + (A_{eu2}/A_2-A_{eu1}/A_1)(F_{y1}-F_y)/(F_{y1}-F_{y2})]$$

STANDARD METHODS FOR DETERMINATION OF UNIFORM AND LOCAL DUCTILITY

1. Scope

This method covers the determination of uniform and local ductility from a tension test. Its primary use is as an alternative method of determining if a steel has adequate ductility as defined in the AISI Specification. It is based on the method suggested by Dhalla and Winter.

2. Referenced Documents

ASTM Standard A370, "Standard Test Methods and Definitions for Mechanical Testing of Steel Products."

AISI Specification for the Design of Cold-Formed Steel Structural Members, 1986 Specification with the December 11, 1989 Addendum.

Dhalla, A. K. and Winter, G., "Steel Ductility Measurements," Journal of Structural Division, Proceedings ASCE, Vol. 100, No. ST2, February 1974.

3. Symbols

e_3 = linear elongation, in., in 3-in. gage length
e_{3e} = linear elongation, in., in 2-in gage length not containing 1-in. length of fractured portion
e_u = linear elongation, in., at ultimate load in standard tension coupon test
ϵ_3 = percent elongation in 3-in. gage length
ϵ_{3e} = percent elongation in 2-in. gage length not containing 1-in. length of fractured portion
ϵ_f = percent elongation at fracture in 2-in. gage length of standard tension coupon
ϵ_u = percent elongation at ultimate load in standard tension coupon test
$\epsilon_{uniform}$ = uniform percent elongation
ϵ_{local} = local percent elongation in 1/2 in. gage length
$\epsilon_{1/2}$ = percent elongation in 1/2 in. gage length

4. Test Procedure

4.1 Prepare a tension coupon according to ASTM Standard A370 except that the central length of 1/2 in. (12.7 mm) uniform width of the coupon should be at least 3½ in. (88.9 mm) long.

4.2 Scribe gage lines at 1/2-in. (12.7 mm) intervals along the entire length of the coupon.

4.3 After completion of the coupon test, measure the following two permanent plastic deformations: (a) the linear elongation in a 3-in. (76.2 mm) gage length, e_3, such that the fractured portion is included (preferably near the middle third of this 3-in. gage length); and (b) the linear elongation in a 1-in. (25.4 mm) gage length containing the fracture.

4.4 Subtract the latter from the former. This difference gives the linear elongation, e_{3e}, in a 2-in. (50.8 mm) gage length not containing the 1-in. length of the fractured portion.

4.5 From the two preceding elongation measurements, e_3 and e_{3e}, calculate the percentage elongations $\epsilon_3 = (e_3/3) \times 100$, and $\epsilon_{3e} = (e_{3e}/2) \times 100$. From these percentage elongations, the uniform and local ductility parameters are obtained as follows.

4.6 Since the fractured portion which includes local elongation is eliminated from ϵ_{3e}, it is a measure of the uniform ductility of the material. Thus

$$\epsilon_{uniform} = \epsilon_{3e} \tag{1}$$

4.7 The local elongation is determined over a small length which includes the fractured portion. For simplicity, this length is here assumed to be 1/2 in. (12.7 mm) which is large enough to include the necked portion of most thicknesses and type of sheet steels used, and is small enough to give valid comparison for different types of steels. Thus

$$\epsilon_{local} = \epsilon_{1/2} = 6\ (\epsilon_3 - \epsilon_{3e}) + \epsilon_{3e} \tag{2}$$

in which 6 = the multiplication factor which converts the local elongation ($\epsilon_3 - \epsilon_{3e}$) measured in 3 in. (76.2 mm) to local elongation in 1/2 in. (12.7 mm) gage length.

5. Alternate Test Procedure

5.1 Prepare a standard tension coupon according to ASTM A370 with a standard 2-in. (50.8 mm) gage length.

5.2 The strain at the tensile strength, i.e., percentage strain ϵ_u at the peak of the stress-strain curve, is a measure of uniform ductility, because up to this strain no necking or local elongation has taken place. Therefore, to obtain the uniform ductility the stress-strain curve is plotted at least up to the maximum load or the linear elongation, e_u, at maximum load is measured directly, so that $\epsilon_u = (e_u/2) \times 100$.

5.3 To obtain a measure of the local ductility it is necessary to measure the percentage strain at fracture ϵ_f, also in a 2-in. gage length. However, the strain which occurs after the maximum load has been passed (descending branch) is the necking strain, and is localized at the eventual fracture zone, thus ($\epsilon_f - \epsilon_u$) is the local percentage elongation referred to in a 2-in. (50.4 mm) gage length. The following equation converts this ($\epsilon_f - \epsilon_u$) into the percentage elongation in a 1/2 in. (12.7 mm) gage length:

$$\epsilon_{local} = \epsilon_{1/2} = \epsilon_u + 4\ (\epsilon_f - \epsilon_u) \tag{3}$$

in which 4 = the multiplication factor to convert a 2-in. gage length local elongation to a 1/2 in. gage length.

American Iron and Steel Institute
1101 17th Street, NW
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