SEI/ASCE 8-02

ASCE STANDARD

American Society of Civil Engineers

Specification for the Design of Cold-Formed Stainless Steel Structural Members

This document uses both Systéme International (SI) units and customary units.

ASCE

SEI
Structural Engineering Institute

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Specification for the Design of Cold-Formed Stainless Steel Structural Members

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Structural Engineering Institute

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ABSTRACT
ASCE's standard *Specification for the Design of Cold-Formed Stainless Steel Structural Members* (ASCE 8-02) provides design criteria for the determination of the strength of stainless steel structural members and connections for use in buildings and other statically loaded structures. The members may be cold-formed to shape from annealed and cold-rolled sheet, strip, plate, or flat bar stainless steel material. Design criteria are provided for axially loaded tension or compression members, flexural members subjected to bending and shear, and members subjected to combined axial load and bending. The specification provides the design strength criteria using the load and resistance factor design (LRFD) and the allowable stress design (ASD) methods. The reasoning behind, and the justification for, various provisions of the specification are also presented. The design strength requirements of this standard are intended for use by structural engineers and those engaged in preparing and administrating local building codes.

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STANDARDS

In April 1980, the Board of Direction approved ASCE Rules for Standards Committees to govern the writing and maintenance of standards developed by the Society. All such standards are developed by a consensus standards process managed by the Management Group F (MGF), Codes and Standards. The consensus process includes balloting by the balanced standards committee made up of Society members and nonmembers, balloting by the membership of ASCE as a whole, and balloting by the public. All standards are updated or reaffirmed by the same process at intervals not exceeding 5 years.

The following Standards have been issued.

ANSI/ASCE 1-82 N-725 Guideline for Design and Analysis of Nuclear Safety Related Earth Structures
ANSI/ASCE 2-91 Measurement of Oxygen Transfer in Clean Water
ANSI/ASCE 3-91 Standard for the Structural Design of Composite Slabs and ANSI/ASCE 9-91 Standard Practice for the Construction and Inspection of Composite Slabs
ASCE 4-98 Seismic Analysis of Safety-Related Nuclear Structures
Building Code Requirements for Masonry Structures (ACI 530-99/ASCE 5-99/TMS 402-99) and Specifications for Masonry Structures (ACI 530.1-99/ASCE 6-99/TMS 602-99)
ASCE 7-98 Minimum Design Loads for Buildings and Other Structures
ASCE 8-90 Standard Specification for the Design of Cold-Formed Stainless Steel Structural Members
ANSI/ASCE 9-91 listed with ASCE 3-91
ASCE 10-97 Design of Latticed Steel Transmission Structures
SEI/ASCE 11-99 Guideline for Structural Condition Assessment of Existing Buildings
ANSI/ASCE 12-91 Guideline for the Design of Urban Subsurface Drainage
ASCE 13-93 Standard Guidelines for Installation of Urban Subsurface Drainage
ASCE 14-93 Standard Guidelines for Operation and Maintenance of Urban Subsurface Drainage
ASCE 15-98 Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations (SIDD)
ASCE 16-95 Standard for Load and Resistance Factor Design (LRFD) of Engineered Wood Construction
ASCE 17-96 Air-Supported Structures
ASCE 18-96 Standard Guidelines for In-Process Oxygen Transfer Testing
ASCE 19-96 Structural Applications of Steel Cables for Buildings
ASCE 20-96 Standard Guidelines for the Design and Installation of Pile Foundations
ASCE 21-96 Automated People Mover Standards—Part 1
ASCE 21-98 Automated People Mover Standards—Part 2
ASCE 21-00 Automated People Mover Standards—Part 3
SEI/ASCE 23-97 Specification for Structural Steel Beams with Web Openings
SEI/ASCE 24-98 Flood Resistant Design and Construction
ASCE 25-97 Earthquake-Actuated Automatic Gas Shut-Off Devices
ASCE 26-97 Standard Practice for Design of Buried Precast Concrete Box Sections
ASCE 27-00 Standard Practice for Direct Design of Precast Concrete Pipe for Jacking in Trenchless Construction
ASCE 28-00 Standard Practice for Direct Design of Precast Concrete Box Sections for Jacking in Trenchless Construction
SEI/ASCE 30-00 Guideline for Condition Assessment of the Building Envelope
EWRI/ASCE 33-01 Comprehensive Transboundary International Water Quality Management Agreement
EWRI/ASCE 34-01 Standard Guidelines for Artificial Recharge of Ground Water
EWRI/ASCE 35-01 Guidelines for Quality Assurance of Installed Fine-Pore Aeration Equipment
CI/ASCE 36-01 Standard Construction Guidelines for Microtunneling
SEI/ASCE 37-02 Design Loads on Structures During Construction

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FOREWORD

Prior to 1990, the design of cold-formed stainless steel structural members was based on the allowable stress design specification issued by the American Iron and Steel Institute. Based on the initiative of Chromium Steels Research Group at Rand Afrikaans University in 1989, a new ASCE Standard Specification for the Design of Cold-Formed Stainless Steel Structural Members was developed at the University of Missouri-Rolla under the Sponsorship of the American Society of Civil Engineers. It was subsequently reviewed and approved by the ASCE Stainless Steel Cold-Formed Sections Standards Committee in 1990. This ASCE project was financially supported by the Chromium Centre in South Africa, the Nickel Development Institute in Canada, and the Specialty Steel Industry of the United States. The development of this new ASCE Standard Specification was primarily based on the 1974 Edition of the AISI specification for stainless steel design and the recent extensive research conducted by Chromium Steels Research Group at Rand Afrikaans University under the sponsorship of Columbus Stainless Steel (the Middleburg Steel and Alloys) in South Africa.

This new ASCE Standard Specification includes both the load and resistance factor design (LRFD) method and the allowable stress design (ASD) method. In the LRFD method, 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.

The material presented in this publication has been prepared in accordance with recognized engineering principles. This Standard and Commentary should not be used without first securing competent advice with respect to suitability for any given application. The publication of the material contained herein is not intended as a representation or warranty on the part of the American Society of Civil Engineers, or of any other person named herein, that this information is suitable for any general or particular use or promises freedom from infringement of any patent or patents. Anyone making use of this information assumes all liability from such use.

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ASCE acknowledges the work of the Stainless Steel Cold-Formed Section Standards Committee of the Management Group F. Codes and Standards. This group comprises individuals from many backgrounds including: consulting engineering, research, construction industry, education, government, design, and private practice.

This Standard was prepared through the consensus standards process by balloting in compliance with procedures of ASCE's Management Group F. Codes and Standards. Those individuals who serve on the Standards Committee are:

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COMMENTARY CONTENTS

(This Commentary is not a part of the standard. It is included for information purposes.)
Commentary on ASCE Standard Specification for the Design of Cold-Formed Stainless Steel Structural Members

Appendices

Tables

Figures

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NOTATION

Symbol	Definition	Section
A	Full, unreduced cross-sectional area of the member	3.3.1.2,3.4
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	App. C.1
A_b	Gross cross-sectional area of bolt	5.3.4
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	App. C.1
A_e	Effective area at the stress F_n	3.4,3.6.2
A_n	Net area of cross section	3.2,5.3.2
A_o	Reduced area of cross section	3.6.2
A_s	Cross-sectional area of transverse stiffeners	2.4,2.4.1,2.4.2, App. C.1
A'_s	Effective area of stiffener	2.4,2.4.1,2.4.2
A_{st}	Gross area of shear stiffener	App. C.2
a	For a reinforced web element, the distance between transverse stiffeners	App. C.2
a	Length of bracing interval	4.3.2.2
b	Effective design width of compression element	2.2.1,2.2.2,2.3.1,2.3.2, 2.4.1,2.4.2,2.5
b_d	Effective width for deflection calculation	2.2.1,2.2.2
b_e	Effective design width of sub-element or element	2.2.2,2.5
b_o	See Figure 4	2.4,2.4.1,2.5
b_1,b_2	Effective widths, see Figure 2	2.2.2
C	Ratio of effective proportional limit-to-yield strength, F_{pr}/F_y	3.6.1
C_b	Bending coefficient dependent on moment gradient	3.3.1.2
C_m	End moment coefficient in interaction formula	3.5
C_{mx}	End moment coefficient in interaction formula	3.5
C_{my}	End moment coefficient in interaction formula	3.5
C_s	Coefficient for lateral torsional buckling	3.3.1.2
C_v	Shear stiffener coefficient	App. C.2
C_w	Torsional warping constant of cross section	3.3.1.2
C_y	Compression strain factor	3.3.1.1
C_1	Coefficient as defined in Figures 4 and 5	2.4,2.4.2
C_2	Coefficient as defined in Figures 4 and 5	2.4,2.4.2
c_f	Amount of curling	2.1.1
D	Outside diameter of cylindrical tube	3.6.1,3.6.2
D	Dead load, includes weight of test specimen	6.2
D	Overall depth of lip	2.1.1,2.4,2.4.2,4.1.1
D	Shear stiffener coefficient	App. C.2
D_n	Nominal dead load	1.5.2
d	Depth of section	2.4,2.1.1,3.3.1, 4.1.1,4.3.2.2
d	Diameter of bolt	5.3,5.3.1,5.3.2,5.3.3
d_h	Diameter of standard hole	5.3.1
d_s	Reduced effective width of stiffener	2.4,2.4.2
d'_s	Actual effective width of stiffener	2.4,2.4.2
E_n	Nominal earthquake load	1.5.2

Symbol	Definition	Section
E_o	Initial modulus of elasticity	2.2.1,2.3.1,2.4,2.5, 3.3.1.1,3.3.1.2,3.3.2, 3.3.5,3.6.1,3.6.2, 4.3.3,App. B
E_r	Reduced modulus of elasticity	2.2.1
E_s	Secant modulus	3.3.1.1,3.4,App. B
E_{sc}	Secant modulus in compression flange	2.2.1
E_{st}	Secant modulus in tension flange	2.2.1
E_s/E_o	Plasticity reduction factor for unstiffened compression elements	3.3.1.1,3.4,App. B
E_t	Tangent modulus in compression	3.3.1.1,3.4,App. B, 3.4.1,3.6.2
E_t/E_o	Plasticity reduction factor for lateral buckling	3.3.1.2,3.6.2,App. B
$\sqrt{E_t/E_o}$	Plasticity reduction factor for stiffened compression elements	3.3.1.1,3.4,App. B
e	Distance measured in line of force from centerline of standard hole to nearest edge of adjacent hole or to end of connected part toward which force is directed	5.3.1
e_y	Yield strain $= F_y/E_o$	3.3.1.1
F_{cr}	Critical buckling stress	3.3.1.1,3.4
F_D	Dead load factor	6.2
F_L	Live load factor	6.2
F_n	Nominal buckling stress	3.4,3.6.2
F_{nt}	Nominal tensile strength of bolts	5.3.4
F_{nv}	Nominal shear strength of bolts	5.3.4
F'_{nt}	Nominal tensile strength for bolts subject to combination of shear and tension	5.3.4
F_p	Nominal bearing stress	5.3.3
F_{pr}	Effective proportional limit	3.6.1
F_t	Nominal tension stress limit on net section	5.3.2
F_u	Tensile strength in longitudinal direction	5.3.1,5.3.2,5.3.3,5.3.4
F_{ua}	Tensile strength of annealed base metal	5.2.1,5.2.2
F_{xx}	Strength level designation in AWS electrode classification	5.2.2
F_y	Yield strength used for design, not to exceed specified yield strength or established in accordance with Section 6.4, or as increased for cold work of forming in Section 1.5.4.2	1.5.4.2,2.2.1,2.5,3.2, 3.3.1,3.3.2,3.3.4,3.3.5, 3.6.1,3.6.2,5.2.1, App. C.1,App. B
F_{yc}	Yield strength in compression	3.3.1.1
F_{yt}	Yield strength in tension	3.3.1.1
F_{ys}	Yield strength of stiffener steel	App. C.1
F_{yv}	Shear yield strength	3.3.2
F_{yw}	Lower value of yield strength in beam web F_y or stiffener section F_{ys}	App. C.1
f	Stress in compression element computed on the basis of the effective design width	2.2.1,2.2.2, 2.3.2,2.4,2.4.1
f_{av}	Average computed stress in full, unreduced flange width	2.1.1
f_b	Perceptible stress for local distortion	3.3.1.1,3.4
f_d	Computed compressive stress in element being considered. Calculations are based on effective section at load for which deflections are determined.	2.2.1,2.2.2, 2.3.1,2.4.1,2.4.2
f_{d1},f_{d2}	Computed stresses f_1 and f_2 as shown in Figure 2. Calculations are based on the effective section at the load for which deflections are determined	2.2.2

Symbol	Definition	Section
f_{d3}	Computed stress f_3 in edge stiffener, as shown in Figure 5. Calculations are based on the effective section at the load for which deflections are determined.	2.3.2
f_v	Computed shear stress on bolt	5.3.4
f_1,f_2	Web stresses defined by Figure 2	2.2.2
f_3	Edge stiffener stress defined by Figure 5	2.3.2
G_o	Initial shear modulus	3.3.2
G_s	Secant shear modulus	3.3.2
G_s/G_o	Plasticity reduction factor for shear stress	3.3.2
g	Vertical distance between two rows of connections nearest to top and bottom flanges	4.1.1
h	Depth of flat portion of web measured along plane of web	2.1.2,3.3.2, 3.3.4,App. C.2
I_a	Adequate moment of inertia of stiffener so that each component element will behave as stiffened element	2.4.1,2.4.2
I_b	Moment of inertia of full, unreduced section about axis of bending	3.5
I_s	Actual moment of inertia of full stiffener about its own centroidal axis parallel to the element to be stiffened	2.1.1,2.4,2.4.1, 2.4.2, 2.5
I_{sf}	Moment of inertia of full area of multiple stiffened element, including intermediate stiffeners, about its own centroidal axis parallel to element to be stiffened	2.5
I_x, I_y	Moments of inertia of full section about principal axes	4.1.1,4.3.2.2
I_{xy}	Product of inertia of full section about major and minor centroidal axes	4.3.2.2
I_{yc}	Moment of inertia of compression portion of section about gravity axis of the entire section about the *y*-axis	3.3.1.2
J	St. Venant torsion constant	3.3.1.2
j	Section property for torsional-flexural buckling	3.3.1.2
K	Effective length factor	3.4,3.4.1
K′	A constant	4.3.2.2
K_b	Effective length factor in plane of bending	3.5
K_c	Reduction factor due to local buckling	3.6.1,3.6.2
K_t	Effective length factor for torsion	3.3.1.2
K_x	Effective length factor for bending about *x*-axis	3.3.1.2
K_y	Effective length factor for bending about *y*-axis	3.3.1.2
k	Plate buckling coefficient	2.2.1,2.2.2,2.3.1, 2.3.2,2.4.1,2.4.2
k_v	Shear buckling coefficient	App. C.2
L	Full span for simple beams, distance between inflection points for continuous beams, twice length of cantilever beams	4.1.1,2.1.1
L	Length of fillet weld	5.2.2
L	Unbraced length of member	3.3.1.2,3.4.1
L_b	Actual unbraced length in plane of bending	3.5
L_n	Nominal live load	1.5.2
L_{rn}	Nominal roof live load	1.5.2
L_{st}	Length of transverse stiffener	App. C.1
L_t	Unbraced length of compression member for torsion	3.3.1.2
L_x	Unbraced length of compression member for bending about *x*-axis	3.3.1.2
L_y	Unbraced length of compression member for bending about *y*-axis	3.3.1.2
M_c	Critical moment	3.3.1.2
M_{ld}	Permissible moment for local distortions	3.3.1.1

Symbol	Definition	Section
M_n	Nominal moment strength	3.3.1.1,3.3.1.2, 3.3.3,3.3.5,3.6.1
M_{nx},M_{ny}	Nominal flexural strength about centroidal axes determined in accordance with Section 3.3	3.5
M_u	Required flexural strength	3.3.3,3.3.5
M_{ux}	Required flexural strength bent about x-axis	3.5
M_{uy}	Required flexural strength bent about y-axis	3.5
M_y	Moment causing maximum strain e_y	2.2.1,3.3.1.2
M_1	Smaller end moment	3.3.1.2,3.5
M_2	Larger end moment	3.3.1.2,3.5
m	Distance from shear center of one channel to mid-plane of its web	4.1.1,4.3.2.2
N	Actual length of bearing	3.3.4
n	Coefficient	App. B
P	Concentrated load or reaction	4.1.1
P_E	$\pi^2 E_o I_b/(K_b L_b)^2$	3.5
P_L	Force to be resisted by intermediate beam brace	4.3.2.2
P_{ld}	Permissible load for load distortions	3.4
P_n	Nominal axial strength of member	3.3.4,3.3.5,3.4,3.6.2
P_n	Nominal strength of connection	5.2.1,5.2.2,5.2.3, 5.3.1,5.3.2,5.3.3,5.3.4
P_{no}	Nominal axial load determined in accordance with Section 3.4 for $F_n = F_y$.	3.5
P_u	Required axial strength	3.3.5,3.5
q	Uniformly distributed factored load in plane of web	4.1.1
R_p	Average tested value	6.2
R	Inside bend radius	3.3.4
R_a	Allowable design strength	App. D
R_{rn}	Nominal roof rain load	1.5.2
R_n	Nominal strength	1.5.1.1,1.5.3
r	Radius of gyration of full, unreduced cross section	3.3.1.1,3.4.1
r	Force transmitted by the bolt or bolts at the section considered, divided by the tension force in the member at that section	5.3.2
r_{cy}	Radius of gyration of one channel about its centroidal axis parallel to web	4.1.1
r_I	Radius of gyration of I-section about the axis perpendicular to the direction in which buckling would occur for given conditions of end support and intermediate bracing	4.1.1
r_o	Polar radius of gyration of cross section about shear center	3.3.1.2,3.4.3
r_x,r_y	Radius of gyration of cross section about centroidal principal axes	3.3.1.2
S	$1.28\sqrt{\mathbf{E}_o/f}$	2.4,2.4.1
S_c	Elastic section modulus of effective section calculated at stress M_c/S_f in extreme compression fiber	3.3.1.1,3.3.1.2
S_e	Elastic section modulus of effective section calculated with extreme compression or tension fiber at F_y	3.3.1.1
S_F	Elastic section modulus of full, unreduced section for the extreme compression fiber	3.3.1.1,3.3.1.2,3.6.1
S_n	Nominal snow load	1.5.2
s	Fastener spacing	4.1.2

Symbol	Definition	Section
s	Spacing in line of stress of welds, rivets, or bolts connecting a compression cover plate or sheet to a nonintegral or other element	5.3.2
s	Weld spacing	4.1.1
s_{max}	Maximum permissible longitudinal spacing of welds or other connectors joining two channels to form I-section	4.1.1
T_n	Nominal tensile strength	3.2
T_s	Strength of connection in tension	4.1.1
t	Base steel thickness of any element or section	1.1.2,1.3.4,1.5.2.1, 2.1.1,2.1.2,2.2.1,2.4, 2.4.1,2.4.2,2.5,2.6.1, 3.3.1.1,3.3.1.3,3.3.2, 3.3.4,3.3.5,3.4,3.6.1, 3.6.2,4.1.2,5.2.2,5.3.2, App. C
t	Thickness of thinnest connected part	5.3.1
t_s	Equivalent thickness of multiple-stiffened element	2.5,App. C1
t_w	Effective throat of weld	5.2.2
V	Actual shear strength	3.3.3
V_n	Nominal shear strength	3.3.2
V_u	Required shear strength	3.3.3
w	Flat width of element exclusive of radii	1.1.2,2.1.2,2.2.1,2.4, 2.4.1,2.4.2,2.5,3.3.1.1, 3.3.1.3,3.3.2,3.3.4, 3.3.5,3.4,3.6.1, 3.6.2,4.1.2
w	Flat width of bearing plate	3.3.5
w_f	Width of flange projection beyond web or half distance between webs for box- or U-type sections	2.1.1c
w_f	Projection of flanges from inside face or web	2.1.1b
W_n	Nominal wind load	1.5.2
x	Distance from concentrated load to brace	4.3.2
x_o	Distance from shear center to centroid along the principal x-axis	3.3.1.2,3.4.3
Y	Yield strength of web steel divided by yield strength of stiffener steel	App. C.2
α	Reduction factor for computing effective area of stiffener section	2.5
α	Coefficient, for sections with stiffening lips, $\alpha = 1.0$; for sections without stiffening lips, $\alpha = 0$	4.1.1
$1/\alpha_{nx}$	Magnification factor	3.5
$1/\alpha_{ny}$	Magnification factor	3.5
β	Coefficient	3.4.3
η	Plasticity reduction factor	3.3.1.1,3.4,App. B
θ	Angle between web and bearing surface $\geq 45°$ but no more than $90°$	3.3.4
μ	Poisson's ratio in elastic range $= 0.3$	3.3.1.1,3.4
σ_{ex}	Buckling stress about x-axis	3.4.2,3.4.3
σ_{ey}	Buckling stress about y-axis	3.3.1.2
σ_t	Torsional buckling stress	3.3.1.2,3.4.3
σ	Normal stress	App. B
ε	Normal strain	App. B
ρ	Reduction factor	2.2.1
λ	Slenderness factor	2.2.1

Symbol	Definition	Section
λ_c	$3.048C$	3.6.1
ψ	f_2/f_1	2.2.2
ϕ	Resistance factor	1.1.1,1.5.1.1,5.2,5.2.1, 5.2.2,5.2.3,5.3.1,5.3.2, 5.3.3,5.3.4,6.2, App. D
ϕ_b	Resistance factor for bending strength	3.3.1,3.3.1.1,3.3.1.2, 3.3.3,3.3.5,3.5, 3.6.1,3.7
ϕ_c	Resistance factor for concentrically loaded compression member	3.4,3.5,3.6.2,App. C
ϕ_d	Resistance factor for local distortion	3.3.1.1,3.4
ϕ_t	Resistance factor for tension member	3.2
ϕ_v	Resistance factor for shear strength	3.3.2,3.3.3,App. C
ϕ_w	Resistance factor for web crippling strength	3.3.4,3.3.5
Ω	Safety factor	App. D

CONVERSION TABLE

This table contains some conversion factors between US Customary and SI Metric Units. The formulas included in this Specification are generally nondimensional, except that some adjustments are required for SI Unit in Section 3.3.4.

Metric Conversion Table

	To convert	to	Multiply by
Length	in.	mm	25.4
	mm	in.	0.03937
	ft	m	0.30480
	m	ft	3.28084
Area	in.^{2}	mm^2	645.160
	mm^2	in.^{2}	0.00155
	ft^2	m^2	0.09290
	m^2	ft^2	10.76391
Forces	kip force	kN	4.448
	lb	N	4.448
	kN	kip	0.2248
Stresses	ksi	MPa	6.895
	MPa	ksi	0.145
Moments	ft-kip	kN-m	1.356
	kN-m	ft-kip	0.7376
Uniform loading	kip/ft	kN/m	14.59
	kN/m	kip/ft	0.06852
	kip/ft^2	kN/m^2	47.88
	kN/m^2	kip/ft^2	0.02089
	psf	N/m^2	47.88
Angle	degree	radian	0.01745
	radian	degree	57.29579

Steel Structural Members

1. GENERAL PROVISIONS

1.1 Limits of Applicability and Terms

1.1.1 Scope and Limits of Applicability

This ASCE Standard Specification shall apply to the design of structural members cold-formed to shape from annealed and cold-rolled sheet, strip, plate, or flat bar stainless steels when used for load-carrying purposes in buildings and other statically loaded structures. It may also be used for structures other than buildings provided appropriate allowances are made for thermal and/or dynamic effects. Appendices to this Specification shall be considered as integral parts of the Specification.

This ASCE Standard supersedes the 1974 edition of the Specification for the Design of Cold-Formed Stainless Steel Structural Members issued by the American Iron and Steel Institute.

1.1.2 Terms

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

1. *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 by a web, flange, stiffening lip, intermediate stiffener, or the like.
2. *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.
3. *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.
4. *Flat-Width-to-Thickness Ratio.* The flat width of an element measured along its plane, divided by its thickness.
5. *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.
6. *Stress.* Stress as used in this Specification means force per unit area.
7. *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 1 through 5 of this Specification or its specific references.
8. *Specified Minimum Yield Strength.* The specified minimum yield strength is the lower limit of yield strength which varies with the rolling direction (transverse or longitudinal) and the type of stress (tension or compression) must be equalled or exceeded in a specification test to qualify a lot of steel for use in a cold-formed stainless steel structural member designed at that yield strength.
9. *Cold-Formed Stainless Steel Structural Members.* Cold-formed stainless 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 slit widths from 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.
10. *Load and Resistance Factor Design (LRFD).* 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.
11. *Design Strength.* Factored resistance or strength (force, moment, as appropriate), ϕR_n, provided by the structural component.
12. *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).
13. *Nominal Loads.* The magnitudes of the loads specified by the applicable code.
14. *Allowable Stress Design (ASD).* A method of proportioning structural components on the basis of working loads and allowable capacities.

1.1.3 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.

1.2 Nonconforming Shapes and Constructions

The provisions of the Specification are not intended to prevent the use of alternate shapes or constructions not specifically included herein. Such alternates shall meet the provisions of Section 6 of the Specification and be approved by the appropriate building code authority.

1.3 Material

1.3.1 Applicable Stainless Steels

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

ASTM A176-85a, Stainless and Heat-Resisting Chromium Steel Plate, Sheet, and Strip.
ASTM A240-86, Heat-Resisting Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels.
ASTM A276-85a, Stainless and Heat-Resisting Steel Bars and Shapes.
ASTM A666-84, Austenitic Stainless Steel, Sheet, Strip, Plate, and Flat Bar for Structural Applications.

NOTE: The maximum thickness for Type 409 ferritic stainless steel used in this Specification is limited to 0.15 in. (3.8 mm). The maximum thickness for Types 430 and 439 ferritic stainless steels is limited to 0.125 in. (3.2 mm).

1.3.2 Other Stainless Steels

The listing in Section 1.3.1 does not exclude the use of stainless steel ordered or produced to other than the listed specifications provided such stainless 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 1.3.3.

1.3.3 Ductility

Stainless steels not listed in Section 1.3.1 and used for structural members and connections shall comply with the following ductility requirements:

The ratio of tensile strength to yield strength in both longitudinal and transverse directions shall not be less than 1.08, and the total elongation shall not be less than 10% for a two-in. gage length standard specimen tested in accordance with ASTM A370-77. The provisions of Sections 2 through 5 of this Specification are limited to stainless steels conforming to these requirements.

1.3.4 Delivered Minimum Thickness

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

1.4 Loads

1.4.1 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.

1.4.2 Live and Snow Load

The live and 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.

1.4.3 Impact Load

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

1.4.4 Wind and Earthquake Load

Wind and 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.

1.4.5 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 structural adequacy under ponding conditions.

1.5 Structural Analysis and Design

1.5.1 Design Basis

This Specification is based on the Load and Resistance Factor Design concept, which is a method of proportioning 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 factored load combinations.

Two types of limit states are to be considered: (1) Limit state of the strength required to resist the extreme loads during the intended life of the structure; and (2) limit state of the ability of the structure to perform its intended function under normal service conditions during its life. These limit states are defined as the Limit State of Strength and the Limit State of Serviceability, respectively, in the LRFD criteria.

The Allowable Stress Design concept described in Appendix D may be used as an alternate to the Load and Resistance Factor Design method.

1.5.1.1 Design for strength. The required strengths of structural members and connections shall be determined by structural analysis for the appropriate factored load combinations given in Section 1.5.2. The design is satisfactory 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 or assemblage.

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 Section 3 for members, in Section 4 for structural assemblies, and in Section 5 for connections. Values of resistance factors ϕ are given in the appropriate sections for the limit states governing member and connection strength.

1.5.1.2 Design for serviceability. The overall structure and the individual members, connections, and connectors should be checked for serviceability.

1.5.2 Loads, Load Factors, and Load Combinations

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 shall be those stipulated in the American Society of Civil Engineers Standard, *Minimum Design Loads for Buildings and Other Structures, ANSI/ASCE 7-88.* For design purposes, the loads stipulated by the applicable code shall be taken as nominal loads.

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

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

where:

D_n = nominal dead load;
E_n = nominal earthquake load;
L_n = nominal live load;
L_{rn} = nominal roof live load;
R_{rn} = nominal roof rain load;
S_n = nominal snow load; and
W_n = nominal wind load (Exception: For wind load on individual purlins, girts, wall panels and roof decks, multiply the load factor for W_n by 0.9).

Exception: The load factor for L_n in combinations (3), (4), and (5) shall 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 (4.79 kN/m^2).

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

1.5.3 Resistance Factors

The resistance factors to be used for determining the design strengths, ϕR_n, of structural members and connections are given in Sections 3, 5, and 6, and Appendix D.

1.5.4 Yield Strength and Strength Increase from Cold Work of Forming

1.5.4.1 Yield strength. The yield strength used in design F_y shall not exceed the specified minimum yield strength, or as established in accordance with Section 6, or as increased for cold work of forming in Section 1.5.4.2.

1.5.4.2 Strength increase from cold work of forming. Except as permitted by this Section, nominal stresses shall be based on the specified properties of the flat unformed material. Utilization, for design purpose, of any increase in material strength that results from a cold-forming operation is permissible provided that the increase in strength obtained is for the kind of stress (tension or compression, transverse or longitudinal) that is to be imposed on the final product in service; and under the limitations prescribed in Sections 1.5.4.2.1 and 1.5.4.2.2.

1.5.4.2.1 Type of sections. The provisions of Section 1.5.4.2 for strength increase from cold work shall apply only to the following, regardless of whether the stress to be imposed on the member in service is in tension or compression: (1) Axially loaded members and flanges of flexural members, whose proportions are such that the quantity ρ is unity as determined according to Section 2.2 for each of the component elements of the section. This includes tubular members composed of flat elements; and (2) cylindrical tubular members in which the ratio D/t of outside diameter to wall thickness does not exceed $0.112E_o/F_y$.

1.5.4.2.2 Limitations. Application of the provisions of Section 1.5.4.2 for strength increase from cold work shall be on the following basis:

1. Mechanical properties shall be determined on the basis of full section tests, in accordance with the provisions of Section 6.4.
2. Provisions shall apply only to the following Sections of the Specification:
   3.2 Tension Members.
   3.3.1.1 Nominal Section Strength of Bending.
   3.3.1.2 Lateral Buckling Strength.
   3.4 Concentrically Loaded Compression Members.
   3.5 Combined Axial Load and Bending.
   3.6 Cylindrical Tubular Members.
   Application of all other provisions of the Specification shall be based on the properties of the unformed material.
3. The effect on mechanical properties of any welding or other applied process with potentially deleterious effect on the member shall be determined on the basis of tests of full section specimens containing, within the gage length, such welding or other intended process. Any necessary allowance for such effect shall be made in the structural design of the member.

1.5.5 Design Tables and Figures

The design tables (Tables A1 through A17) and figures (Figures A1 through A12) used in this Specification are given in Appendix A. For the design of cold-formed stainless steel members, the secant modulus, the tangent modulus, and the plasticity reduction factor can be determined by using either the tabulated values provided in Appendix A or the modified Ramberg-Osgood equation given in Appendix B.

1.6 Reference Documents

This Specification recognizes other published and latest approved specifications and manuals, when applicable, for use in designs contemplated herein, as follows:

1. American Society of Civil Engineers, ASCE 7-88, "Minimum Design Loads in Buildings and Other Structures," American Society of Civil Engineers (ASCE).
2. Applicable standards of the American Society for Testing and Materials, (ASTM), 1916 Race Street, Philadelphia, Pa. 19013.
3. Applicable Standards of the American Welding Society, (AWS), 550 N. W. LeJeune Road, Miami, Fla. 33126.

2. ELEMENTS

2.1 Dimensional Limits and Considerations

2.1.1 Flange Flat-Width-to-Thickness Considerations

1. *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:

i. Stiffened compression element having one longitudinal edge connected to a web or flange element, the other stiffened by: Simple lip. 50
   Any other kind of stiffener having $I_s > I_a$ and $D/w < 0.8$ according to Section 2.4.2. 90
ii. Stiffened compression element with both longitudinal edges connected to other stiffened elements. 400
iii. Unstiffened compression element and elements with edge stiffener having $I_s < I_a$ and $D/w \leq 0.8$ according to Section 2.4.2. 50

NOTE: Unstiffened compression elements with w/t ratios larger than approximately 30 and stiffened compression elements that have w/t ratios exceeding approximately 75 are likely to develop noticeable out-of-plane distortions at the design load. These distortions do not impair the load-carrying capacity of the element; however, when it is necessary to minimize or prevent visible distortions for elements with larger w/t ratios, Sections 3.3.1.1 and 3.4 stipulate the design requirements of local distortion for flexural and compression members, respectively.

Stiffened elements having w/t ratios larger than 400 may be used with safety to support loads, but substantial deformation of such elements under load may occur and may render inapplicable the design formulas of this Specification.

2. *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 the compression and tension flanges, either stiffened or unstiffened:

$$w_f = \sqrt{\frac{0.061tdE_o}{f_{av}}} \times \sqrt[4]{\frac{100c_f}{d}} \qquad (2.1.1\text{-}1)$$

where:

w_f = width of flange projecting beyond web; or half of the distance between webs for box- or U-type beams;
t = flange thickness;
c_f = amount of curling;*
d = depth of beam;
E_o = initial modulus of elasticity, as given in Tables A4 and A5; and
f_{av} = average stress in full, unreduced flange width. (Where members are designed by effective design width procedure, the average stress equals the maximum stress multiplied by the ratio of the effective design width to the actual width.)

3. *Shear Lag Effects—Unusually Short Spans Supporting Concentrated Loads.* Where the span of the beam is less than $30w_f$ (w_f as presently defined), 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 1. 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 distance between inflection points for continuous beams; or twice the length of cantilever beams; and w_f = width of flange projection beyond web for I-beam and similar sections or half 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.

2.1.2 Maximum Web Depth-to-Thickness Ratio

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

1. For unreinforced webs: $(h/t)_{max} = 200$
2. For webs which are provided with transverse stiffeners satisfying the requirements of Appendix C.1:
   i. When using bearing stiffeners only, $(h/t)_{max} = 260$
   ii. When using bearing stiffeners and intermediate stiffeners, $(h/t)_{max} = 300$

In the foregoing:

h = depth of flat portion of web measured along plane of web.
t = web thickness.

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

2.2 Effective Widths of Stiffened Elements

2.2.1 Uniformly Compressed Stiffened Elements

1. *Load Capacity Determination.* The effective widths, *b,* of uniformly compressed elements shall be determined from the following formulas:

$$b = w \quad \text{when} \quad \lambda \leq 0.673 \qquad (2.2.1\text{-}1)$$

$$b = \rho w \quad \text{when} \quad \lambda > 0.673 \qquad (2.2.1\text{-}2)$$

where:

w = flat width as shown in Figure 1

$$\rho = \frac{1\text{-}0.22/\lambda}{\lambda} \qquad (2.2.1\text{-}3)$$

λ is a slenderness factor determined as follows:

$$\lambda = \left(\frac{1.052}{\sqrt{k}}\right)\left(\frac{w}{t}\right)\left(\sqrt{\frac{f}{E_o}}\right) \qquad (2.2.1\text{-}4)$$

t = thickness of the uniformly compressed stiffened elements.

* 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.

where:

f for load capacity determination is as follows:
For flexural members:

i. If Procedure I of Section 3.3.1.1 is used, $f = F_{yc}$ if the initial yielding is in compression in the element considered. If the initial yielding is not in compression in the element considered, then the stress f shall be determined for the element considered on the basis of the effective section at M_y (moment causing initial yield).

ii. If procedure II of Section 3.3.1.1 is used, then f is the stress in the element considered at M_n determined on the basis of the effective section.

iii. If Section 3.3.1.2 is used, then the $f = M_c/S_f$ as described in that Section in determining S_c.

For compression members f is taken equal to F_n as determined in Section 3.4.

E_o = initial modulus of elasticity as given in Tables A4 and A5
k = plate buckling coefficient
= 4.0 for stiffened elements supported by a web on each longitudinal edge. Values for stiffened elements with an edge stiffener or one intermediate stiffener are given in Section 2.4.

2. *Deflection Determination.* The effective widths, b_d, used in computing deflection shall be determined from the following formulas:

$$b_d = w \quad \text{when} \quad \lambda \leq 0.673 \qquad (2.2.1\text{-}5)$$

$$b_d = \rho w \quad \text{when} \quad \lambda > 0.673 \qquad (2.2.1\text{-}6)$$

where:

w = flat width
ρ = reduction factor determined from Eqs. 2.2.1-3 and 2.2.1-4, except that f_d is substituted for f, where f_d = computed compression stress in element being considered, and that reduced modulus of elasticity, E_r, shall be substituted for E_o in Eq. 2.2.1-4.

where:

$$E_r = \frac{E_{st} + E_{sc}}{2} \qquad (2.2.1\text{-}7)$$

E_{st} = Secant modulus corresponding to stress in tension flange; and
E_{sc} = Secant modulus corresponding to stress in compression flange.

Values of the secant moduli may be obtained from Tables A2 and A3, Figures A1 and A2 of Appendix A, or determined by using Eq. B-1 in Appendix B.

2.2.2 Effective Widths of Webs and Stiffened Elements with Stress Gradient

1. *Load Capacity Determination.* The effective widths, b_1 and b_2, as shown in Figure 2 shall be determined from the following formulas:

$$b_1 = \frac{b_e}{3 - \psi} \qquad (2.2.2\text{-}1)$$

For $\psi \leq -0.236$

$$b_2 = \frac{b_e}{2} \qquad (2.2.2\text{-}2)$$

$b_1 + b_2$ shall not exceed compression portion of web calculated on the basis of effective section.
For $\psi > -0.236$

$$b_2 = b_e - b_1 \qquad (2.2.2\text{-}3)$$

where:

b_e = effective width b determined in accordance with Section 2.2.1 with f_1 substituted for f and with k determined as follows:
$k = 4 + 2(1 - \psi)^3 + 2(1 - \psi) \qquad (2.2.2\text{-}4)$
$\psi = f_2/f_1$

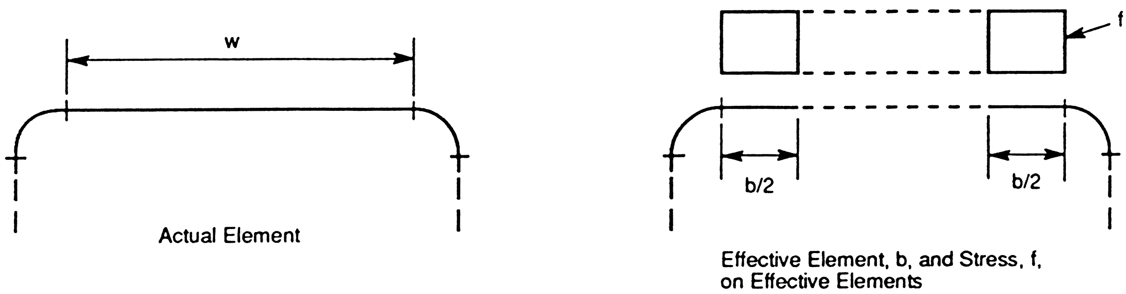

FIGURE 1. Stiffened Elements with Uniform Compression

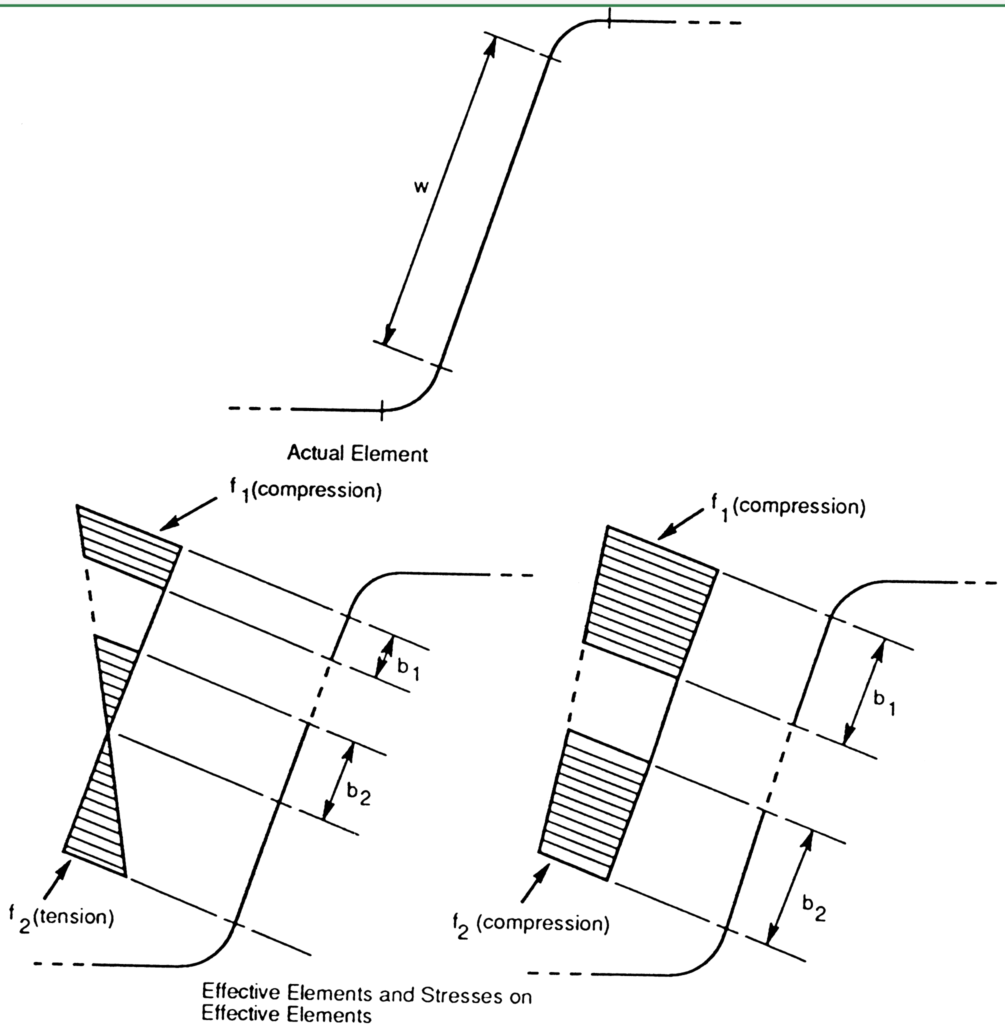

FIGURE 2. Stiffened Elements with Stress Gradient and Webs

f_1, f_2 = stresses shown in Figure 2 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, $f_1 \geq f_2$.

2. *Deflection Determination.* The effective widths in computing deflections at a given load shall be determined in accordance with Section 2.2.2(1) 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 2. Calculations are based on the effective section at the load for which deflections are determined.

2.3 Effective Widths of Unstiffened Elements

2.3.1 Uniformly Compressed Unstiffened Elements

1. *Load Capacity Determination.* Effective widths, *b,* of unstiffened compression elements with uniform compression shall be determined in accordance with Section 2.2.1(1) with the exception that k shall be taken as 0.5 and w as defined in Figure 3.

2. *Deflection Determination.* The effective widths used in computing deflection shall be determined in accordance with Section 2.2.1(2), except that f_d is substituted for f and $k = 0.5$.

2.3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

1. *Load Capacity Determination.* Effective widths, *b,* of unstiffened compression elements and edge stiffeners with stress gradient shall be determined in accordance with Section 2.2.1(1) with $f = f_3$ as shown in Figure 5 in the element and $k = 0.5$.

2. *Deflection Determination.* Effective widths, *b,* of unstiffened compression elements and edge stiffeners with stress gradient shall be determined in accor-

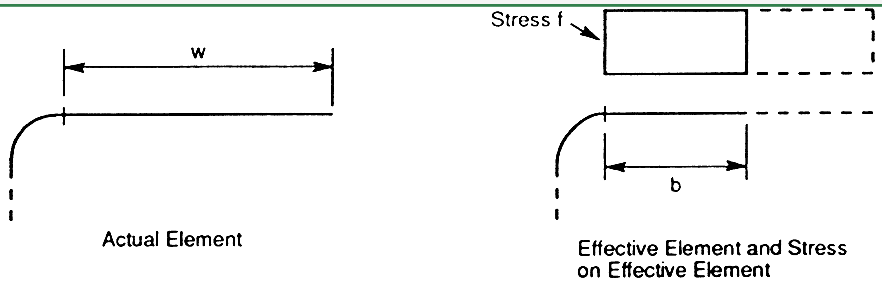

FIGURE 3. Unstiffened Elements with Uniform Compression

dance with Section 2.2.1(2) except that f_{d3} is substituted for f and $k = 0.5$.

2.4 Effective Widths of Elements with Edge Stiffener or One Intermediate Stiffener

The following notation is used in this section:

$$S = 1.28\sqrt{E_o/f}; \tag{2.4-1}$$

k = buckling coefficient;

b_o = dimension defined in Figure 4;

d, w, D = dimensions defined in Figure 5;

d_s = reduced effective width of stiffener as specified in this section, with d_s, calculated according to Section 2.4.2, to be used in computing overall effective section properties (see Figure 5);

d_s' = effective width of stiffener calculated according to Section 2.3.1 (see Figure 5);

C_1, C_2 = coefficients defined in Figures 4 and 5; and

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

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

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

For the stiffener shown in Figure 5.

$$I_s = (d^3 t \sin^2\theta)/12; \tag{2.4-2}$$

$$A_s' = d_s' t \tag{2.4-3}$$

2.4.1 Uniformly Compressed Elements with Intermediate Stiffener

1. *Load Capacity Determination.*

Case I: $\dfrac{b_o}{t} \le S$ (2.4.1-1)

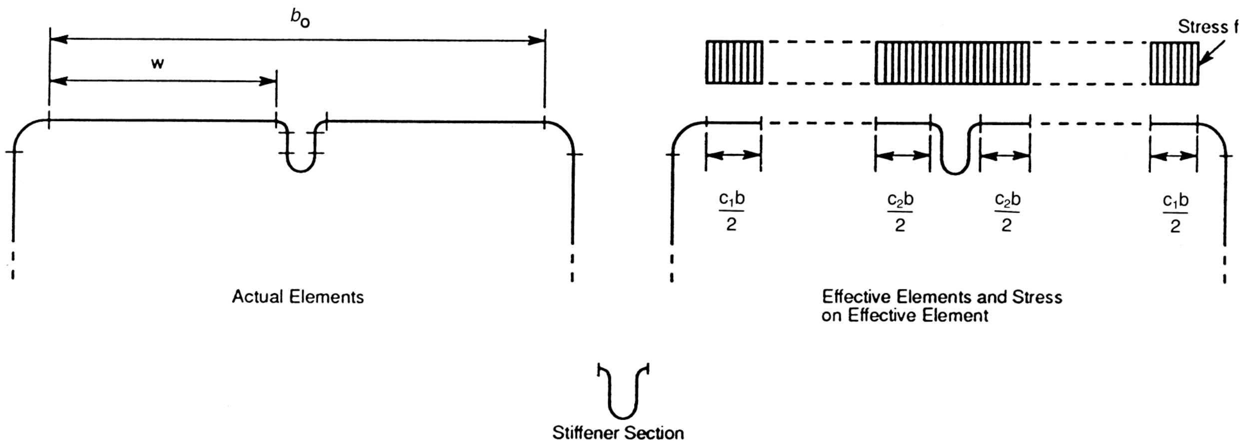

FIGURE 4. Elements with Intermediate Stiffener

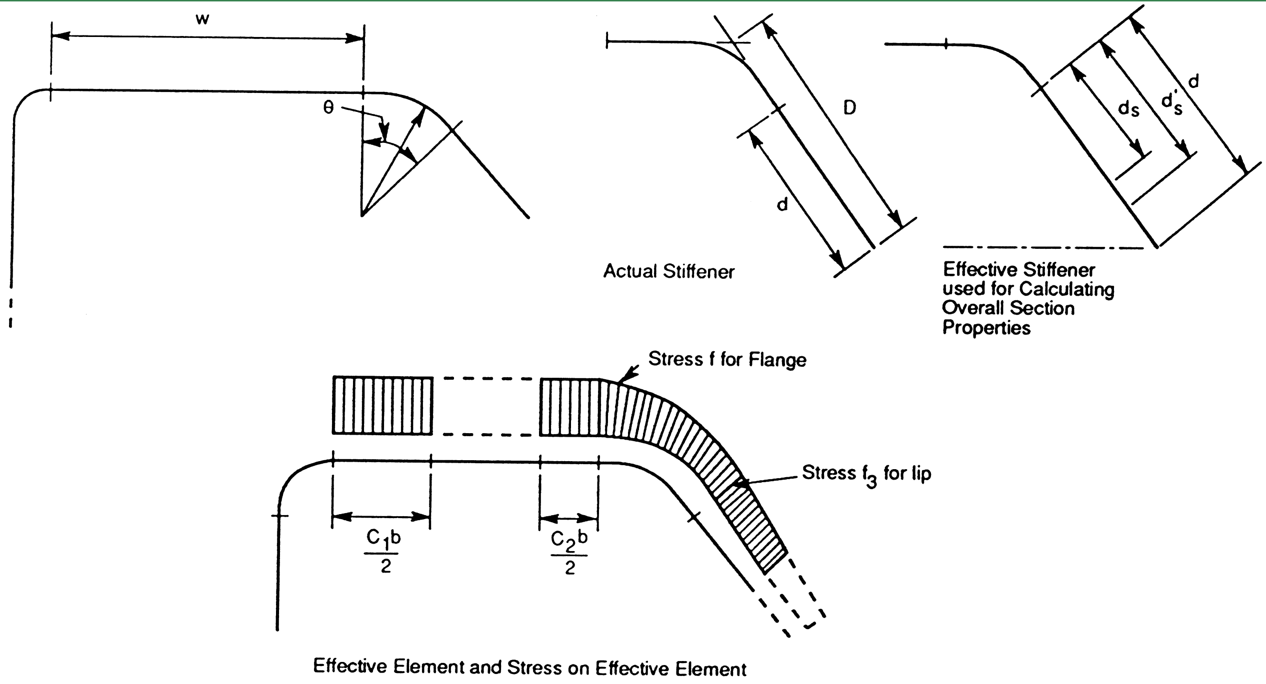

FIGURE 5. Elements with Edge Stiffener

$I_a = 0$ (no intermediate stiffener needed) (2.4.1-2)

$b = w$ (2.4.1-3)

$A_s = A'_s$ (2.4.1-4)

Case II: $S < \dfrac{b_o}{t} < 3S$ (2.4.1-5)

$$\frac{I_a}{t^4} = \frac{50(b_o/t)}{S} - 50 \quad (2.4.1\text{-}6)$$

b and A_s are calculated according to Section 2.2.1(1)

where:

$$k = 3\left(\frac{I_s}{I_a}\right)^{1/2} + 1 \leq 4 \quad (2.4.1\text{-}7)$$

$$A_s = A'_s\left(\frac{I_s}{I_a}\right) \leq A'_s \quad (2.4.1\text{-}8)$$

Case III: $\dfrac{b_o}{t} \geq 3S$

$$\frac{I_a}{t^4} = \frac{128(b_o/t)}{S} - 285 \quad (2.4.1\text{-}9)$$

b and A_s are calculated according to Section 2.2.1(1), where

$$k = 3\left(\frac{I_s}{I_a}\right)^{1/3} + 1 \leq 4 \quad (2.4.1\text{-}10)$$

$$A_s = A'_s\left(\frac{I_s}{I_a}\right) \leq A'_s \quad (2.4.1\text{-}11)$$

2. *Deflection Determination.* Effective widths shall be determined as in Section 2.4.1(1) except that f_d is substituted for f.

2.4.2 Uniformly Compressed Elements with Edge Stiffener

1. *Load Capacity Determination.*

Case I: $\dfrac{w}{t} \leq \dfrac{S}{3}$ (2.4.2-1)

$I_a = 0$ (no edge stiffener needed) (2.4.2-2)

$b = w$ (2.4.2-3)

$d_s = d'_s$ for simple lip stiffener (2.4.2-4)

$A_s = A'_s$ for other stiffener shapes (2.4.2-5)

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

$$\frac{I_a}{t^4} = 399\left(\frac{w/t}{S} - \sqrt{\frac{k_u}{4}}\right)^3$$
$$n = \frac{1}{2} \qquad (2.4.2\text{-}6)$$

$$C_2 = \frac{I_s}{I_a} \leq 1 \qquad (2.4.2\text{-}7)$$

$$C_1 = 2 - C_2 \qquad (2.4.2\text{-}8)$$

b shall be calculated according to Section 2.2.1 where

$$k = C_2^n(k_a - k_u) + k_u$$
$$k_u = 0.43 \qquad (2.4.2\text{-}9)$$

For simple lip stiffener with $140° \geq \theta \geq 40°$ and $D/w \leq 0.8$ where θ is as shown in Fig. 5

$$k_a = 5.25 - 5\,(D/w) \leq 4.0 \qquad (2.4.2\text{-}10)$$

$$d_s = C_2 d_s' \qquad (2.4.2\text{-}11)$$

For stiffener shape other than simple lip:

$$k_a = 4.0$$
$$A_s = C_2 A_s' \qquad (2.4.2\text{-}12)$$

Case III: $w/t \geq S$

$$\frac{I_a}{t^4} = \frac{115\,(w/t)}{S} + 5 \qquad (2.4.2\text{-}13)$$

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

2. *Deflection Determination.* Effective widths shall be determined as in Section 2.4.2(1) except that f_d is substituted for f.

2.5 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:

$$I_{\min} = \left(3.66\sqrt{\left(\frac{w}{t}\right)^2 - \frac{0.119E_o}{F_y}}\right)t^4 \qquad (2.5\text{-}1)$$

but not less than $18.4t^4$

where:

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

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

1. If the spacing of intermediate stiffeners between two webs is such that for the sub-element between stiffeners $b < w$ as determined in Section 2.2.1, only two intermediate stiffeners (those nearest each web) shall be considered effective.
2. 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 2.2.1, only one intermediate stiffener, that nearest the web, shall be considered effective.
3. If intermediate stiffeners are spaced so closely that for the elements between stiffeners $b = w$ as determined in Section 2.2.1, 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:

$$t_s = \sqrt[3]{\frac{12I_{sf}}{b_o}} \qquad (2.5\text{-}2)$$

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.

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

$$\frac{b_e}{t} = \frac{b}{t} - 0.10\left(\frac{w}{t} - 60\right) \qquad (2.5\text{-}3)$$

where:

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

b = effective design width determined in accordance with provisions of Section 2.2.1; and
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 subjected to the reduction noted in effective width, the area of stiffeners (edge stiffener or intermediate stiffeners) shall be considered reduced to an effective area as follows:

For $60 < \frac{w}{t} < 90$:

$$A_{ef} = \alpha A_{st} \qquad (2.5\text{-}4)$$

where:

$$\alpha = \left(3 - \frac{2b_e}{w}\right) - \left(\frac{1}{30}\right)\left(1 - \frac{b_e}{w}\right)\left(\frac{w}{t}\right) \qquad (2.5\text{-}5)$$

For $\frac{w}{t} \geq 90$:

$$A_{ef} = \frac{b_e}{w} A_{st} \qquad (2.5\text{-}6)$$

In these 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.

2.6 Stiffeners

Provisions for the design of transverse stiffeners and shear stiffeners are given in Appendix C.

3. MEMBERS

3.1 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.

3.2 Tension Members

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

$$\phi_t = 0.85$$
$$T_n = A_n F_y \qquad (3.2\text{-}1)$$

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; and
F_y = specified yield strength as given in Table A1.

When mechanical fasteners are used in connections for tension members, the design tensile strength shall also be limited by Section 5.3.2.

3.3 Flexural Members

3.3.1 Strength for Bending Only

For flexural members subjected only to bending moment, the design flexural strength, $\phi_b M_n$, shall be the smaller of the values calculated according to Sections 3.3.1.1 and 3.3.1.2.

3.3.1.1 Nominal section strength. The design flexural strength, $\phi_b M_n$, shall be determined with $\phi_b = 0.90$ for sections with stiffened and partially stiffened compression flanges, and $\phi_b = 0.85$ for sections with unstiffened compression flanges, and the nominal section strength, M_n, is 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.

1. *Procedure I—Based on Initiation of Yielding.* The effective yield moment, M_n, based on section strength shall be determined as follows:

$$M_n = S_e F_y \qquad (3.3.1.1\text{-}1)$$

where:

F_y = specified yield strength in compression, F_{yc}, or in tension, F_{yt}, as given in Table A1;
S_e = elastic section modulus of effective section calculated with the extreme compression fiber at F_{yc} or extreme tension fiber at F_{yt}, whichever initiates yielding first.

2. *Procedure II—Based on Inelastic Reserve Capacity.* The inelastic reserve capacity of flexural members may be used when the following conditions are met:

i. The member is not subject to twisting or to lateral, torsional, or torsional-flexural buckling.

ii. The effect of cold-forming is not included in determining the yield strength F_y.
iii. The ratio of the depth of the compressed portion of the web to its thickness does not exceed λ_1.
iv. The shear force does not exceed $0.35F_y$ times the web area, $(h)(t)$.
v. The angle between any web and the vertical does not exceed 30°.

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

e_y = yield strain = F_y/E_o;
E_o = initial modulus of elasticity (Tables A4 and A5); and
C_y = compression strain factor determined as follows:

(a) Stiffened compression elements without intermediate stiffeners

$$C_y = 3 \quad \text{for} \quad \frac{w}{t} \le \lambda_1$$

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

$$C_y = 1 \quad \text{for} \quad \frac{w}{t} \ge \lambda_2$$

where:

$$\lambda_1 = \frac{1.11}{\sqrt{\dfrac{F_{yc}}{E_o}}} \tag{3.3.1.1-2}$$

$$\lambda_2 = \frac{1.28}{\sqrt{\dfrac{F_{yc}}{E_o}}} \tag{3.3.1.1-3}$$

(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 in tension and 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 3.3.5.

3. *Local Distortion Consideration.* When local distortions in flexural members under nominal service loads must be limited, the design flexural strength, $\phi_d M_{ld}$, shall be determined by the following equations:

$$\phi_d = 1.0$$
$$M_{ld} = S_f f_b \tag{3.3.1.1-4}$$

where:

S_f = elastic section modulus of the full, unreduced cross section;
f_b = permissible compressive stress for local distortion, determined as follows. In no case shall the permissible stress f_b exceed the yield strength F_y.

i. If small, barely perceptible amounts of local distortions are allowed:
For stiffened compression elements

$$f_b = 1.2\, F_{cr} \tag{3.3.1.1-5}$$

For unstiffened compression elements

$$f_b = F_{cr} \tag{3.3.1.1-6}$$

ii. If no local distortions are permissible:
For stiffened compression elements

$$f_b = 0.9\, F_{cr} \tag{3.3.1.1-7}$$

For unstiffened compression elements

$$f_b = 0.75\, F_{cr} \tag{3.3.1.1-8}$$

where:

F_{cr} = critical buckling stress

$$= \frac{\pi^2 k \eta E_o}{12(1-\mu^2)(w/t)^2} \tag{3.3.1.1-9}$$

η = Plasticity reduction factor corresponding to compression stress;

$\sqrt{\dfrac{E_t}{E_o}}$, for stiffened compression elements

= as given in Tables A6 and A7, or Figures A3 and A4 of Appendix A, or determined by using Eq. B-3 in Appendix B

$= E_s/E_o$, for unstiffened compression elements as given in Tables A8 and A9, or Figures A5 and A6 of Appendix A or determined by using Eq. B-4 in Appendix B

k = plate-buckling coefficient as defined in Section 2

μ = Poisson's ratio in the elastic range = 0.3

E_o = initial modulus of elasticity (Tables A4 and A5)

3.3.1.2 Lateral buckling strength. The design strength of the laterally unbraced segments of doubly or singly symmetric sections* subjected to lateral buckling, $\phi_b M_n$, shall be determined with $\phi_b = 0.85$ and M_n calculated as follows:

$$M_n = S_c(M_c/S_f) \qquad (3.3.1.2\text{-}1)$$

where:

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

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

M_c is the critical moment calculated according to (1), (2), or (3) with a maximum value of M_y:

1. For doubly symmetric I-sections bent about the centroidal axis perpendicular to the web (x-axis):

$$M_c = \pi^2 E_o C_b \left(\frac{E_t}{E_o}\right)\left(\frac{dI_{yc}}{L^2}\right) \qquad (3.3.1.2\text{-}2)$$

Alternatively, M_c can be calculated by using Eq. 3.3.1.2-4.

2. For point-symmetric Z-sections bent about the centroidal axis perpendicular to the web (x-axis):

$$M_c = 0.5\pi^2 E_o C_b \left(\frac{E_t}{E_o}\right)\left(\frac{dI_{yc}}{L^2}\right) \qquad (3.3.1.2\text{-}3)$$

Alternatively, M_c can be calculated as half the value given in Eq. 3.3.1.2-4 with a maximum value of M_y.

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

   i. For bending about the symmetry axis (x-axis is the axis of symmetry oriented such that the shear center has a negative x-coordinate):

$$M_c = C_b r_o A \sqrt{\sigma_{ey}\sigma_t} \qquad (3.3.1.2\text{-}4)$$

   Alternatively, M_c can be calculated by using Eq. 3.3.1.2-2 for doubly symmetric I-sections given in (1).

   ii. For bending about the centroid axis perpendicular to the symmetry axis:

$$M_c = C_s C_b A \sigma_{ex} \times \left(j + C_s\sqrt{j^2 + r_o^2 \frac{\sigma_t}{\sigma_{ex}}}\right) \qquad (3.3.1.2\text{-}5)$$

In the foregoing:

M_y = moment causing initial yield at the extreme compression fiber of full section;
$= S_f F_y$;

L = unbraced length of member;

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

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

C_s = − 1 for moment causing tension on shear center side of centroid;

$$\sigma_{ex} = \left[\frac{\pi^2 E_o}{(K_x L_x/r_x)^2}\right]\left(\frac{E_t}{E_o}\right) \qquad (3.3.1.2\text{-}6)$$

$$\sigma_{ey} = \left[\frac{\pi^2 E_o}{(K_y L_y/r_y)^2}\right]\left(\frac{E_t}{E_o}\right) \qquad (3.3.1.2\text{-}7)$$

$$\sigma_t = \left(\frac{1}{A r_o^2}\right)\left(G_o J + \frac{\pi^2 E_o C_w}{(K_t L_t)^2}\right)\left(\frac{E_t}{E_o}\right) \qquad (3.3.1.2\text{-}8)$$

A = full cross-sectional area;

E_o = initial modulus of elasticity (Tables A4 and A5);

E_t/E_o = plasticity reduction factor corresponding to stress, as given in Tables A10 and A11 or Figures A7 and A8, or determined by using Eq. B-5 in Appendix B.

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

$$C_b = \frac{12.5 M_{\max}}{2.5 M_{\max} + 3M_A + 4M_B + 3M_C}$$

* 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.

where:

M_{max} = absolute value of maximum moment in the unbraced segment
M_A = absolute value of moment at quarter point of unbraced segment
M_B = absolute value of moment at centerline of unbraced segment
M_C = absolute value of moment at three-quarter point of unbraced segment

C_b is permitted to be conservatively taken as unity for all cases. For cantilevers or overhangs where the free end is unbraced, C_b shall be taken as unity. For members subject to combined axial load and bending moment (Section 3.5), C_b shall be taken as unity.

d = depth of section;
r_o = polar radius of gyration of cross section about shear center:

$$= \sqrt{r_x^2 + r_y^2 + x_o^2} \tag{3.3.1.2-9}$$

r_x, r_y = radii of gyration of cross section about centroidal principal axes;
G_o = initial shear modulus (Tables A4 and A5):
K_x, K_y = effective length factors for bending about x- and y-axes, respectively;
K_t = effective length factor for twisting;
L_x, L_y = unbraced lengths of compression member for bending about x- and y-axes, respectively;
L_t = unbraced length of compression member for twisting;
x_o = distance from shear center to centroid along principal x-axis, taken as negative;
J = St. Venant torsion constant of cross section:
C_w = torsional warping constant of cross section:

$$j = \frac{1}{2I_y}\left(\int_A x^3 dA + \int_A xy^2 dA\right) - x_o \tag{3.3.1.2-10}$$

3.3.2 Strength for Shear Only

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

$$\phi_v = 0.85$$
$$V_n = \frac{4.84 E_o t^3 (G_s/G_o)}{h} \tag{3.3.2-1}$$

In no cases shall $\phi_v V_n$ exceed $(0.95)(F_{yv}\,ht)$.
where:

ϕ_v = resistance factor for shear;
V_n = nominal shear strength of beam;
t = web thickness;
h = depth of flat portion of web measured along plane of web;
G_s/G_o = plasticity reduction factor corresponding to shear stress, as given in Tables A12 or Figures A9 and A10; and
F_{yv} = specified shear yield strength as given in Table A1.

When the web consists of two or more sheets, each sheet shall be considered as a separate element carrying its share of the shear force.
Exception: For beam webs with transverse stiffeners satisfying the requirements of Appendix C, the nominal shear strength shall be calculated as $V_n = 0.904 k_v E_o t^3 (G_s/G_o)/h$, where k_v is determined in accordance with Section C2.

3.3.3 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:

$$\left(\frac{M_u}{\phi_b M_n}\right)^2 + \left(\frac{V_u}{\phi_v V_n}\right)^2 \le 1.0 \tag{3.3.3-1}$$

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 $M_u/(\phi_b M_n) > 0.5$ and $V_u/(\phi_v V_n) > 0.7$, then M_u and V_u shall satisfy the following interaction equation:

$$0.6\frac{M_u}{\phi_b M_n} + \frac{V_u}{\phi_v V_n} \le 1.3 \tag{3.3.3-2}$$

In these equations:

ϕ_b = resistance factor for bending (See Section 3.3.1);
ϕ_v = resistance factor for shear (See Section 3.3.2);
M_n = nominal flexural strength when bending alone exists, based on Section 3.3.1.1; and
V_n = nominal shear strength when shear alone exists.

3.3.4 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 concentrated loads and reactions shall not exceed the values of $\phi_w P_n$, with $\phi_w = 0.70$ for single unreinforced webs and I-sections, and P_n given in Table 2. 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 2 apply when $N/t \le 210$, $N/h \le 3.5$, and for beams with $R/t \le 6$ and deck with $R/t \le 7$.

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.

$$t^2 C_3 C_4 C_\theta \left(331 - 0.61 \frac{h}{t}\right) \times \left(1 + 0.01 \frac{N}{t}\right) \times C_t \qquad (3.3.4\text{-}1)$$

$$t^2 C_3 C_4 C_\theta \left(217 - 0.28 \frac{h}{t}\right) \times \left(1 + 0.01 \frac{N}{t}\right) \times C_t \qquad (3.3.4\text{-}2)$$

when $\frac{N}{t} < 60$, the factor $\left(1 + 0.01 \frac{N}{t}\right)$ may be increased to $\left(0.71 + 0.015 \frac{N}{t}\right)$

$$t^2 F_y C_6 \left(10 + 1.25 \sqrt{\frac{N}{t}}\right) \qquad (3.3.4\text{-}3)$$

$$t^2 C_1 C_2 C_\theta \left(538 - 0.74 \frac{h}{t}\right) \times \left(1 + 0.007 \frac{N}{t}\right) \times C_t \qquad (3.3.4\text{-}4)$$

when $\frac{N}{t} > 60$, the factor $\left(1 + 0.007 \frac{N}{t}\right)$ may be increased to $\left(0.75 + 0.011 \frac{N}{t}\right)$

$$t^2 F_y C_5 (0.88 + 0.12m) \times \left(15 + 3.25 \sqrt{\frac{N}{t}}\right) \qquad (3.3.4\text{-}5)$$

TABLE 2. Nominal Web Crippling Strength, P_n

		Shapes Having Single Webs: Stiffened or partially stiffened flanges	Shapes Having Single Webs: Unstiffened flanges	I-Sections or Similar Sections[a]: Stiffened, partially stiffened, and unstiffened flanges
Opposing loads spaced $> 1.5h$[b]	End reaction[c]	Eq. 3.3.4-1	Eq. 3.3.4-2	Eq. 3.3.4-3
	Interior reaction[d]	Eq. 3.3.4-4	Eq. 3.3.4-4	Eq. 3.3.4-5
Opposing loads spaced $\le 1.5h$[e]	End reaction[c]	Eq. 3.3.4-6	Eq. 3.3.4-6	Eq. 3.3.4-7
	Interior reaction[d]	Eq. 3.3.4-8	Eq. 3.3.4-8	Eq. 3.3.4-9

[a] I-sections made of two channels connected back-to-back, or similar sections which provide high degree of restraint against rotation of web (such as I-sections made by welding two angles to channel).
[b] At locations of one concentrated load or reaction acting either on top or bottom flange, when clear distance between bearing edges of this and adjacent opposite concentrated loads or reactions is greater than $1.5h$.
[c] For end reactions of beams or concentrated loads on end of cantilevers when distance from edge of bearing to end of beam is less than $1.5h$.
[d] For reactions and concentrated loads when distance from edge of bearing to end of beam is equal to or greater than $1.5h$.
[e] At locations of two opposite concentrated loads or of concentrated load and opposite reaction acting simultaneously on top and bottom flanges, when clear distance between their adjacent bearing edges is equal to or less than $1.5h$.

$$t^2 C_3 C_4 C_\theta \times \left(244 - 0.57\frac{h}{t}\right) \times \left(1 + 0.01\frac{N}{t}\right) \times C_t \quad (3.3.4\text{-}6)$$

$$t^2 F_y C_8 (0.64 + 0.31m) \times \left(10 + 1.25\sqrt{\frac{N}{t}}\right) \quad (3.3.4\text{-}7)$$

$$t^2 C_1 C_2 C_\theta \times \left(771 - 2.26\frac{h}{t}\right) \times \left(1 + 0.0013\frac{N}{t}\right) \times C_t \quad (3.3.4\text{-}8)$$

$$t^2 F_y C_7 (0.82 + 0.15m) \times \left(15 + 3.25\sqrt{\frac{N}{t}}\right) \quad (3.3.4\text{-}9)$$

In these formulas:

ϕ_w = resistance factor for web crippling;
P_n = nominal strength for concentrated load or reaction per web (kips for US Customary Units and N for SI units)
C_t = 1.0 for US Customary Units
C_t = 6.9 for SI Units
C_1 = $(1.22 - 0.22k)k$, when $F_y/(91.5C_t) \le 1.0$ (3.3.4-10)
= 1.69 when $F_y/(91.5C_t) > 1.0$

$$C_2 = \left(1.06 - \frac{0.06R}{t}\right) \le 1.0 \quad (3.3.4\text{-}11)$$

C_3 = $(1.33 - 0.33k)k$ when $F_y/66.5C_t \le 1.0$
= 1.34 when $F_y/66.5C_t > 1.0$ (3.3.4-12)
C_4 = $(1.15 - 0.15R/t) \le 1.0$ but not less than 0.50 (3.3.4-13)
C_5 = $(1.49 - 0.53k) \ge 0.6$ (3.3.4-14)

$$C_6 = 1 + \frac{h/t}{750}, \text{ when } h/t \le 150 \quad (3.3.4\text{-}15)$$

$$= 1.20, \text{ when } \frac{h}{t} > 150 \quad (3.3.4\text{-}16)$$

$$C_7 = 1/k \text{ when } \frac{h}{t} \le 66.5 \quad (3.3.4\text{-}17)$$

$$= \left(1.10 - \frac{h/t}{665}\right)\left(\frac{1}{k}\right) \text{ when } \frac{h}{t} > 66.5 \quad (3.3.4\text{-}18)$$

$$C_8 = \left(0.98 - \frac{(h/t)}{865}\right)\left(\frac{1}{k}\right) \quad (3.3.4\text{-}19)$$

$$C_\theta = 0.7 + 0.3\left(\frac{\theta}{90}\right)^2 \quad (3.3.4\text{-}20)$$

F_y = specified yield strength in longitudinal compression, (ksi for US Customary Units and MPa for SI Units)
h = depth of flat portion of web measured along plane of web.

$$k = \frac{F_y}{33C_t} \quad (3.3.4\text{-}21)$$

m = $t/0.075$ for US Customary Units
= $t/1.91$ for SI Units (3.3.4-22)
t = web thickness.
N = actual length of bearing. For case of two equal and opposite concentrated loads distributed over unequal bearing lengths, smaller value of N shall be taken;
R = inside bend radius;
θ = angle between plane of web and plane of bearing surface $\ge 45°$, but not more than 90°.

In the above formulas the units of h, t, N, and R are in inches and in mm for US Customary Units and SI Units, respectively.

3.3.5 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:

1. For shapes having single unreinforced webs:

$$\frac{1.07\,P_u}{\phi_w P_n} + \frac{M_u}{\phi_b M_n} \le 1.42 \quad (3.3.5\text{-}1)$$

Exception: At the interior supports of continuous spans, this 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 in. (254 mm).

2. 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);

$$\frac{0.82\,P_u}{\phi_w P_n} + \frac{M_u}{\phi_b M_n} \le 1.32 \quad (3.3.5\text{-}2)$$

Exception: When $h/t \le 2.33/\sqrt{(F_y/E_o)}$ and $\lambda \le 0.673$, the nominal concentrated load or reaction may be determined by Section 3.3.4. In these formulas:

ϕ_b = resistance factor for bending (see Section 3.3.1);
ϕ_w = resistance factor for web crippling (see Section 3.3.4);

P_u = required strength for concentrated load or reaction in presence of bending moment;
P_n = nominal strength for concentrated load or reaction in absence of bending moment determined in accordance with Section 3.3.4;
M_u = required flexural strength at, or immediately adjacent to, point of application of concentrated load or reaction P_u;
M_n = nominal flexural strength determined according to Section 3.3.1.1, if bending alone exists;
w = flat width of beam flange which contacts bearing plate;
t = thickness of web or flange; and
λ = slenderness factor given by Section 2.2.1.

3.4 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.

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

$$\phi_c = 0.85$$
$$P_n = A_e F_n \qquad (3.4\text{-}1)$$

where:

A_e = effective area calculated at stress F_n.
F_n = the least of the flexural, torsional, and torsional-flexural buckling stress determined according to Sections 3.4.1 through 3.4.4.

2. When local distortions in compression members under service loads must be limited, the design axial strength, $\phi_d P_{id}$, shall be determined by the following equations:

$$\phi_d = 1.0$$
$$P_{ld} = A f_b \qquad (3.4\text{-}2)$$

in which A = the area of the full, unreduced cross section; and f_b = the permissible compressive stresses given in Eqs. 3.3.1.1-5 through 3.3.1.1-8.

3. 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.

4. 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.

3.4.1 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 flexural buckling stress, F_n, shall be determined as follows:

$$F_n = \frac{\pi^2 E_t}{(KL/r)^2} \le F_y \qquad (3.4.1\text{-}1)$$

where:

E_t = tangent modulus in compression corresponding to buckling stress, as given in Tables A13 and A14 or Figures A11 and A12, or determined by using Eq. B-2 in Appendix B;
K = effective length factor*
L = unbraced length of member; and
r = radius of gyration of full, unreduced cross section.

3.4.2 Doubly or Point-Symmetric Sections Subject to Torsional Buckling

For doubly or point-symmetric sections which may be subject to torsional buckling, F_n shall be taken as the smaller of F_n calculated according to Section 3.4.1 and F_n calculated as follows:

$$F_n = \sigma_t = \left(\frac{1}{Ar_o^2}\right)\left(G_o J + \frac{\pi^2 E_o C_w}{(K_t L_t)^2}\right)\left(\frac{E_t}{E_o}\right) \qquad (3.4.2\text{-}1)$$

where σ_t is defined in Section 3.3.1.2.

3.4.3 Singly Symmetric Sections Subject to Torsional-Flexural Buckling

For sections subject to torsional-flexural buckling, F_n shall be taken as the smaller of F_n calculated according to Section 3.4.1 and F_n calculated as follows:

$$F_n = \frac{1}{2\beta}\left(\sigma_{ex} + \sigma_t - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\right) \qquad (3.4.3\text{-}1)$$

* 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.

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

$$F_n = \frac{\sigma_t \sigma_{ex}}{\sigma_t + \sigma_{ex}} \qquad (3.4.3\text{-}2)$$

where σ_t is defined in Eq. (3.4.2-1); and

$$\sigma_{ex} = \left(\frac{\pi^2 E_o}{(K_x L_x / r_x)^2}\right)\left(\frac{E_t}{E_o}\right) \qquad (3.4.3\text{-}3)$$

$$\beta = 1 - \left(\frac{x_o}{r_o}\right)^2 \qquad (3.4.3\text{-}4)$$

where x_o, and r_o are defined in Section 3.3.1.2.

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

3.4.4 Nonsymmetric Sections

For shapes whose cross sections do not have any symmetry, either about an axis or about a point, F_n shall be determined by rational analysis. Alternatively, compression members composed of such shapes may be tested in accordance with Section 6.

3.5 Combined Axial Load and Bending

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

$$\frac{P_u}{\phi_c P_n} + \frac{C_{mx} M_{ux}}{\phi_b M_{nx} \alpha_{nx}} + \frac{C_{my} M_{uy}}{\phi_b M_{ny} \alpha_{ny}} \leq 1.0 \qquad (3.5\text{-}1)$$

$$\frac{P_u}{\phi_c P_{no}} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0 \qquad (3.5\text{-}2)$$

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

$$\frac{P_u}{\phi_c P_n} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \leq 1.0 \qquad (3.5\text{-}3)$$

where:

P_u = required axial strength;

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

P_n = nominal axial strength determined in accordance with Section 3.4;

P_{no} = nominal axial strength determined in accordance with Section 3.4, with $F_n = F_y$;

M_{nx}, M_{ny} = nominal flexural strengths about centroidal axes determined in accordance with Section 3.3;

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

$$= \frac{1}{\left(1 - \dfrac{P_u}{P_E}\right)} \qquad (3.5\text{-}4)$$

ϕ_b = 0.90 for beam sections with stiffened and partially stiffened compression flanges, 0.85 for beam sections with unstiffened compression flanges (Section 3.3.1.1), or 0.85 for laterally unbraced beams (Section 3.3.1.2);

ϕ_c = 0.85;

$$P_E = \frac{\pi^2 E_o I_b}{(K_b L_b)^2} \qquad (3.5\text{-}5)$$

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

L_b = actual unbraced length in plane of bending;

K_b = effective length factor in 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

$$C_m = 0.6 - 0.4(M_1/M_2) \qquad (3.5\text{-}6)$$

where:

M_1/M_2 = ratio of smaller to larger moment at ends of that portion of member under consideration which is unbraced in plane of bending. M_1/M_2 is positive when 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 plane of loading and subject to transverse loading between their supports, 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,
$C_m = 1.0$.

3.6 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 $0.881E_o/F_y$.

3.6.1 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 $\phi_b = 0.90$ and M_n is calculated as follows:

For $\dfrac{D}{t} \le \dfrac{0.112E_o}{F_y}$

$$M_n = F_y S_f \tag{3.6.1-1}$$

For $\dfrac{0.112E_o}{F_y} < \dfrac{D}{t} \le \dfrac{0.881E_o}{F_y}$

$$M_n = K_c F_y S_f \tag{3.6.1-2}$$

where:

F_y = specified yield strength as given in Table A1;
S_f = elastic section modulus of full, unreduced cross section; and

$$K_c = \frac{(1 - C)(E_o/F_y)}{(8.93 - \lambda_c)(D/t)} + \frac{5.882C}{8.93 - \lambda_c} \tag{3.6.1-3}$$

In Eq. 3.6.1-3:

C = ratio of effective proportional limit-to-yield strength as given in Table A17; and

$\lambda_c = 3.048C$, limiting value of $\dfrac{(E_o/F_y)}{(D/t)}$, based on specified ratio C.

3.6.2 Compression

The requirements of this Section apply to members in which the resultant of all factored 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:

$$\phi_c = 0.80$$
$$P_n = F_n A_e \tag{3.6.2-1}$$

In this equation:

F_n = flexural buckling stress determined according to Section 3.4.1;

$$A_e = \left[1 - \left(1 - \left(\frac{E_t}{E_o}\right)^2\right)\left(1 - \frac{A_o}{A}\right)\right]A \tag{3.6.2-2}$$

$$A_o = K_c A \le A, \quad \text{for} \quad \frac{D}{t} \le \frac{0.881E_o}{F_y} \tag{3.6.2-3}$$

A = area of full, unreduced cross section;
E_t/E_o = plasticity reduction factor corresponding to buckling stress, as given in Tables A10 and A11 or Figures A7 and A8, or can be determined by using Eq. B-5 in Appendix B; and

K_c is defined in Section 3.6.1.

3.6.3 Combined Bending and Compression

Combined bending and compression shall satisfy the provisions of Section 3.5.

3.7 Arc-and-Tangent Corrugated Sheets

When arc-and-tangent corrugated sheets are used for roofing, siding, and curtain wall, the design flexural strength, $\phi_b M_n$, may be taken as 0.90 $F_y S_f$ or based on Section 3.6.1, whichever is applicable.

The design strength of arc-and-tangent corrugated sheets may be established in accordance with Section 6.2, "Tests for Determining Structural Performance."

4. STRUCTURAL ASSEMBLIES

4.1 Built-Up Sections

4.1.1 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

1. For compression members:

$$s_{max} = \frac{Lr_{cy}}{(2r_I)} \tag{4.1.1-1}$$

where:

s_{max} = longitudinal spacing of connections;
L = unbraced length of compression member;
r_I = radius of gyration of I-section about axis perpendicular to direction in which buckling would occur for given conditions of end support and intermediate bracing; and
r_{cy} = radius of gyration of one channel about its centroidal axis parallel to web.

2. For flexural members:

$$s_{max} = \frac{L}{6} \tag{4.1.1-2}$$

In no case shall spacing exceed value

$$s_{max} = \frac{2gT_s}{(mq)} \tag{4.1.1-3}$$

where:

L = span of beam;
T_s = design strength of connection in tension (Section 5);
g = vertical distance between two rows of connections nearest to top and bottom flanges;
q = intensity of factored load on beam (for methods of determination, see as follows);
m = distance from shear center of one channel to midplane of its web;

$$m = \left(\frac{\bar{b}t}{12I_x}\right)[6\bar{D}(\bar{d})^2 + 3(\bar{b})(\bar{d})^2 - 8(\bar{D})^3] \tag{4.1.1-4}$$

$\bar{b} = B - \left(\frac{t}{2} + \frac{\alpha t}{2}\right)$;

$\bar{d} = d - t$;

$\bar{D} = \alpha\left(D - \frac{t}{2}\right)$;

B = flange width;
d = depth of channel beam;
D = depth of stiffening lip;
t = thickness of channel section;
α = coefficient; for sections with stiffening lips, α = 1.0; for sections without stiffening lips, α = 0; and
I_x = moment of inertia of one channel about its centroidal axis normal to 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 weld spacing, s, the required strength of the welds or connections closest to the load or reaction P_u is

$$T_s = \frac{P_u m}{(2g)} \tag{4.1.1-5}$$

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: (1) Connection spacing may be varied along the beam according to the variation of the load intensity; or (2) 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.

4.1.2 Spacing of Connections in Compression Elements

The spacing, s, in the line of stress, of welds, rivets, or bolts connecting a compression cover plate or sheet to a nonintegral stiffener or other element shall not exceed:

1. 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
2. $1.11t\,(\sqrt{E_t/f})$, where t = the thickness of the cover plate or sheet; f = the stress at service load in the cover plate or sheet; and E_t = the tangent modulus in compression; nor
3. Three times the flat width, w, of the narrowest unstiffened compression element tributary to the connections, but need not be less than $1.03t\sqrt{(E_o/F_y)}$ if $w/t < 0.50\sqrt{(E_o/F_y)}$, or $1.24t\sqrt{(E_o/F_y)}$ if $w/t \geq 0.50\sqrt{(E_o/F_y)}$, unless closer spacing is required by (1) or (2) preceding.

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 1/2 in. (12.7 mm). 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 ma-

terial and are not considered as load-carrying elements.

4.2 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. When two materials are in contact the possible interaction shall be considered by the design engineer.

4.3 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.

4.3.1 Symmetrical Beams and Columns

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

4.3.2 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: (1) Top flange is connected to deck or sheathing material in such a manner as effectively to restrain lateral deflection of the connected flange*; or (2) neither flange is so connected. When both flanges are so connected, no further bracing is required.

4.3.2.1 Bracing when one flange is connected. Channels and Z-sections used to support attached covering material and loaded in a plane parallel to the web shall be designed taking into account the restraining effects of the covering material and fasteners. Provisions shall be made for forces from each beam which may accumulate in the covering material. These forces shall be transferred from the covering material to a member or assembly of sufficient strength and stiffness to resist these forces.

The design of braces shall be in accordance with Section 4.3.2.2. In addition, tests in accordance with Section 6 shall be performed to insure that the type and/or spacing of braces selected is such that the test strength of the braced beam assembly is equal to or greater than its nominal flexural strength, instead of that required by Section 6.

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

4.3.2.2 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:

1. For uniform loads, $P_L = 1.5K'$ times the factored load within a distance $0.5a$ each side of the brace.
2. For concentrated loads, $P_L = 1.0K'$ times each factored 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 these formulas:
For channels and Z-sections:

x = distance from the concentrated load to brace.
a = distance between center line of braces.

For channels:

$$K' = \frac{m}{d} \qquad (4.3.2.2\text{-}1)$$

where:

m = distance from shear center to mid-plane of web, as specified in Section 4.1.1;
d = depth of channel.

For Z-sections:

$$K' = \frac{I_{xy}}{I_x} \qquad (4.3.2.2\text{-}2)$$

where:

I_{xy} = product of inertia of full section about centroidal axes parallel and perpendicular to web; and
I_x = moment of inertia of full section about centroidal axis perpendicular to web.

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

When braces are provided, they shall be attached in such a manner to effectively restrain the section against lateral deflection of both flanges at the ends and at any intermediate brace points.

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 torsional rotation and lateral displacement, no additional braces will be required except those required for strength according to Section 3.3.1.2.

4.3.3 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 $0.086E_o/F_y$.

5. CONNECTIONS AND JOINTS

5.1 General Provisions

Connections shall be designed to transmit the maximum forces resulting from the factored loads acting on the connected member. The internal forces and moments shall be distributed in a realistic manner and must be in equilibrium with the applied forces and moments. Each element participating in the assumed load paths shall be capable of resisting the forces assumed in the analysis. Proper regard shall be given to eccentricity.

5.2 Welded Connections

The following LRFD criteria govern welded connections used for cold-formed stainless steel structural members. All arc welds shall comply with the provisions of "Structural Welding Code—Sheet Steel." D1.3, of the American Welding Society, and revisions, except as otherwise specified herein and excepting such provisions of that Code as are clearly not applicable to material of the type and thickness to which this specification applies. Welded connections shall not be used for Type 430 stainless steel.

Welders and welding procedures shall be qualified as specified in AWS D1.3. Filler metal shall conform with the requirements of:

"Specification for Covered Corrosion-Resisting Chromium and Chromium-Nickel Steel Welding Electrodes," American Welding Society Specification A5.4-81; or

"Specification for Corrosion-Resisting Chromium and Chromium-Nickel Steel Bare and Composite Metal Cored and Stranded Welding Electrodes and Welding Rods," American Welding Society Specification A5.9-81.

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 5.2.1 and 5.2.2; and

P_n = nominal strength of welds determined according to Sections 5.2.1 and 5.2.2.

The design strengths, ϕP_n, for resistance welds made in conformance with the procedures given in AWS C1.1-66, "Recommended Practices for Resistance Welding," are given in Section 5.2.3.

5.2.1 Groove Welds in Butt Joints

The design strength, ϕP_n, in tension or compression of a groove weld in a butt joint, welded from one or both sides, shall be determined by the following equations, provided that an effective throat equal to or greater than the thickness of the material is consistently obtained and that matching weld must be used.

$$\phi = 0.60 \qquad P_n = LtF_{ua} \tag{5.2.1-1}$$

where:

ϕ = resistance factor for welded connections;

P_n = nominal strength of groove weld;

F_{ua} = tensile strength of annealed base metal, as given in Table A16;

L = length of weld; and

t = thickness of thinnest welded sheet.

5.2.2 Fillet Welds

Fillet welds covered by this Specification apply to the welding in lap or T-joints. The design shear strength, ϕP_n, of a fillet weld shall be determined as follows:

1. For longitudinal loading:

For $\dfrac{L}{t} < 30$:

$$\phi = 0.55 \qquad P_n = \left(0.7 - \frac{0.009L}{t}\right) tLF_{ua} \tag{5.2.2-1}$$

For $\dfrac{L}{t} \geq 30$:

$$\phi = 0.55 \qquad P_n = 0.43tLF_{ua} \tag{5.2.2-2}$$

In addition, the design strength thus determined shall not exceed the following value of ϕP_n:

$$\phi = 0.55 \qquad P_n = 0.75t_wLF_{xx} \tag{5.2.2-3}$$

2. For transverse loading:

$$\phi = 0.55 \qquad P_n = tLF_{ua} \tag{5.2.2-4}$$

In addition, the design strength determined by Eq. 5.2.2-4 shall not exceed the following value of ϕP_n:

$$\phi = 0.65 \qquad P_n = t_wLF_{xx} \tag{5.2.2-5}$$

where:

ϕ = resistance factor for welded connections;

P_n = nominal strength of fillet weld;
L = length of fillet weld;
t = thickness of thinnest connected sheet; and
t_w = effective throat = $0.707w_1$ or $0.707w_2$, whichever is smaller.
F_{xx} = strength level designation in AWS electrode classification (Table A15)

F_{ua} is defined in Section 5.2.1, and w_1 and w_2 are the legs on weld.

TABLE 3. Nominal Shear Strength of Arc Spot Welding

Thickness of thinnest outside sheet inch (mm)	Shear Strength per Spot: Annealed, 1/16 Hard kips (kN)	Shear Strength per Spot: 1/4 Hard kips (kN)	Shear Strength per Spot: 1/2 Hard kips (kN)
0.006 (0.152)	0.06 (0.27)	0.07 (0.31)	0.09 (0.4)
0.008 (0.203)	0.10 (0.44)	0.13 (0.58)	0.15 (0.67)
0.010 (0.254)	0.15 (0.67)	0.17 (0.76)	0.21 (0.93)
0.012 (0.305)	0.19 (0.85)	0.21 (0.93)	0.25 (1.11)
0.014 (0.356)	0.24 (1.07)	0.25 (1.11)	0.32 (1.42)
0.016 (0.406)	0.28 (1.25)	0.30 (1.33)	0.38 (1.69)
0.018 (0.457)	0.32 (1.42)	0.36 (1.60)	0.47 (2.09)
0.021 (0.533)	0.37 (1.64)	0.47 (2.09)	0.50 (2.22)
0.025 (0.635)	0.50 (2.22)	0.60 (2.67)	0.68 (3.02)
0.031 (0.787)	0.68 (3.02)	0.80 (3.56)	0.93 (4.13)
0.034 (0.864)	0.80 (3.56)	0.92 (4.09)	1.10 (4.89)
0.040 (1.016)	1.00 (4.45)	1.27 (5.65)	1.40 (6.23)
0.044 (1.118)	1.20 (5.34)	1.45 (6.45)	1.70 (7.56)
0.050 (0.222)	1.45 (6.45)	1.70 (7.56)	2.00 (8.89)
0.056 (1.422)	1.70 (7.56)	2.00 (8.9)	2.45 (10.9)
0.062 (1.575)	1.95 (8.67)	2.40 (10.68)	2.90 (12.9)
0.070 (1.778)	2.40 (10.68)	2.80 (12.45)	3.55 (15.79)
0.078 (1.981)	2.70 (12.01)	3.40 (15.12)	4.00 (17.79)
0.094 (2.388)	3.55 (15.79)	4.20 (18.68)	5.30 (23.57)
0.109 (2.769)	4.20 (18.68)	5.00 (22.24)	6.40 (28.47)
0.125 (3.175)	5.00 (22.24)	6.00 (26.69)	7.60 (33.80)

[a] Nominal tensile strength per spot may conservatively be taken as 25% of shear strength.

5.2.3 Resistance Welds

For Types 301, 304, and 316 stainless steel sheets, the design shear strength, ϕP_n, of spot welding shall be determined as follows:

ϕ = 0.60; and

P_n = tabulated value given in Table 3.

When Types 301, 304, or 316 stainless steel sheets are joined by pulsation welding, the nominal shear strength per spot shall be determined as follows:

TABLE 4. Nominal Shear Strength of Pulsation Welding

Thickness of thinnest outside sheet inch (mm.)	Shear Strength per Spot: 1/4 Hard kips (kN)	Shear Strength per Spot: 1/2 Hard kips (kN)
0.156 (3.962)	7.60 (33.8)	10.00 (44.48)
0.187 (4.75)	9.75 (43.37)	12.30 (54.71)
0.203 (5.156)	10.60 (47.15)	13.00 (57.82)
0.250 (6.35)	13.50 (57.82)	17.00 (75.62)

(These values are based on "Recommended Practice for Resistance Welding," C1.1-66, American Welding Society, 1966. Values for intermediate thicknesses may be obtained by straight line interpolation. These values may also be applied conservatively for Type 201. In all cases, welding shall be performed in accordance with the American Welding Society's "Recommended Practice for Resistance Welding.")

5.3 Bolted Connections

The following LRFD design criteria govern bolted connections used for cold-formed stainless steel structural members.

Bolts, nuts, and washers 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 5, except that larger holes may be used in column base details and structural systems connected to concrete walls.

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 6.

5.3.1 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:

$$\phi = 0.70$$
$$P_n = teF_u \qquad (5.3.1\text{-}1)$$

where:

ϕ = resistance factor;
P_n = nominal resistance per bolt;

TABLE 5. Maximum Size of Bolt Holes

Nominal bolt diameter, d inch (mm)	Standard hole diameter inch (mm)	Oversized hole diameter inch (mm)	Short-slotted hole dimensions inch (mm)	Long-slotted hole dimensions inch (mm)
$< 1/2$	$d + 1/32$	$d + 1/16$	$(d + 1/32)$ by $(d + 1/4)$	$(d + 1/32)$ by $(2\frac{1}{2} d)$
(12.7)	$(d + .794)$	$(d + 1.588)$	$(d + .794)$ $(d + 6.35)$	$(d + .793)$
$\geq 1/2$	$d + 1/16$	$d + 1/8$	$(d + 1/16)$ by $(d + 1/4)$	$(d + 1/16)$ by $(2\frac{1}{2} d)$
(12.7)	$(d + 1.588)$	$(d + 3.175)$	$(d + 1.588)$ $(d + 6.35)$	$(d + 1.588)$

e = distance measured in line of force from center of standard hole to nearest edge of adjacent hole or to end of connected part;
t = thickness of thinnest connected part;
d = nominal bolt diameter; and
F_u = specified tensile strength of connected sheet in longitudinal direction as given in Table A16.

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 $1\frac{1}{2}\,d$.

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 5. 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.

5.3.2 Tension in Connected Part

The design tensile strength, ϕP_n, on the net section of the connected part shall be determined as follows:

$$\phi = 0.70$$
$$P_n = A_n F_t \qquad (5.3.2\text{-}1)$$

where:

A_n = net area of connected part;

F_t = nominal tension stress for connections with washers under both bolt head and nut, determined as follows:

1. For single shear connection:
$$F_t = (1.0 - r + 2.5r\,d/s)F_u \leq F_u \qquad (5.3.2\text{-}2)$$
2. For double shear connection:
$$F_t = (1.0 - 0.9r + 3r\,d/s)F_u \leq F_u \qquad (5.3.2\text{-}3)$$

r = force transmitted by bolt or bolts at section considered, divided by tension force in 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 case of a single bolt, s = width of sheet.

F_u, d, and t are defined in Section 5.3.1.

In addition, the design tensile strength shall not exceed the following value of ϕP_n:

$$\phi = 0.85$$
$$P_n = F_y A_n \qquad (5.3.2\text{-}4)$$

in which F_y is the specified yield strength in tension of the connected part as given in Table A1.

5.3.3 Bearing

The design bearing strength, ϕP_n, shall be determined as follows:

$$\phi = 0.65$$
$$P_n = F_p dt \qquad (5.3.3\text{-}1)$$

where:

F_p = nominal bearing stress for bolts with washers under both bolt head and nut, determined as follows:

1. For single shear connection:
$$F_p = 2.00\,F_u \qquad (5.3.3\text{-}2)$$

2. For double shear connection:

$$F_p = 2.75\ F_u \qquad (5.3.3\text{-}3)$$

d = nominal bolt diameter; and

t and F_u are as defined in Section 5.3.1.

5.3.4 Shear and Tension in Stainless Steel 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 6

$$P_n = A_b F \qquad (5.3.4\text{-}1)$$

where:

A_b = gross cross-sectional area of bolt; and

F is given by F_{nv} or F_{nt} in Table 6.

The pullover shear/tension forces in the stainless 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.

When bolts are subjected to a combination of shear and tension, the required tension strength shall not exceed the design strength, ϕP_n, based on $\phi = 0.75$ and $P_n = A_b F'_{nt}$, in which F'_{nt} is determined as follows:

1. Threads in shear plane:

$$F'_{nt} = 1.25\ F_{nt} - 2.4\ f_v \le F_{nt} \qquad (5.3.4\text{-}2)$$

TABLE 6. Nominal Shear and Tensile Stresses for Stainless Steel Bolts

Type of stainless steels	Diameter d inch (mm)	Nominal shear stress F_{nv} $\phi = 0.65$: No threads in shear plane ksi (MPa)	Nominal shear stress F_{nv} $\phi = 0.65$: Threads in shear plane[g] ksi (MPa)	Nominal tensile stress[g] F_{nt} $\phi = 0.75$ ksi (MPa)
201[a]	all	45.0 (310.8)	33.7 (232.4)	56.0 (386.1)
304,316[b]	all	45.0	33.7	56.0
304,316[c]	$\le$ 1/2 (12.7)	54.0 (372.3)	40.5 (279.2)	67.5 (465.4)
	$>$ 1/2 (12.7)	45 (310.3)	33.7 (232.4)	56.0 (386.1)
304,316[d]	$\le$ 3/4 (19.1)	75.0 (517.1)	56.2 (387.5)	93.7 (646.1)
430[a]	all	36.0 (248.2)	27.0 (186.2)	45.0 (310.3)
430[e]	$1/4 \le d \le 1\cdot1/2$ $(6.4 \le d \le 38.1)$	42.0 (289.6)	31.5 (217.2)	52.5 (361.9)
304,316[e]	$1/4 \le d \le 1\cdot1/2$ $(6.4 \le d \le 38.1)$	42.0 (289.6)	31.5 (217.2)	52.5 (362.0)
304,316[f]	$1/4 \le d \le 5/8$ $(6.4 \le d \le 15.9)$	57.0 (393.0)	42.8 (295.1)	71.2 (490.9)
	$3/4 \le d \le 1\cdot1/2$ $(19.1 \le d \le 38.1)$	48.0 (331.0)	36.0 (248.2)	60.0 (413.7)

[a] Condition A in ASTM A276-85a, hot- or cold-finished.
[b] Condition A in ASTM A276-85a, hot-finished and Class 1 (solution-treated) in ASTM A193/A193M-86.
[c] Condition A in ASTM A276-85a, cold-finished.
[d] Condition B (cold-worked) in ASTM A276-85a, cold-finished and Class 2 (solution-treated and strain-hardened) in ASTM A193/A193M-86.
[e] Condition A in ASTM F593-86a, machined from annealed or solution-annealed stock, or hot-formed and solution-annealed. The minimum tensile strength is based on tests on the machined specimen.
[f] Condition CW in ASTM F593-86a, headed and rolled from annealed stock thus acquiring degree of cold work. Sizes 3/4 in. (19.05 mm) diameter and larger may be hot-worked. The minimum tensile strength is based on tests on the machined specimen.
[g] No reduction of nominal stress given in Table 6 is required for $d \ge 1/2$ in. (12.7 mm). For $d < 1/2$ in. (12.7 mm) given value shall be reduced to $0.9F_{nv}$ for nominal shear stress, and $0.9F_{nt}$ for nominal tensile stress.

2. No threads in shear plane:

$$F'_{nt} = 1.25\,F_{nt} - 1.9\,f_v \leq F_{nt} \qquad (5.3.4\text{-}3)$$

in which F_{nt} = the nominal tensile stress given in Table 6, and f_v = the shear stress produced by the same factored loads. The required shear strength shall not exceed the design shear strength, $\phi A_b F_{nv}$, in which ϕ and F_{nv} are determined in accordance with Table 6.

6. TESTS

Tests shall be made by an independent testing laboratory or by a manufacturer's testing laboratory witnessed by a professional engineer.

6.1 Determination of Stress-Strain Relationships

For stainless steels produced to other than ASTM Designations A666 and A240, the stress-strain relationship and mechanical properties used for the purpose of design shall be established on the basis of the tests required by ASTM A666, A240, and A370, supplemented by the following test methods as applicable:

"Tension Testing of Metallic Materials," ASTM Designation E8-85;
"Compression Testing of Metallic Materials at Room Temperature," ASTM Designation E9-81;
"Young's Modulus, Tangent Modulus, and Chord Modulus," ASTM Designation E111-82; and
"Verification and Classification of Extensometers," ASTM Designation E83-85.

Statistical studies shall be made to insure that the mechanical properties so determined shall be those for which there is a 90% probability that they will be equalled or exceeded in a random selection of the material lot under consideration. ASTM Designation E105-58, "Probability Sampling of Materials," and E141-69, "Acceptance of Evidence Based on the Results of Probability Sampling," may be used as guides for appropriate procedure.

6.2 Tests for Determining Structural Performance

Where the composition or configuration of elements, assemblies, connections, or details of structural members formed from sheet or strip stainless steel 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.

1. 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%. If such deviation from the average value exceeds 10%, 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.
2. The load-carrying capacity of the tested elements, assemblies, connections, or members shall satisfy Eq. 6.2-1.

$$\phi R_p \geq \Sigma \gamma_i Q_i \qquad (6.2\text{-}1)$$

where:

$\Sigma\gamma_i Q_i$ = required resistance based on most critical load combination determined in accordance with Section 1.5.2. γ_i and Q_i are load factors and load effects, respectively.
R_p = average value of all test results;
ϕ = resistance factor;

$$= 1.5(M_m F_m P_m)$$

$$\times \exp\left(-\beta_o \sqrt{V_M^2 + V_F^2 + C_p V_P^2 + V_Q^2}\right) \qquad (6.2\text{-}2);$$

M_m = mean value of material factor;
= 1.10 for yield strength and tensile strength of stainless steels;
F_m = mean value of fabrication factor;
= 1.00 for structural members and connections;
P_m = mean value of tested-to-predicted load ratios determined in Section 6.2(1);
β_o = target reliability;
= 3.0 for structural members and 4.0 for connections;
V_M = coefficient of variation of material factor;
= 0.10 for yield strength of stainless steels;
= 0.05 for tensile strength of stainless steels;

V_F = coefficient of variation of fabrication factor;
= 0.05 for structural members and bolted connections;
= 0.15 for welded connections;
C_p = correction factor;
= $(n - 1)/(n - 3)$ (6.2-3);
V_P = coefficient of variation of tested-to-predicted load ratios determined in Section 6.2(1);
n = number of tests;
V_Q = coefficient of variation of load effect;
= 0.21;

These values of M_m, F_m, V_M, and V_F do not exclude the use of other documented statistical data if they are established from sufficient results on material properties and fabrication. For stainless steels not listed in Section 1.3.1, 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. 6.2-1, except that the resistance factor ϕ is taken as unity and that the load factor for dead load may be taken as 1.0.

3. If the yield strength of the stainless 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 strength of the stainless steel which the manufacturer intends to use. The test results shall not be adjusted upward if the yield strength of the test specimen is less than the minimum specified yield strength. Similar adjustments shall be made on the basis of tensile strength instead of yield strength when 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.

6.3 Tests for Determining Mechanical Properties of Full Sections

Tests for determination of mechanical properties of formed sections to be used in Section 1.5.2.2 shall be conducted on full formed section as follows:

1. Tensile testing procedures shall agree with Standard Methods and Definitions for Mechanical Testing of Steel Products, ASTM A370-77. Compressive yield strength determinations shall be made by means of compression tests of short stub column specimens of the section.
2. The compressive yield stresses shall be taken as either the maximum compressive strength of the section divided by the cross-section area or the stress determined by the 0.2% offset method, whichever is reached first in the test.
3. Where the principal effect of the loading to which the member will be subjected in service will be to produce bending stresses, the yield strength shall be the lower of the yield strength determined in tension and in compression. In determining such yield strengths in flanged sections, tension and compression tests shall be made on specimens cut from the section. Each such 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.
4. For acceptance and control purposes, two full section tests shall be made from each lot of not more than 50 tons (45t) nor less than 30 tons (27t) of each section, or one test from each lot of less than 30 tons (27t) 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.
5. 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.

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APPENDIX A: DESIGN TABLES AND FIGURES

The mechanical properties given in the following tables (Tables A1 through A17) and figures (Figures A1 through A12) shall be used in this Specification.

APPENDIX B: MODIFIED RAMBERG-OSGOOD EQUATION

For the design of cold-formed stainless steel structural members, the values of secant modulus (E_s), the tangent modulus (E_t), and the plasticity reduction factor (η) corresponding to different stresses are given in the Design Tables and Figures of this Specification. Alternatively, these properties may be determined by using analytical expressions, which are based on the Modified Ramberg-Osgood equation, as follows:

1. *Secant Modulus, E_s*

The secant modulus, E_s, defined as the ratio of the stress and the strain, can be determined as follows:

$$E_s = \frac{\sigma}{\varepsilon} = \frac{E_o}{\left(1 + 0.002E_o\left(\frac{\sigma^{n-1}}{F_y^n}\right)\right)} \tag{B-1}$$

2. *Tangent Modulus, E_t*

The tangent modulus, E_t, which is defined as the slope of the stress strain curve in the inelastic range, is derived from the first derivative of the stress-strain ratio. Eq. B-2 gives the tangent modulus as a function of stress.

$$E_t = \frac{d\sigma}{d\varepsilon} = \frac{E_oF_y}{F_y + 0.002\, nE_o\left(\frac{\sigma}{F_y}\right)^{n-1}} \tag{B-2}$$

3. *Plasticity Reduction Factor, η*

The plasticity reduction factors used for the design of cold-formed stainless steel structural members can be obtained from the following equations, which are based on the secant and tangent moduli previously derived:

For stiffened compression elements:

$$\eta = \sqrt{\frac{E_t}{E_o}} = \sqrt{\frac{F_y}{F_y + 0.002nE_o\left(\frac{\sigma}{F_y}\right)^{n-1}}} \tag{B-3}$$

For unstiffened compression elements:

$$\eta = \frac{E_s}{E_o} = \frac{1}{1 + 0.002E_o\left(\frac{\sigma^{n-1}}{F_y^n}\right)} \tag{B-4}$$

TABLE A1. Specified Yield Strengths of Stainless Steels

Type of Stress	F_y, ksi (MPa)									
	Types 201, 301, 304, 316						S20400		Type 409	Types 430, 439
	Annealed		1/16 Hard		1/4 Hard	1/2 Hard	Annealed	1/4 Hard		
Longitudinal tension	30 (206.9)	45# (310.3)	40* (275.8)	45	75 (517.1)	110 (758.5)	48 (330)	100 (690)	30 (206.9)	40† (275.8)
Transverse tension	30	45#	40*	45	75	110	48	100	35† (241.3)	45† (310.3)
Transverse compression	30	45#	40*	45	90	120	48	110 (758)	35†	45†
Longitudinal compression	28 (193.1)	41# (282.7)	36* (248.2)	41	50 (344.8)	65 (448.2)	48	65 (448)	30	40†
Shear yield strength, F_{yv}	17 (117.2)	25# (172.4)	23* (158.6)	25	42 (289.6)	56 (386.1)	27 (186)	57 (393)	19 (131)	24 (165.5)

For Type 201-2 (Class 2).
* Flat bars, for Type 201 only.
† Adjusted yield strengths; ASTM specified yield strength is 30 ksi (206.9 MPa) for Types 409, 430, 439.

TABLE A2a. Secant Moduli for Deflection Calculations (Types 201, 301, 304, 316)

Stress ksi (MPa)	Secant Modulus, E_s, ksi $\times 10^3$ (MPa $\times 10^3$)					
	Longitudinal compression			Transverse compression		
	Annealed and 1/16 Hard	1/4 Hard	1/2 Hard	Annealed and 1/16 Hard	1/4 Hard	1/2 Hard
0	28.0 (193.1)	27.0 (186.2)	27.0	28.0	28.0	28.0
4 (27.6)	28.0	27.0	27.0	28.0	28.0	28.0
8 (55.2)	28.0	27.0	27.0	28.0	28.0	28.0
12 (82.7)	28.0	27.0	27.0	28.0	28.0	28.0
16 (110.3)	24.8 (171)	27.0	27.0	28.0	28.0	28.0
20 (137.9)	21.3 (146.9)	27.0	27.0	28.0	28.0	28.0
24 (165.5)	18.5 (127.6)	26.2 (180.6)	26.7 (185)	27.7 (191)	28.0	28.0
28 (193.1)		24.0 (165.5)	25.4 (175.1)		28.0	28.0
32 (220.6)		21.3 (146.9)	24.2 (166.9)		28.0	28.0
36 (248.2)		18.8 (129.6)	23.0 (158.6)		27.9 (192.4)	28.0
40 (275.8)		16.9 (116.5)	21.8 (150.3)		27.8 (191.7)	28.0
44 (303.4)		15.3 (105.5)	20.6 (142)		27.4 (188.9)	28.0
48 (331)		13.9 (95.8)	19.4 (133.8)		27.0 (186.2)	28.0
52 (358.5)		12.5 (86.2)	18.2 (125.5)		26.4 (182)	27.9 (192.4)
56 (386.1)			17.1 (117.2)			27.6 (190.3)
60 (413.7)			16.0 (110.3)			27.2 (187.5)
64 (441.3)			15.0 (103.4)			26.8 (184.8)
68 (468.9)			14.0 (96.5)			26.4 (182)
	Longitudinal tension			Transverse tension		
0	28.0 (193.1)	27.0 (186.2)	27.0	28.0	28.0	28.0
4 (27.6)	28.0	27.0	27.0	28.0	28.0	28.0
8 (55.2)	28.0	27.0	27.0	28.0	28.0	28.0
12 (82.7)	28.0	27.0	27.0	28.0	28.0	28.0
16 (110.3)	28.0	27.0	27.0	28.0	28.0	28.0
20 (137.9)	28.0	27.0	27.0	28.0	28.0	28.0
24 (165.5)	27.1 (186.9)	27.0	27.0	25.6 (176.5)	28.0	28.0
28 (193.1)		27.0	27.0		28.0	28.0
32 (220.6)		26.8 (184.786)	27.0		28.0	28.0
36 (248.2)		26.1 (180)	27.0		27.9 (192.4)	28.0
40 (275.8)		25.4 (175.1)	27.0		27.4 (189)	28.0
44 (303.4)		24.6 (169.6)	26.8 (184.8)		26.8 (184.8)	28.0
48 (331)		23.8 (164.1)	26.6 (183.4)		25.9 (178.6)	28.0
52 (358.5)		22.9 (157.9)	26.4 (182)		25.0 (172.4)	27.9 (192.4)
56 (386.1)			26.1 (180)			27.6 (190.3)
60 (413.7)			25.7 (177.2)			27.2 (187.5)
64 (441.3)			25.3 (174.4)			26.7 (184.1)
68 (468.9)			24.9 (171.7)			26.2 (180.6)

TABLE A2b. Secant Moduli for Deflection Calculations (UNS S20400)

Stress ksi (MPa)	Secant Modulus, E_s, ksi × 10^3 (MPa × 10^3)			
	Longitudinal compression		Transverse compression	
	Annealed	1/4 Hard	Annealed	1/4 Hard
0	28.0 (193)	28.0 (193)	28.0	28.0
4 (27.6)	28.0	27.7 (191)	28.0	27.9 (192)
8 (55.2)	28.0	27.2 (188)	28.0	27.8 (192)
12 (82.7)	27.8 (192)	26.5 (183)	28.0	27.7 (191)
16 (110.3)	27.5 (190)	25.7 (177)	28.0	27.5 (190)
20 (137.9)	26.9 (185)	24.8 (171)	28.0	27.2 (188)
24 (165.5)	25.8 (178)	23.9 (165)	27.9 (192)	27.0 (186)
28 (193.1)	24.3 (168)	22.9 (158)	27.7 (191)	26.7 (184)
32 (220.6)	22.4 (154)	22.0 (152)	27.0 (186)	26.4 (182)
36 (248.2)	20.1 (139)	21.0 (145)	25.4 (175)	26.0 (179)
40 (275.8)	17.7 (122)	20.1 (139)	22.4 (154)	25.7 (177)
44 (303.4)				25.3 (174)
48 (331)				24.9 (172)
52 (358.5)				24.5 (169)
56 (386.1)				24.1 (166)
60 (413.7)				23.7 (163)
64 (441.3)				23.3 (161)
68 (468.9)				22.9 (158)
	Longitudinal tension		Transverse tension	
0	28.0 (193)	28.0 (193)	28.0	28.0
4 (27.6)	28.0	27.8 (192)	28.0	27.9 (192)
8 (55.2)	27.9 (192)	27.6 (190)	28.0	27.6 (190)
12 (82.7)	27.6 (190)	27.2 (188)	28.0	27.3 (188)
16 (110.3)	27.1 (187)	26.9 (185)	27.9 (192)	27.0 (186)
20 (137.9)	26.2 (181)	26.5 (183)	27.6 (190)	26.6 (183)
24 (165.5)	24.9 (172)	26.1 (180)	27.1 (187)	26.2 (181)
28 (193.1)	23.3 (161)	25.6 (176)	26.1 (180)	25.8 (178)
32 (220.6)	21.3 (147)	25.2 (174)	24.5 (169)	25.4 (175)
36 (248.2)	19.2 (132)	24.8 (171)	22.1 (152)	25.0 (172)
40 (275.8)	17.0 (117)	24.3 (168)	19.2 (132)	24.5 (169)
44 (303.4)		23.8 (164)		24.0 (165)
48 (331)		23.4 (161)		23.6 (163)
52 (358.5)		22.9 (158)		23.1 (159)
56 (386.1)		22.5 (155)		22.7 (156)
60 (413.7)		22.0 (152)		22.2 (153)
64 (441.3)		21.6 (149)		21.7 (150)
68 (468.9)		21.2 (146)		21.3 (147)

TABLE A3. Secant Moduli for Deflection Calculations (Types 409, 430, 439)

Stress ksi (MPa)	Secant Modulus, E_s, ksi $\times$ 10^3 (MPa $\times$ 10^3)							
	Type 409				Types 430, 439			
	Long. comp.	Tran. comp.	Long. ten.	Tran. ten.	Long. comp.	Tran. comp.	Long. ten.	Tran. ten.
0	27.0 (186.2)	29.0 (200)	27.0	29.0	27.0	29.0	27.0	29.0
4 (27.6)	27.0	29.0	27.0	29.0	27.0	29.0	27.0	29.0
8 (55.2)	27.0	29.0	27.0	29.0	27.0	29.0	27.0	29.0
12 (82.7)	27.0	29.0	27.0	29.0	27.0	29.0	27.0	29.0
16 (110.3)	26.8 (184.8)	29.0	26.9 (185.5)	29.0	27.0	29.0	27.0	29.0
20 (137.9)	25.6 (176.5)	28.9 (199.3)	26.1 (180)	28.9	26.6 (183.4)	29.0	26.9 (185.5)	29.0
24 (165.5)	21.5 (148.2)	27.1 (186.9)	22.4 (154.4)	27.1	25.3 (174.4)	29.0	26.6 (183.4)	29.0
28 (193.1)	13.6 (93.8)	17.1 (117.9)	14.1 (97.2)	17.1	22.2 (153.1)	29.0	24.8 (171)	29.0
32 (220.6)	6.5 (44.8)	4.8 (33.1)	6.2 (42.7)	4.8	17.3 (119.3)	28.7 (138.6)	20.1 (138.6)	28.7
36 (248.2)					12.0	26.4 (182)	13.0 (89.6)	26.3
40 (275.8)					7.7 (53.1)	16.4 (113.1)	6.9 (47.6)	16.3 (112.4)
44 (303.4)						5.2 (35.9)		5.3 (36.5)

Note: Long. comp. = longitudinal compression.
Tran. comp. = transverse compression.
Long. ten. = longitudinal tension.
Tran. ten. = transverse tension.

TABLE A4a. Initial Moduli of Elasticity and Initial Shear Moduli (Types 201, 301, 304, 316)

Type / Modulus	Annealed and 1/16 Hard	1/4 Hard and 1/2 Hard	
	Longitudinal and transverse tension and compression	Longitudinal tension and compression	Transverse tension and compression
Initial modulus of elasticity: E_o, ksi $\times$ 10^3 (MPa $\times$ 10^3)	28.0 (193.1)	27.0 (186.2)	28.0 (193.1)
Initial shear modulus: G_o, ksi $\times$ 10^3 (MPa $\times$ 10^3)	10.8 (74.5)	10.5 (72.4)	10.8 (74.5)

TABLE A4b. Initial Moduli of Elasticity and Initial Shear Moduli (UNS S20400)

Type / Modulus	Annealed	1/4 Hard	
	Longitudinal and transverse tension and compression	Longitudinal tension and compression	Transverse tension and compression
Initial modulus of elasticity: E_o, ksi $\times$ 10^3 (MPa $\times$ 10^3)	28.0 (193.1)	28.0	28.0
Initial shear modulus: G_o, ksi $\times$ 10^3 (MPa $\times$ 10^3)	10.8 (74.5)	10.8	10.8

TABLE A5. Initial Moduli of Elasticity and Initial Shear Moduli (Types 409, 430, 439)

Modulus \ Type	Longitudinal tension and compression	Transverse tension and compression
Initial modulus of elasticity: E_o, ksi $\times 10^3$ (MPa $\times 10^3$)	27.0 (186.2)	29.0 (200)
Initial shear modulus: G_o, ksi $\times 10^3$ (MPa $\times 10^3$)	10.5 (72.4)	11.2 (77.2)

TABLE A6a. Plasticity Reduction Factors for Stiffened Elements (Types 201, 301, 304, 316)

	$\sqrt{E_t/E_o}$					
	Longitudinal compression			Transverse compression		
Stress ksi (MPa)	Annealed and 1/16 Hard	1/4 Hard	1/2 Hard	Annealed and 1/16 Hard	1/4 Hard	1/2 Hard
0	1.00	1.00	1.00	1.00	1.00	1.00
4 (27.6)	1.00	1.00	1.00	1.00	1.00	1.00
8 (55.2)	1.00	1.00	1.00	1.00	1.00	1.00
12 (82.7)	1.00	1.00	1.00	1.00	1.00	1.00
16 (10.3)	0.77	1.00	1.00	1.00	1.00	1.00
20 (137.9)	0.67	1.00	1.00	1.00	1.00	1.00
24 (165.5)	0.58	0.79	0.98	0.81	1.00	1.00
28 (193.1)	0.50	0.71	0.86	0.62	1.00	1.00
32 (220.6)		0.65	0.80		1.00	1.00
36 (248.2)		0.60	0.75		1.00	1.00
40 (275.8)		0.56	0.71		0.99	1.00
44 (303.4)		0.52	0.68		0.93	1.00
48 (331)		0.48	0.64		0.88	1.00
52 (358.5)		0.45	0.60		0.84	1.00
56 (386.1)			0.57		0.80	0.97
60 (413.3)			0.53		0.77	0.94
64 (441.3)			0.50		0.73	0.91
68 (468.9)			0.47		0.70	0.87

TABLE A6b. Plasticity Reduction Factors for Stiffened Elements (UNS S20400)

Stress ksi (MPa)	$\sqrt{E_t/E_o}$ Longitudinal compression Annealed	1/4 Hard	Transverse compression Annealed	1/4 Hard
0	1.00	1.00	1.00	1.00
4 (27.6)	1.00	0.99	1.00	1.00
8 (55.2)	1.00	0.96	1.00	0.99
12 (82.7)	0.99	0.93	1.00	0.98
16 (10.3)	0.96	0.90	1.00	0.98
20 (137.9)	0.91	0.86	1.00	0.96
24 (165.5)	0.84	0.83	0.98	0.95
28 (193.1)	0.76	0.80	0.95	0.94
32 (220.6)	0.67	0.76	0.86	0.93
36 (248.2)	0.59	0.73	0.71	0.91
40 (275.8)	0.51	0.70	0.55	0.90
44 (303.4)	0.45	0.67	0.40	0.88
48 (331)	0.39	0.65	0.29	0.87
52 (358.5)	0.34	0.62	0.21	0.85
56 (386.1)		0.60		0.83
60 (413.3)		0.58		0.82
64 (441.3)		0.56		0.80
68 (468.9)		0.54		0.79

TABLE A7. Plasticity Reduction Factors for Stiffened Elements (Types 409, 430, 439)

Stress ksi (MPa)	$\sqrt{E_t/E_o}$ Type 409 Longitudinal compression	Type 409 Transverse compression	Types 430, 439 Longitudinal compression	Types 430, 439 Transverse compression
0	1.00	1.00	1.00	1.00
4 (27.6)	1.00	1.00	1.00	1.00
8 (55.2)	1.00	1.00	1.00	1.00
12 (82.7)	0.99	1.00	1.00	1.00
16 (110.3)	0.96	1.00	0.99	1.00
20 (137.9)	0.81	0.96	0.96	1.00
24 (165.5)	0.54	0.69	0.84	1.00
28 (193.1)	0.30	0.28	0.66	0.99
32 (220.6)	0.17	0.10	0.47	0.94
36 (248.2)			0.33	0.64
40 (275.8)			0.24	0.28
44 (303.4)				0.14

TABLE A8a. Plasticity Reduction Factors for Unstiffened Elements (Types 201, 301, 304, 316)

	E_s/E_o					
	Longitudinal compression			Transverse compression		
Stress ksi (MPa)	Annealed and 1/16 Hard	1/4 Hard	1/2 Hard	Annealed and 1/16 Hard	1/4 Hard	1/2 Hard
0	1.00	1.00	1.00	1.00	1.00	1.00
4 (27.6)	1.00	1.00	1.00	1.00	1.00	1.00
8 (55.2)	1.00	1.00	1.00	1.00	1.00	1.00
12 (82.7)	1.00	1.00	1.00	1.00	1.00	1.00
16 (110.4)	0.89	1.00	1.00	1.00	1.00	1.00
20 (137.9)	0.76	1.00	1.00	1.00	1.00	1.00
24 (165.5)	0.66	0.97	0.99	0.99	1.00	1.00
28 (193.1)	0.57	0.89	0.94	0.86	1.00	1.00
32 (220.6)	0.46	0.79	0.90	0.70	1.00	1.00
36 (248.2)	0.35	0.70	0.85	0.49	1.00	1.00
40 (275.8)	0.23	0.63	0.81	0.28	0.99	1.00
44 (303.4)	0.12	0.57	0.76	0.08	0.98	1.00
48 (331)		0.51	0.72		0.96	1.00
52 (358.5)		0.46	0.67		0.94	0.99
56 (386.1)			0.63		0.92	0.98
60 (413.7)			0.59		0.89	0.97
64 (441.3)			0.56		0.86	0.96
68 (468.9)			0.52		0.83	0.94
72 (496.4)					0.80	0.92
76 (524)					0.76	0.91
80 (551.6)					0.72	0.89
84 (579.2)					0.68	0.87
88 (606.8)					0.64	0.86
92 (620.6)					0.59	0.84
96 (661.9)						0.82
100 (689.5)						0.80
104 (717.1)						0.78
108 (744.7)						0.76
112 (772.2)						0.74
116 (799.8)						0.71
120 (827.4)						0.68

TABLE A8b. Plasticity Reduction Factors for Unstiffened Elements (UNS S20400)

Stress ksi (MPa)	E_s/E_o Longitudinal compression Annealed	Longitudinal compression 1/4 Hard	Transverse compression Annealed	Transverse compression 1/4 Hard
0	1.00	1.00	1.00	1.00
4 (27.6)	1.00	0.99	1.00	1.00
8 (55.2)	1.00	0.97	1.00	0.99
12 (82.7)	0.99	0.95	1.00	0.99
16 (110.4)	0.98	0.92	1.00	0.98
20 (137.9)	0.96	0.89	1.00	0.97
24 (165.5)	0.92	0.85	1.00	0.96
28 (193.1)	0.87	0.82	0.99	0.95
32 (220.6)	0.80	0.78	0.96	0.94
36 (248.2)	0.72	0.75	0.91	0.93
40 (275.8)	0.63	0.72	0.80	0.92
44 (303.4)	0.54	0.69	0.64	0.90
48 (331)	0.46	0.65	0.46	0.89
52 (358.5)	0.39	0.62	0.30	0.88
56 (386.1)		0.60		0.86
60 (413.7)		0.57		0.85
64 (441.3)		0.54		0.83
68 (468.9)		0.52		0.82
72 (496.4)		0.50		0.80
76 (524)				0.79
80 (551.6)				0.77
84 (579.2)				0.76
88 (606.8)				0.74
92 (620.6)				0.73
96 (661.9)				0.71
100 (689.5)				0.70
104 (717.1)				0.68
108 (744.7)				0.67
112 (772.2)				0.66
116 (799.8)				0.64
120 (827.4)				0.63

TABLE A9. Plasticity Reduction Factors for Unstiffened Elements (Types 409, 430, 439)

	E_s/E_o			
	Type 409		Types 430, 439	
Stress ksi (MPa)	Longitudinal compression	Transverse compression	Longitudinal compression	Transverse compression
0	1.00	1.00	1.00	1.00
4 (27.6)	1.00	1.00	1.00	1.00
8 (55.2)	1.00	1.00	1.00	1.00
12 (82.7)	1.00	1.00	1.00	1.00
16 (110.4)	0.99	1.00	1.00	1.00
20 (137.9)	0.95	1.00	0.99	1.00
24 (165.5)	0.79	0.93	0.94	1.00
28 (193.1)	0.50	0.59	0.82	1.00
32 (220.6)	0.24	0.17	0.64	0.99
36 (248.2)			0.44	0.91
40 (275.8)			0.28	0.56
44 (303.4)				0.18

TABLE A10a. Plasticity Reduction Factors for Lateral Buckling Strengths (Types 201, 301, 304, 316)

Stress ksi (MPa)	E_t/E_o					
	Longitudinal compression			Transverse compression		
	Annealed and 1/16 Hard	1/4 Hard	1/2 Hard	Annealed and 1/16 Hard	1/4 Hard	1/2 Hard
0	1.00	1.00	1.00	1.00	1.00	1.00
4 (27.6)	1.00	1.00	1.00	1.00	1.00	1.00
8 (55.2)	1.00	1.00	1.00	1.00	1.00	1.00
12 (82.7)	1.00	1.00	1.00	1.00	1.00	1.00
16 (110.4)	0.60	1.00	1.00	1.00	1.00	1.00
20 (137.9)	0.45	1.00	1.00	1.00	1.00	1.00
24 (165.5)	0.34	0.63	0.96	0.66	1.00	1.00
28 (193.1)	0.25	0.50	0.74	0.38	1.00	1.00
32 (220.6)	0.16	0.42	0.64	0.21	1.00	1.00
36 (248.2)	0.10	0.36	0.56	0.09	1.00	1.00
40 (275.8)	0.05	0.31	0.51	0.04	0.98	1.00
44 (303.4)	0.01	0.27	0.46	0.02	0.86	1.00
48 (331)		0.23	0.41		0.78	1.00
52 (358.5)		0.20	0.36		0.71	1.00
56 (386.1)		0.18	0.33		0.65	0.94
60 (413.7)		0.15	0.29		0.59	0.88
64 (441.3)		0.13	0.25		0.54	0.82
68 (468.9)		0.11	0.22		0.49	0.77
72 (496.4)		0.10	0.19		0.44	0.73
76 (524)			0.17		0.39	0.68
80 (551.6)			0.16		0.34	0.64
84 (579.2)			0.14		0.29	0.60
88 (606.8)			0.13		0.25	0.56
92 (620.6)			0.12		0.20	0.53
96 (661.9)			0.11		0.16	0.49
100 (689.5)			0.11		0.13	0.46
104 (717.1)					0.10	0.43
108 (744.7)					0.07	0.39
112 (772.2)						0.36
116 (799.8)						0.32
120 (824.4)						0.29
124 (855)						0.26
128 (882.6)						0.23
132 (910.1)						0.20
136 (937.7)						0.16
140 (965.3)						0.13
144 (992.9)						0.10
148 (1020.5)						0.07

TABLE A10b. Plasticity Reduction Factors for Lateral Buckling Strengths (UNS S20400)

Stress	E_t/E_o			
	Longitudinal compression		Transverse compression	
ksi (MPa)	Annealed	1/4 Hard	Annealed	1/4 Hard
0	1.00	1.00	1.00	1.00
4 (27.6)	1.00	0.98	1.00	1.00
8 (55.2)	0.99	0.93	1.00	0.98
12 (82.7)	0.97	0.87	1.00	0.97
16 (110.4)	0.92	0.81	1.00	0.95
20 (137.9)	0.83	0.75	0.99	0.93
24 (165.5)	0.71	0.69	0.97	0.91
28 (193.1)	0.58	0.63	0.90	0.88
32 (220.6)	0.45	0.58	0.74	0.86
36 (248.2)	0.35	0.54	0.51	0.83
40 (275.8)	0.26	0.49	0.30	0.80
44 (303.4)	0.20	0.45	0.16	0.78
48 (331)	0.15	0.42	0.08	0.75
52 (358.5)	0.12	0.39	0.04	0.72
56 (386.1)	0.09	0.36	0.02	0.70
60 (413.7)	0.07	0.34	0.01	0.67
64 (441.3)	0.06	0.31	0.01	0.65
68 (468.9)	0.05	0.29	0.00	0.62
72 (496.4)	0.04	0.27		0.60
76 (524)		0.26		0.58
80 (551.6)		0.24		0.56
84 (579.2)		0.23		0.54
88 (606.8)		0.21		0.52
92 (620.6)		0.20		0.50
96 (661.9)		0.19		0.48
100 (689.5)		0.18		0.46
104 (717.1)		0.17		0.44
108 (744.7)		0.16		0.43
112 (772.2)		0.16		0.41
116 (799.8)		0.15		0.40
120 (824.4)		0.14		0.39
124 (855)		0.14		0.37
128 (882.6)		0.13		0.36
132 (910.1)		0.12		0.35
136 (937.7)		0.12		0.34
140 (965.3)		0.11		0.33
144 (992.9)		0.11		0.32
148 (1020.5)		0.11		0.31

TABLE A11. Plasticity Reduction Factors for Lateral Buckling Strengths (Types 409, 430, 439)

Stress ksi (MPa)	E_t/E_o Type 409 Longitudinal compression	E_t/E_o Type 409 Transverse compression	E_t/E_o Types 430, 439 Longitudinal compression	E_t/E_o Types 430, 439 Transverse compression
0	1.00	1.00	1.00	1.00
4 (27.6)	1.00	1.00	1.00	1.00
8 (55.2)	1.00	1.00	1.00	1.00
12 (82.7)	0.99	1.00	1.00	1.00
16 (110.4)	0.93	1.00	0.99	1.00
20 (137.9)	0.66	0.93	0.92	1.00
24 (165.5)	0.29	0.47	0.71	1.00
28 (193.1)	0.09	0.08	0.43	0.99
32 (220.6)	0.03	0.01	0.22	0.89
36 (248.2)			0.11	0.41
40 (275.8)			0.06	0.08
44 (303.4)				0.02

TABLE A12. Plasticity Reduction Factors for Shear Strengths

Stress ksi (MPa)	G_s/G_o Types 201, 301, 304, 316 Annealed and 1/16 Hard	G_s/G_o Types 201, 301, 304, 316 1/4 Hard	G_s/G_o Types 201, 301, 304, 316 1/2 Hard	G_s/G_o Type 409	G_s/G_o Types 430, 439
0	1.00	1.00	1.00	1.00	1.00
4 (27.6)	1.00	1.00	1.00	1.00	1.00
8 (55.2)	0.98	1.00	1.00	1.00	1.00
12 (82.7)	0.84	1.00	1.00	0.83	1.00
16 (110.4)	0.58	0.98	1.00		0.99
20 (137.9)	0.24	0.95	0.99		0.70
24 (165.5)	0.03	0.90	0.97		
28 (193.1)		0.85	0.95		
32 (220.6)		0.78	0.93		
36 (248.2)		0.70	0.89		
40 (275.8)		0.61	0.85		
44 (303.4)		0.51	0.81		
48 (331)			0.77		
52 (358.5)			0.71		
56 (386.1)			0.65		

TABLE A13a. Tangent Moduli for Design of Columns (Types 201, 301, 304, 316)

	Tangent Modulus, E_t, ksi $\times 10^3$ (MPa $\times 10^3$)					
	Longitudinal compression			Transverse compression		
Stress ksi (MPa)	Annealed and 1/16 Hard	1/4 Hard	1/2 Hard	Annealed and 1/16 Hard	1/4 Hard	1/2 Hard
0	28.0 (193.1)	27.0 (186.2)	27.0	28.0	28.0	28.0
4 (27.6)	28.0	27.0	27.0	28.0	28.0	28.0
8 (55.2)	28.0	27.0	27.0	28.0	28.0	28.0
12 (82.7)	28.0	27.0	27.0	28.0	28.0	28.0
16 (110.4)	16.7 (115.1)	27.0	27.0	28.0	28.0	28.0
20 (137.9)	12.5 (86.2)	27.0	27.0	28.0	28.0	28.0
24 (165.5)	9.5 (65.5)	17.0 (117.2)	26.0 (179.3)	18.5 (127.6)	28.0	28.0
28 (193.1)	7.0 (48.3)	13.5 (93.1)	20.0 (137.9)	10.7 (73.8)	28.0	28.0
32 (220.6)	4.6 (31.7)	11.3 (77.9)	17.2 (118.6)	5.9 (40.7)	28.0	28.0
36 (248.2)	2.7 (18.6)	9.7 (66.9)	15.2 (104.8)	2.5 (17.2)	28.0	28.0
40 (275.8)	1.4 (9.7)	8.4 (57.9)	13.7 (94.5)	1.2 (8.3)	27.3 (188.2)	28.0
44 (303.4)	0.4 (2.8)	7.2 (49.6)	12.4 (85.5)	0.6 (4.1)	24.0 (165.5)	28.0
48 (331)		6.3 (43.4)	11.0 (75.8)		21.7 (149.6)	28.0
52 (358.5)		5.5 (37.9)	9.8 (67.6)		19.9 (137.2)	28.0
56 (386.1)			8.8 (60.7)		18.1 (124.8)	26.4 (182)
60 (413.7)			7.7 (53.1)		16.6 (114.5)	24.5 (168.9)
64 (441.3)			6.8 (46.9)		15.1 (104.1)	23.0 (158.6)
68 (468.9)			6.0 (41.4)		13.7 (94.5)	21.5 (148.2)
72 (496.4)					12.3 (84.8)	20.4 (140.7)
76 (524)					10.9 (75.2)	19.1 (131.7)
80 (551.6)					9.5 (65.5)	18.0 (124.1)
84 (579.9)					8.2 (56.5)	16.9 (116.5)
88 (606.8)					6.9 (47.6)	15.8 (108.9)
92 (620.6)					5.7 (39.3)	14.8 (102)
96 (661.9)						13.8 (95.2)
100 (689.5)						12.8 (88.3)
104 (717.1)						11.9 (77.2)
108 (744.7)						10.9 (75.2)
112 (772.2)						10.0 (69)
116 (799.8)						9.0 (62.1)
120 (827.4)						8.1 (55.8)

TABLE A13b. Tangent Moduli for Design of Columns (UNS S20400)

Stress ksi (MPa)	Tangent Modulus, E_t, ksi $\times$ 10^3 (MPa $\times$ 10^3)			
	Longitudinal compression		Transverse compression	
	Annealed	1/4 Hard	Annealed	1/4 Hard
0	28.0 (193)	28.0	28.0	28.0
4 (27.6)	28.0	27.3 (188)	28.0	27.9 (192)
8 (55.2)	27.8 (192)	26.0 (179)	28.0	27.6 (190)
12 (82.7)	27.2 (188)	24.4 (168)	28.0	27.1 (187)
16 (110.4)	25.8 (178)	22.7 (156)	28.0	26.6 (183)
20 (137.9)	23.3 (161)	20.9 (144)	27.8 (192)	26.0 (179)
24 (165.5)	19.9 (137)	19.3 (133)	27.2 (188)	25.4 (175)
28 (193.1)	16.2 (112)	17.7 (122)	25.1 (173)	24.7 (170)
32 (220.6)	12.7 (87.6)	16.3 (112)	20.7 (143)	24.0 (165)
36 (248.2)	9.7 (66.9)	15.0 (103)	14.3 (98.6)	23.2 (160)
40 (275.8)	7.4 (51.0)	13.8 (95.1)	8.3 (57.2)	22.5 (155)
44 (303.4)	5.6 (38.6)	12.7 (87.6)	4.5 (31.0)	21.7 (150)
48 (331)	4.2 (29.0)	11.8 (81.4)	2.3 (15.9)	21.0 (145)
52 (358.5)	3.3 (22.8)	10.9 (75.2)	1.2 (8.3)	20.2 (139)
56 (386.1)		10.1 (69.6)		19.5 (134)
60 (413.7)		9.4 (64.8)		18.8 (130)
64 (441.3)		8.8 (60.7)		18.1 (125)
68 (468.9)		8.2 (56.5)		17.4 (120)
72 (496.4)		7.7 (53.1)		16.8 (116)
76 (524)				16.2 (112)
80 (551.6)				15.6 (108)
84 (579.9)				15.0 (103)
88 (606.8)				14.4 (99.3)
92 (620.6)				13.9 (95.8)
96 (661.9)				13.4 (92.4)
100 (689.5)				12.9 (88.9)
104 (717.1)				12.4 (85.5)
108 (744.7)				12.0 (82.7)
112 (772.2)				11.6 (80.0)
116 (799.8)				11.2 (77.2)
120 (827.4)				10.8 (74.5)

TABLE A14. Tangent Moduli for Design of Columns (Types 409, 430, 439)

	Tangent Modulus: E_t, ksi $\times 10^3$ (MPa $\times 10^3$)			
	Type 409		Types 430, 439	
Stress ksi (MPa)	Longitudinal compression	Transverse compression	Longitudinal compression	Transverse compression
0	27.0 (186.2)	29.0 (200)	27.0	29.0
4 (27.6)	27.0	29.0	27.0	29.0
8 (55.2)	27.0	29.0	27.0	29.0
12 (82.7)	26.8 (46.9)	29.0	27.0	29.0
16 (110.4)	25.1 (173.1)	28.9 (199.3)	26.7 (184.1)	29.0
20 (137.9)	17.8 (122.7)	26.9 (185.5)	24.7 (170.3)	29.0
24 (165.5)	7.7 (53.1)	13.6 (93.8)	19.1 (131.7)	29.0
28 (193.1)	2.6 (17.9)	2.4 (16.5)	11.5 (79.3)	28.8 (200.2)
32 (220.6)	0.9 (6.2)	0.4 (2.8)	6.0 (41.4)	25.8 (177.9)
36 (248.2)			3.1 (21.4)	12.0 (82.7)
40 (275.8)			1.6 (11)	2.4 (16.5)
44 (303.4)				0.4 (2.8)

TABLE A15. Tensile Strength of Weld Metal

AWS Classification	Tensile Strength, min. ksi (MPa)
E209	100 (689.5)
E219	90 (620.6)
E240	100
E307	85 (586.1)
E308	80 (551.6)
E308H	80
E308L	75 (571.1)
E308Mo	80
E308MoL	75
E309	80
E309L	75
E309Cb	80
E309Mo	80
E310	80
E310H	90
E310Cb	80
E310Mo	80
E312	95
E316	75
E316H	75
E316L	70
E317	80
E317L	75
E318	80
E320	80
E320LR	75
E330	75
E330H	90
E347	75
E349	100
E410	65 (448.2)
E410NiMo	110 (758.5)
E430	65
E502	60 (413.7)
E505	60
E630	135 (930.8)
E16-8-2	80
E7Cr	60

TABLE A16. Tensile Strengths of Annealed, 1/16 Hard, 1/4 Hard, and 1/2 Hard Base Metals

Types of Stainless Steels	Minimum Tensile Strength, ksi	(MPa)
	Annealed	
201-1	90 (Class 1)	(620.6)
201-2	95 (Class 2)	(655)
S20400	95	(655)
301	90	
304,316	75	(571.1)
409	55	(379.2)
430,439	65	(448.2)
	1/16 Hard	
201 PSS[a]	90	
FB[b]	75	
301	90	
304 PSS	80	(551.6)
FB	90	
316 PSS	85	(586.1)
FB	90	
	1/4 Hard	
201	125	(861.9)
S20400	125	
301	125	
304	125	
316	125	
	1/2 Hard	
201	150	(1,034.3)
301	150	
304	150	
316	150	

[a] PSS = plate, sheet, and strip.
[b] FB = flat bar.

TABLE A17. Ratio of Effective Proportional Limit-to-Yield Strength

Type of Stress	Effective Proportional Limit/Yield Strength (F_{pr}/F_y) Types 201, 301, 304, 316: Annealed and 1/16 Hard	Types 201, 301, 304, 316: 1/4 Hard	Types 201, 301, 304, 316: 1/2 Hard	UNS S20400: Annealed	UNS S20400: 1/4 Hard	Type 409	Types 430, 439
Longitudinal tension	0.67	0.50	0.45	0.49	0.29	0.76	0.70
Transverse tension	0.57	0.55	0.60	0.61	0.30	0.83	0.81
Transverse compression	0.66	0.50	0.50	0.73	0.33	0.83	0.82
Longitudinal compression	0.46	0.50	0.49	0.53	0.32	0.73	0.62

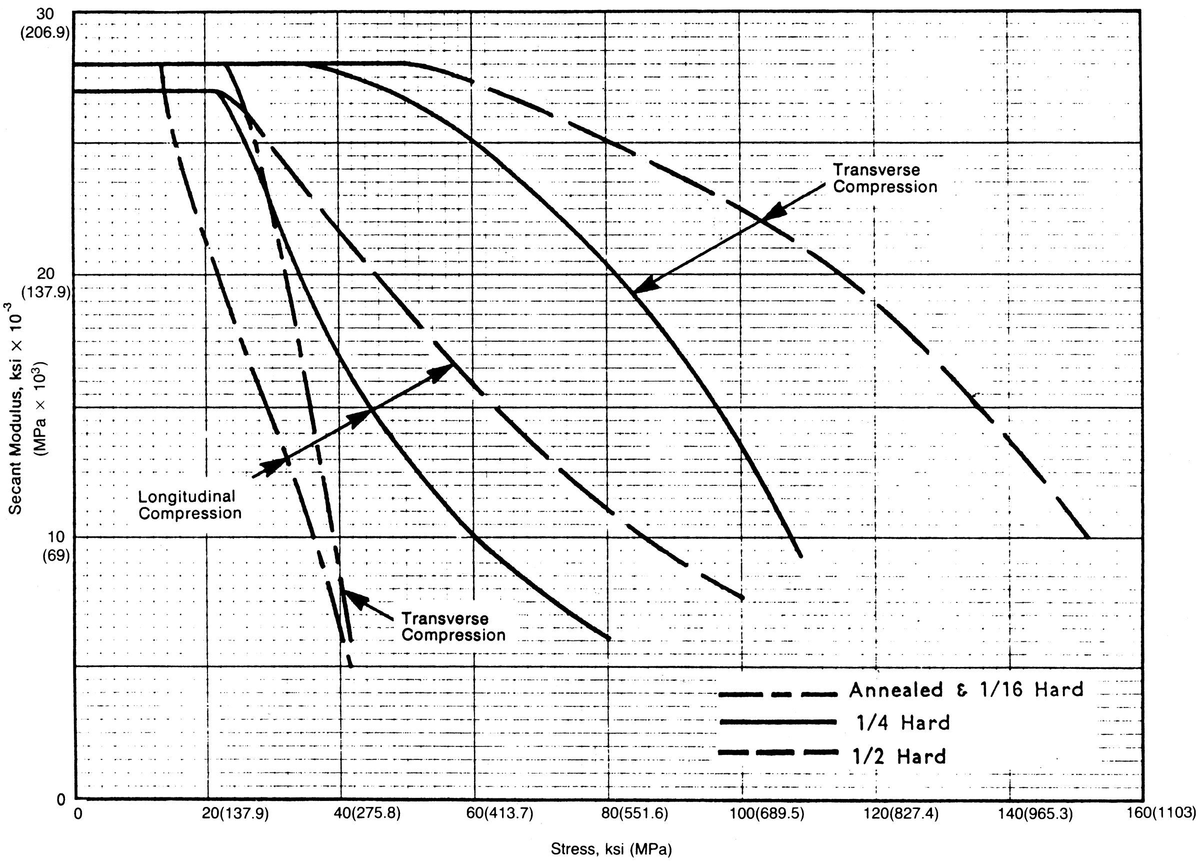

FIGURE A1a. Secant Moduli for Deflection Calculations (Types 201, 301, 304, and 316)

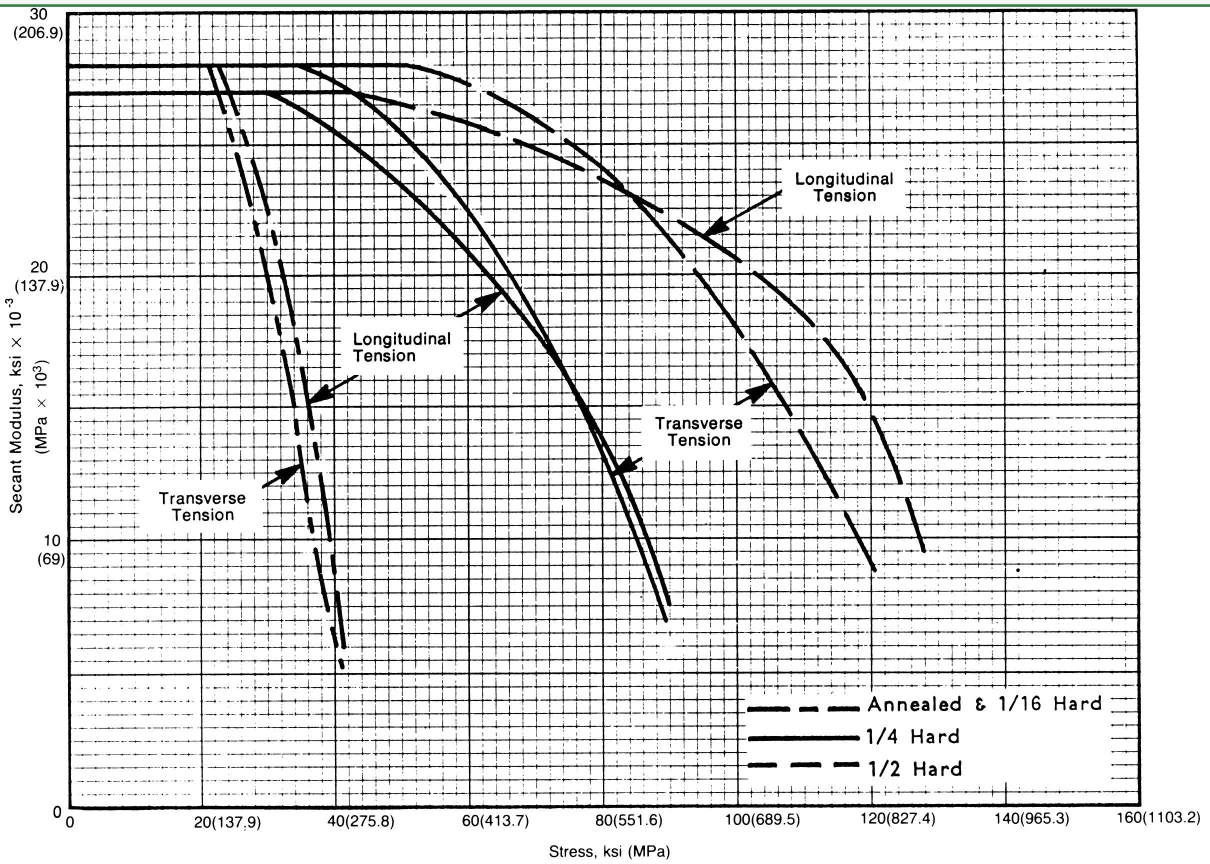

FIGURE A1a. ***Continued***

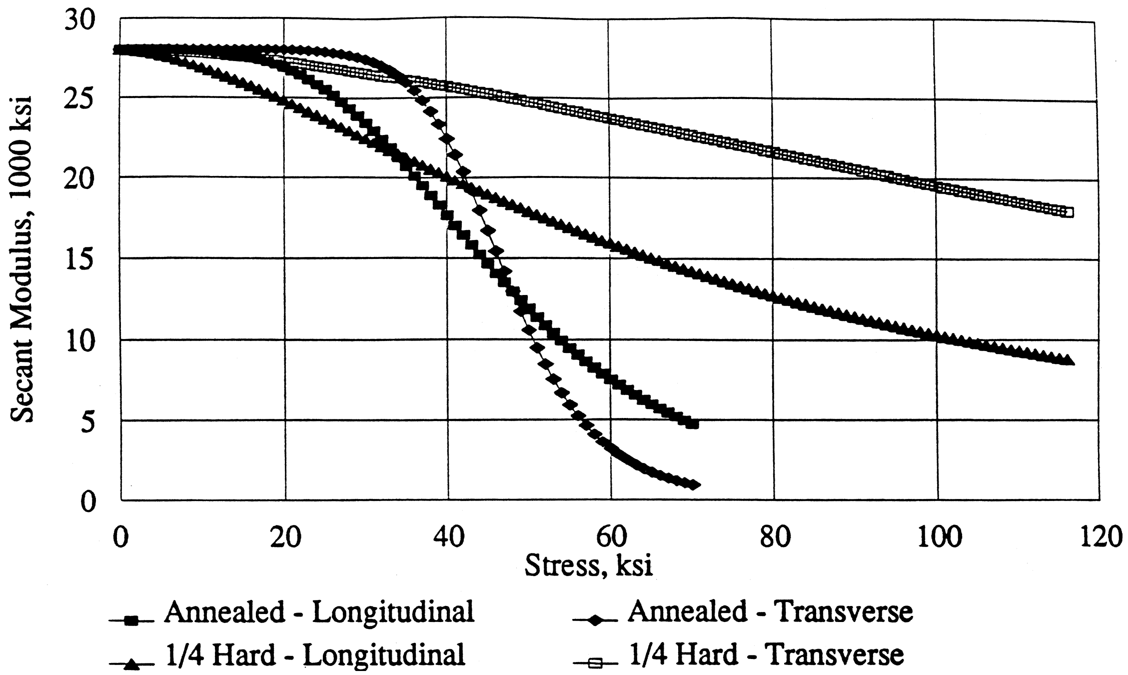

FIGURE A1b. Secant Moduli for Deflection Calculations (S20400, Compression)

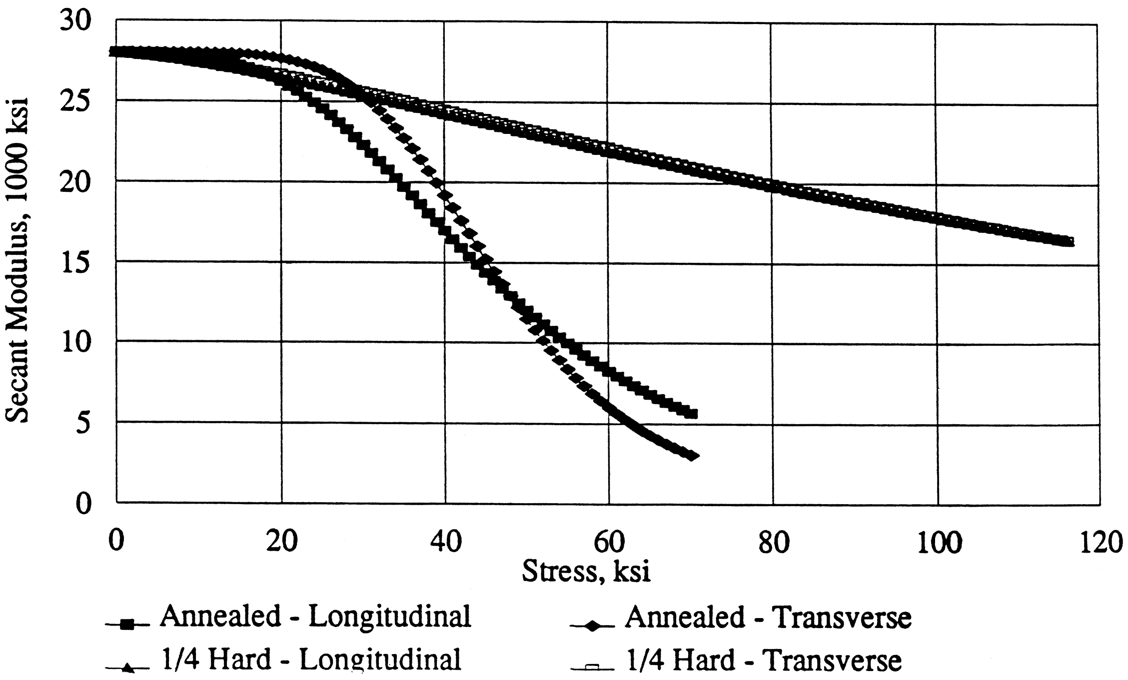

FIGURE A1b. ***Continued*** **(S20400, Tension)**

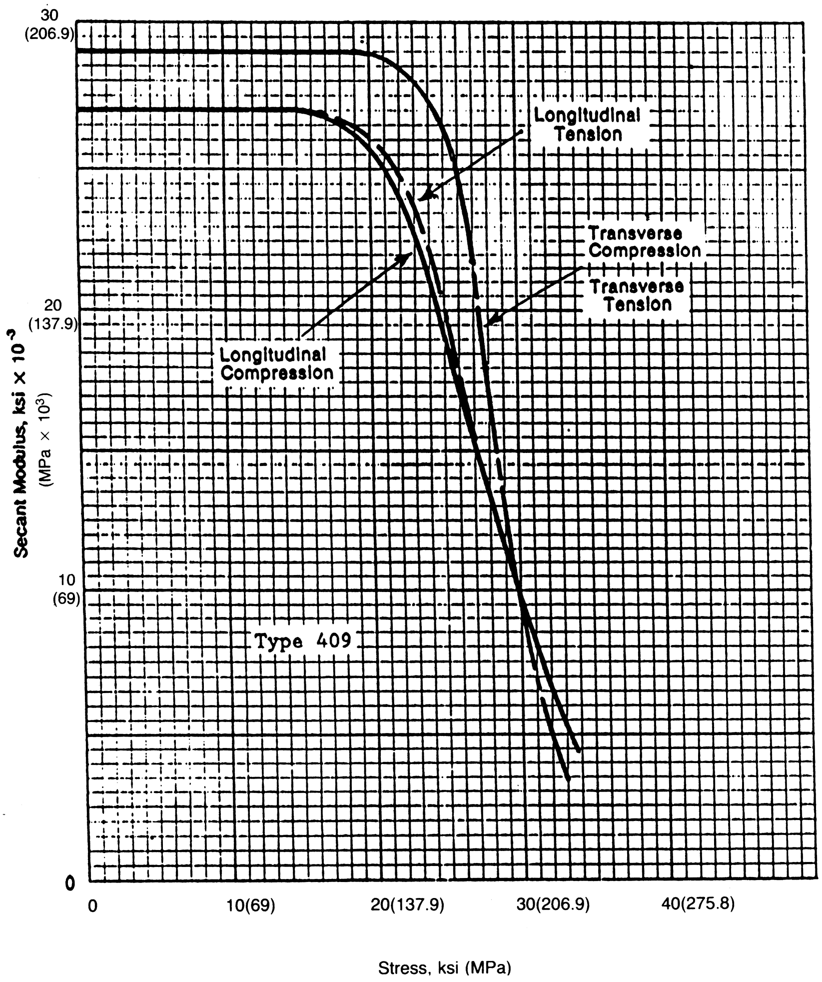

FIGURE A2. Secant Moduli for Deflection Calculations (Types 409, 430, and 439)

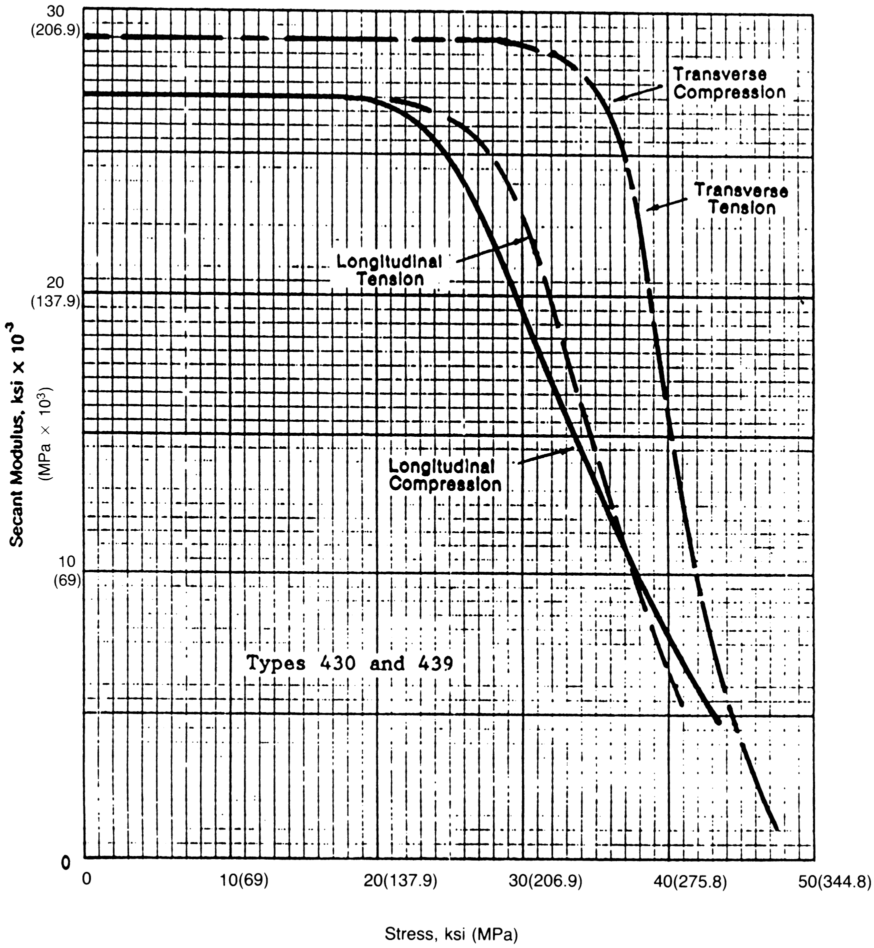

FIGURE A2. *Continued*

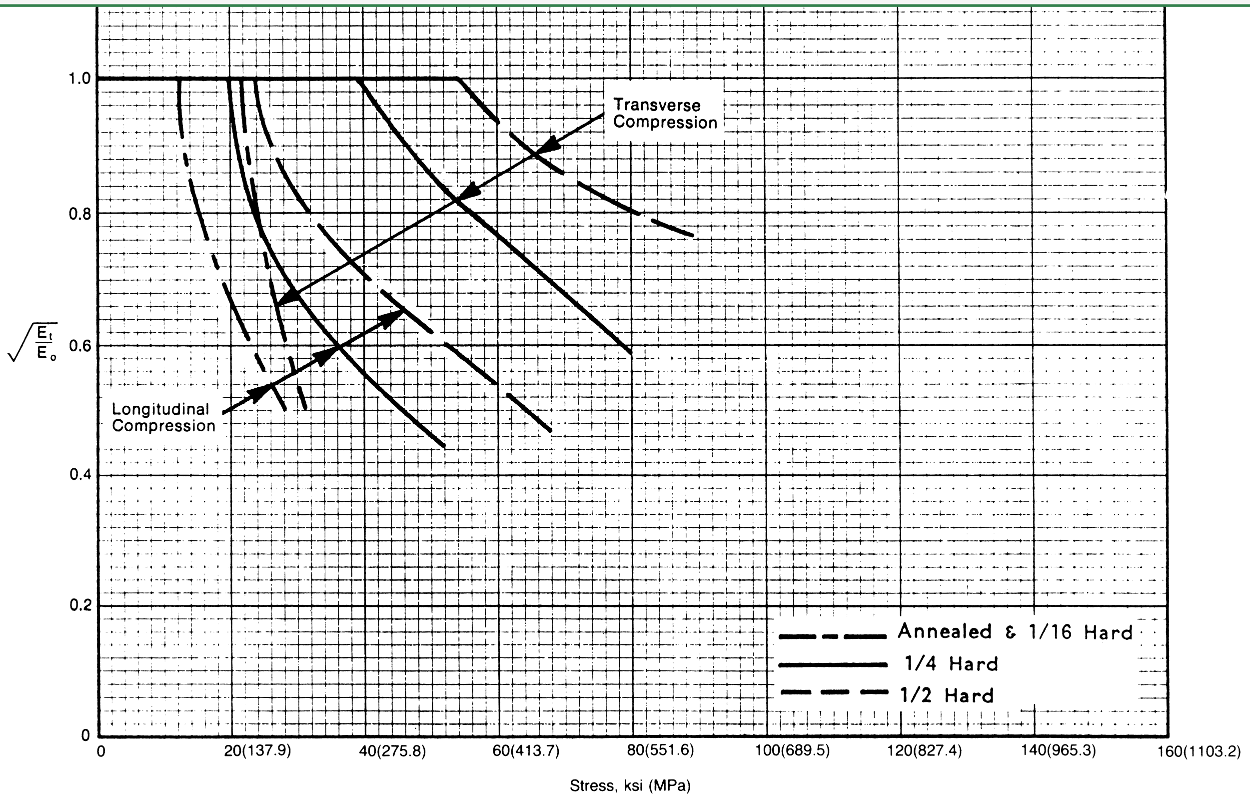

FIGURE A3a. Plasticity Reduction Factors for Stiffened Compression Elements (Types 201, 301, 304, and 316)

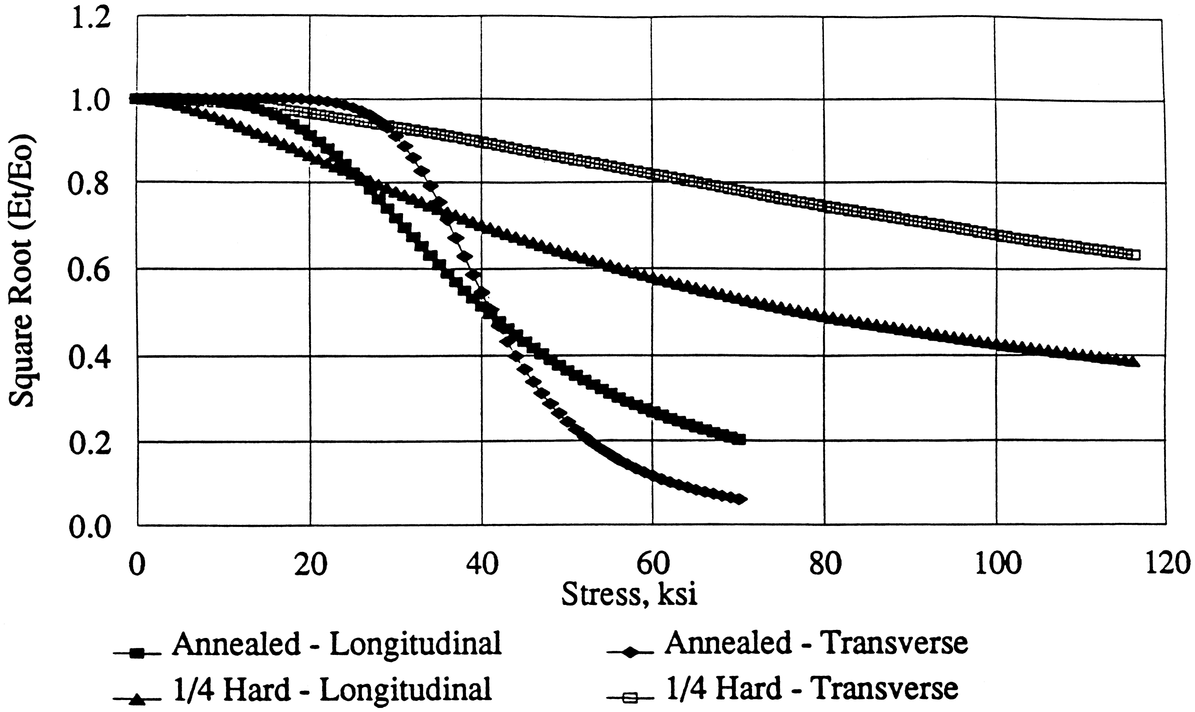

FIGURE A3b. Plasticity Reduction Factors for Stiffened Compression Elements (S20400)

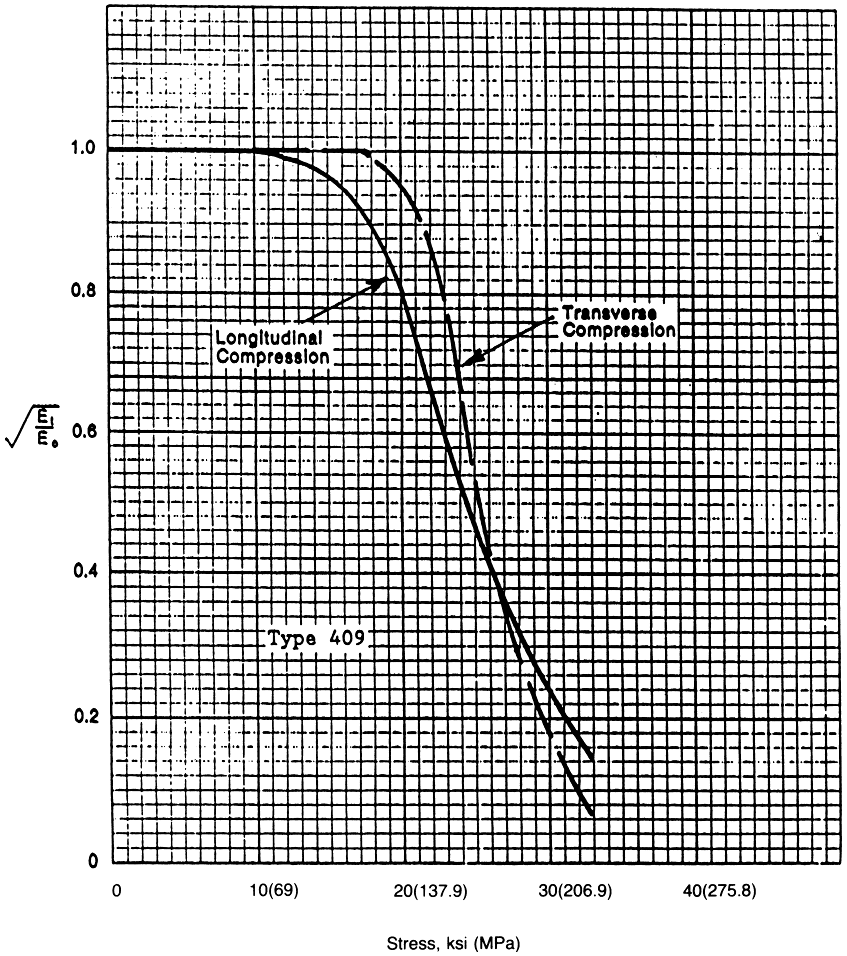

FIGURE A4. Plasticity Reduction Factors for Stiffened Compression Elements (Types 409, 430, and 439)

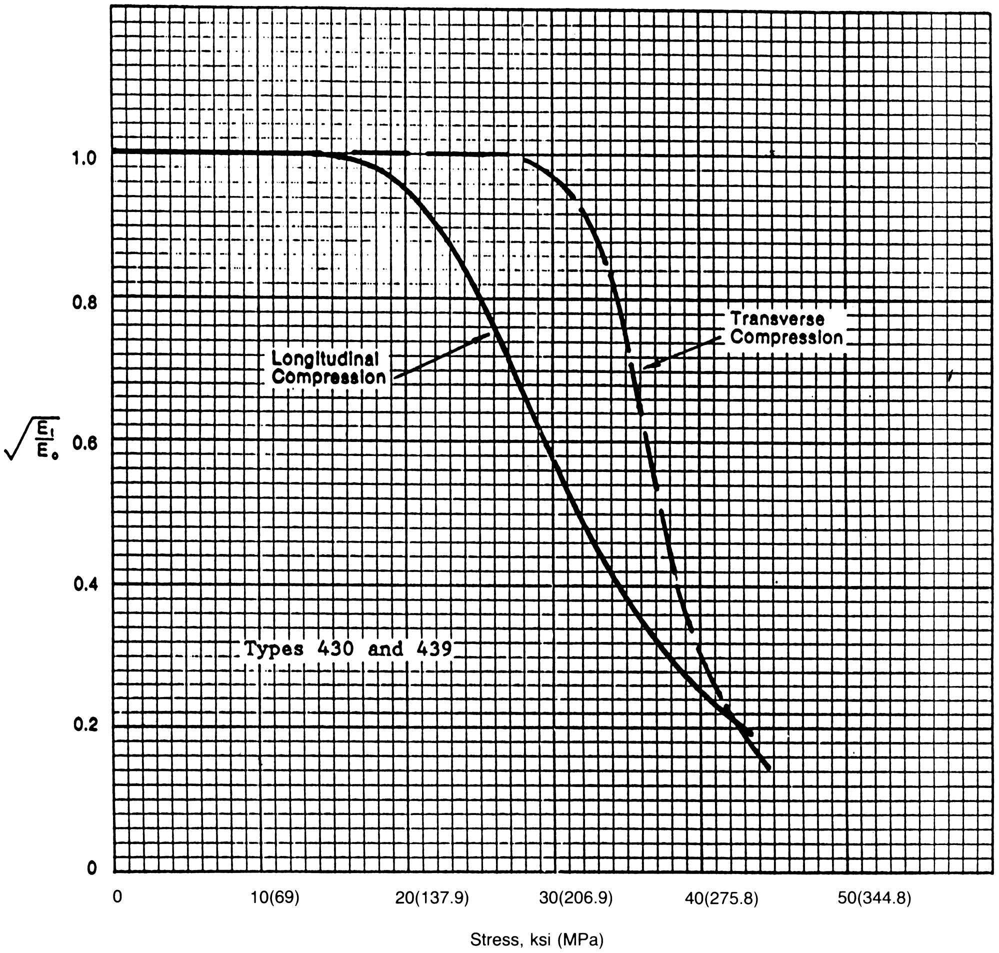

FIGURE A4. ***Continued***

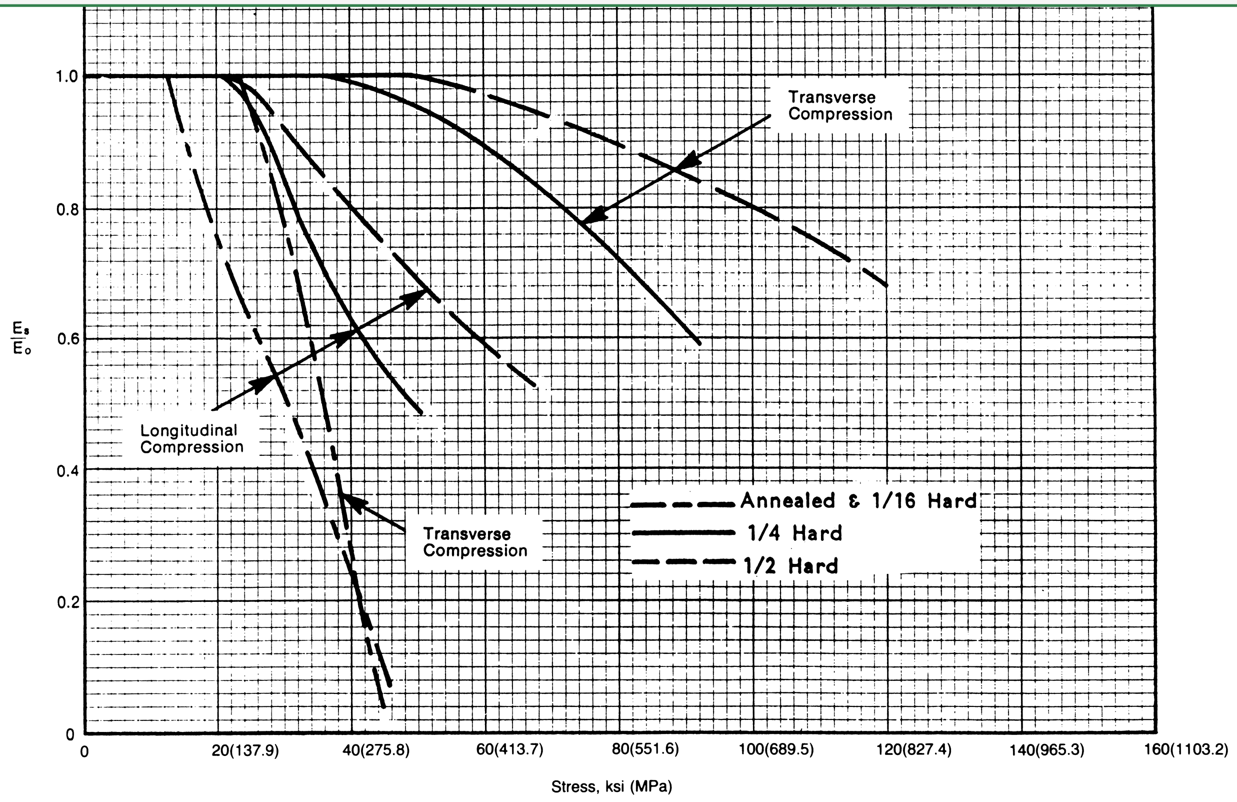

FIGURE A5a. Plasticity Reduction Factors for Unstiffened Compression Elements (Types 201, 301, 304, and 316)

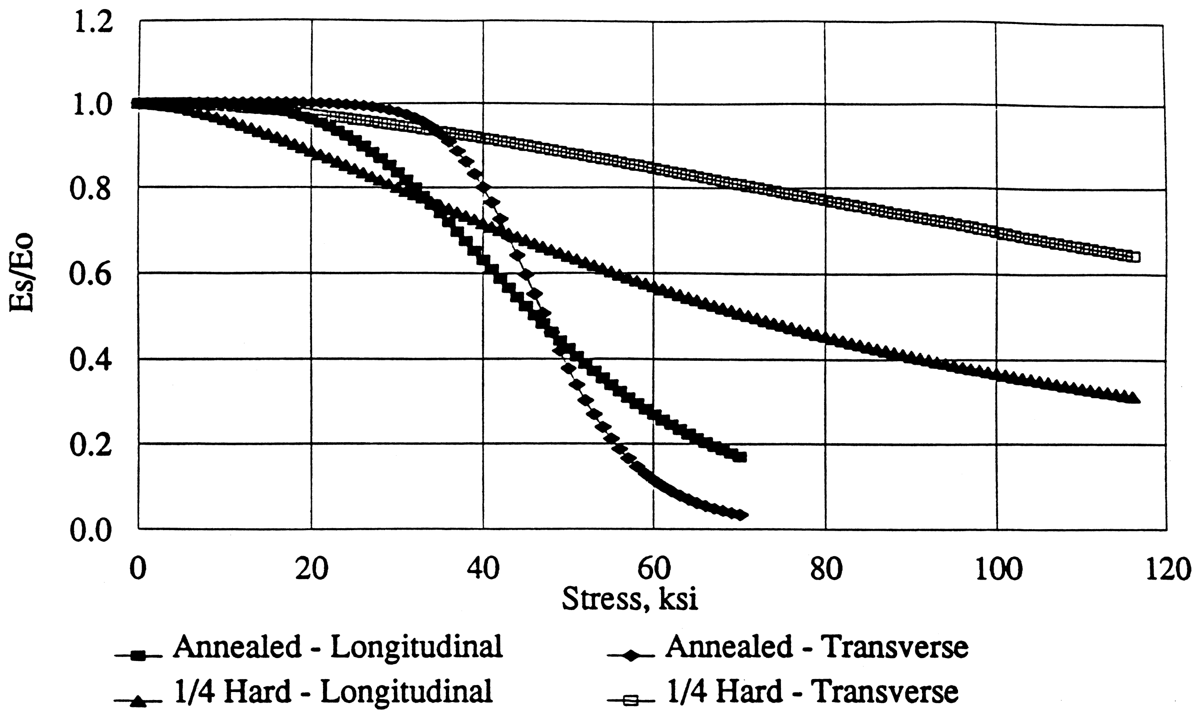

FIGURE A5b. Plasticity Reduction Factors for Unstiffened Compression Elements (S20400)

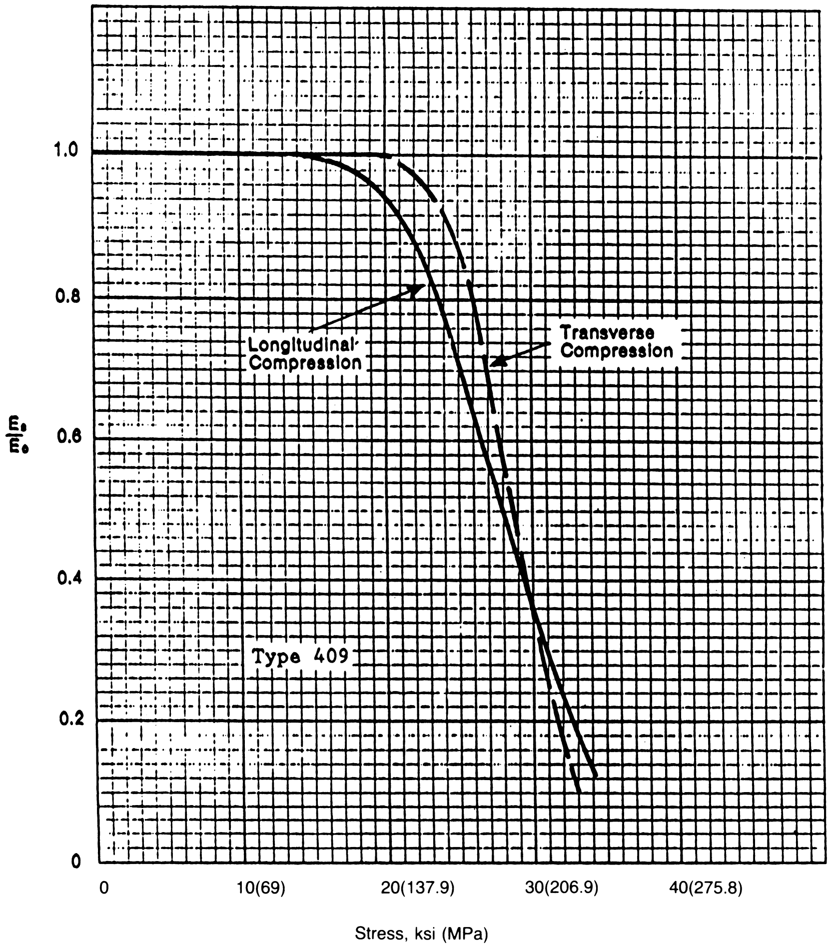

FIGURE A6. Plasticity Reduction Factors for Unstiffened Compression Elements (Types 409, 430, and 439)

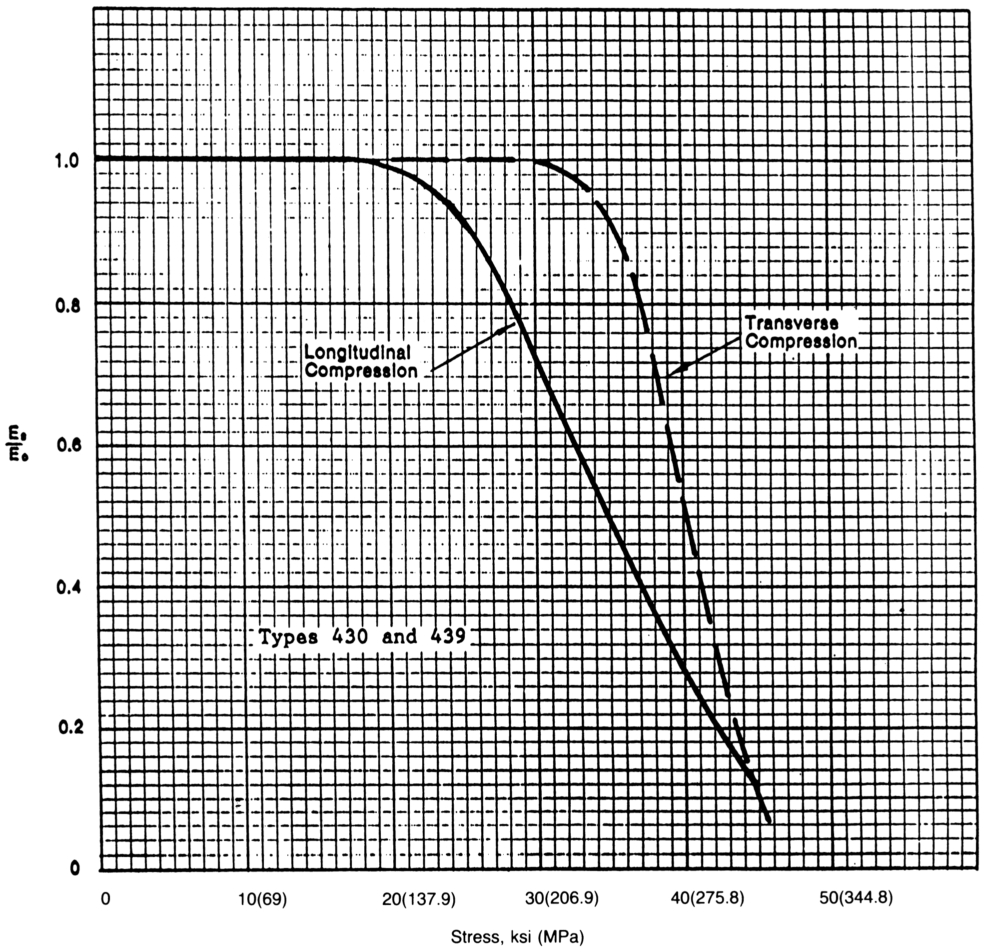

FIGURE A6. *Continued*

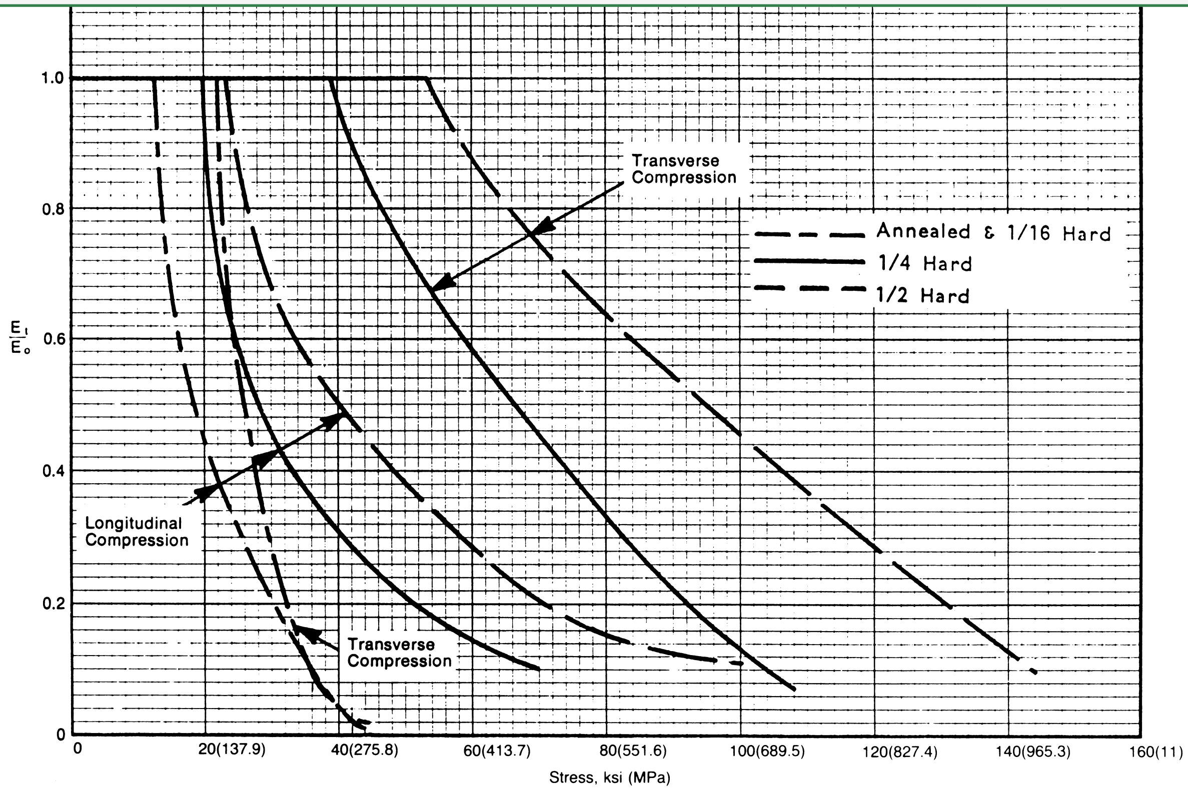

FIGURE A7a. Plasticity Reduction Factors For Design of Laterally Unbraced Single Web Beams (Types 201, 301, 304, and 316)

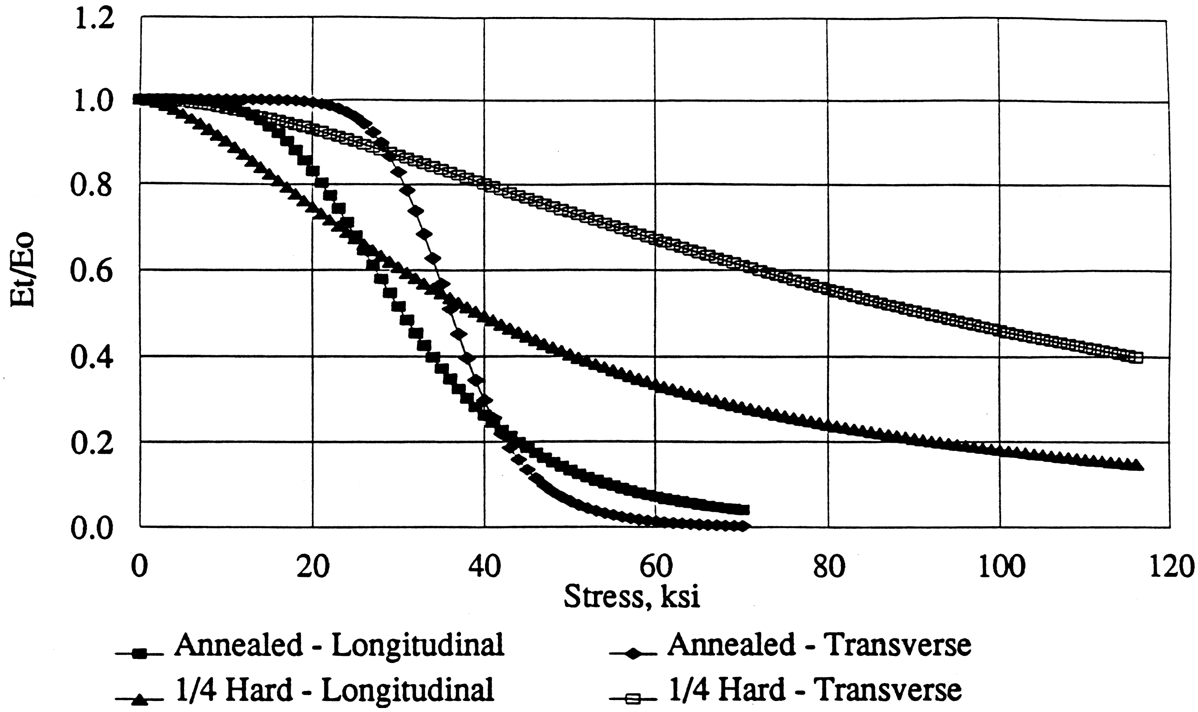

FIGURE A7b. Plasticity Reduction Factors For Design of Laterally Unbraced Single Web Beams (S20400)

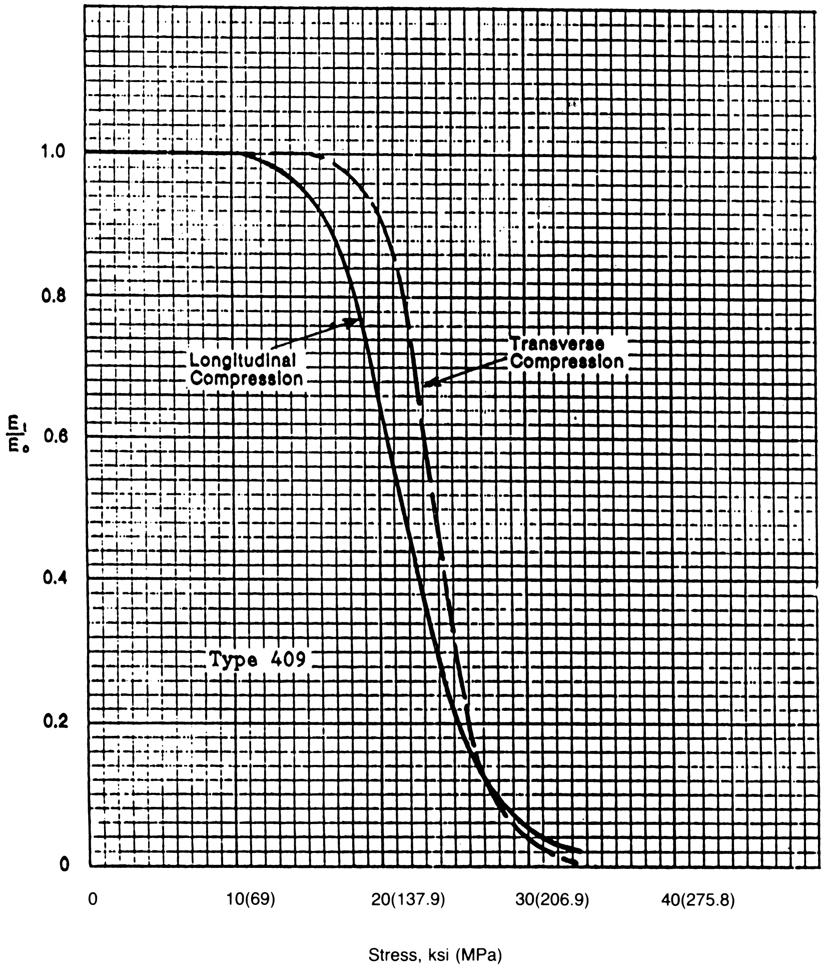

FIGURE A8. Plasticity Reduction Factors for Design of Laterally Unbraced Single Web Beams (Types 409, 430, and 439)

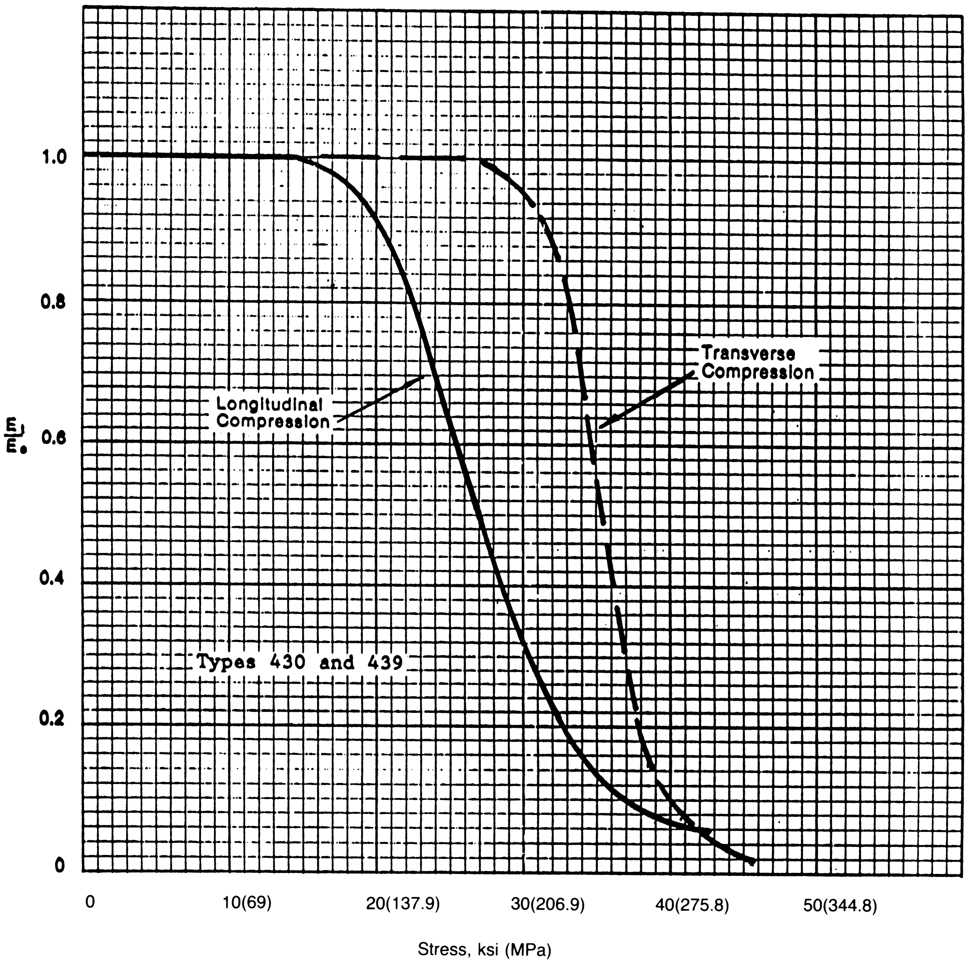

FIGURE A8. *Continued*

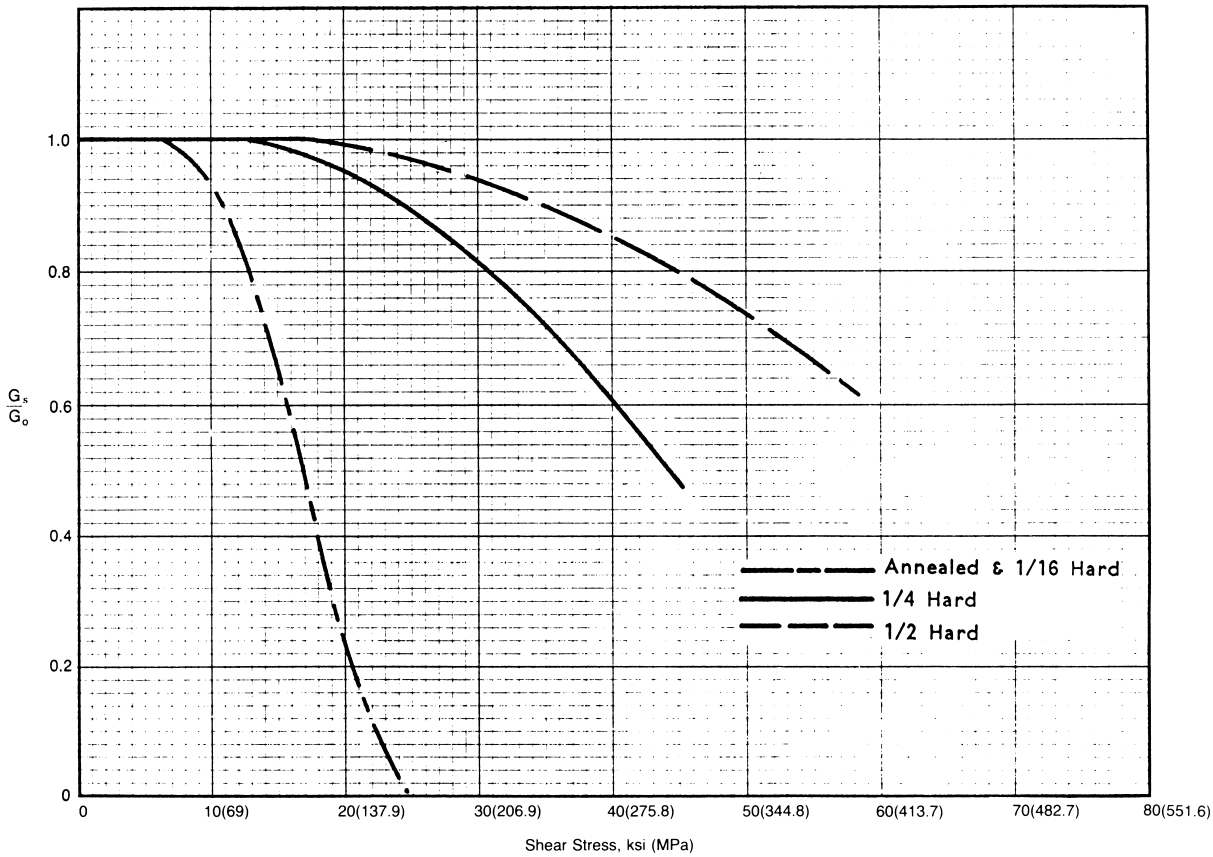

FIGURE A9. Plasticity Reduction Factors for Shear Stresses in Webs (Types 201, 301, 304, and 316)

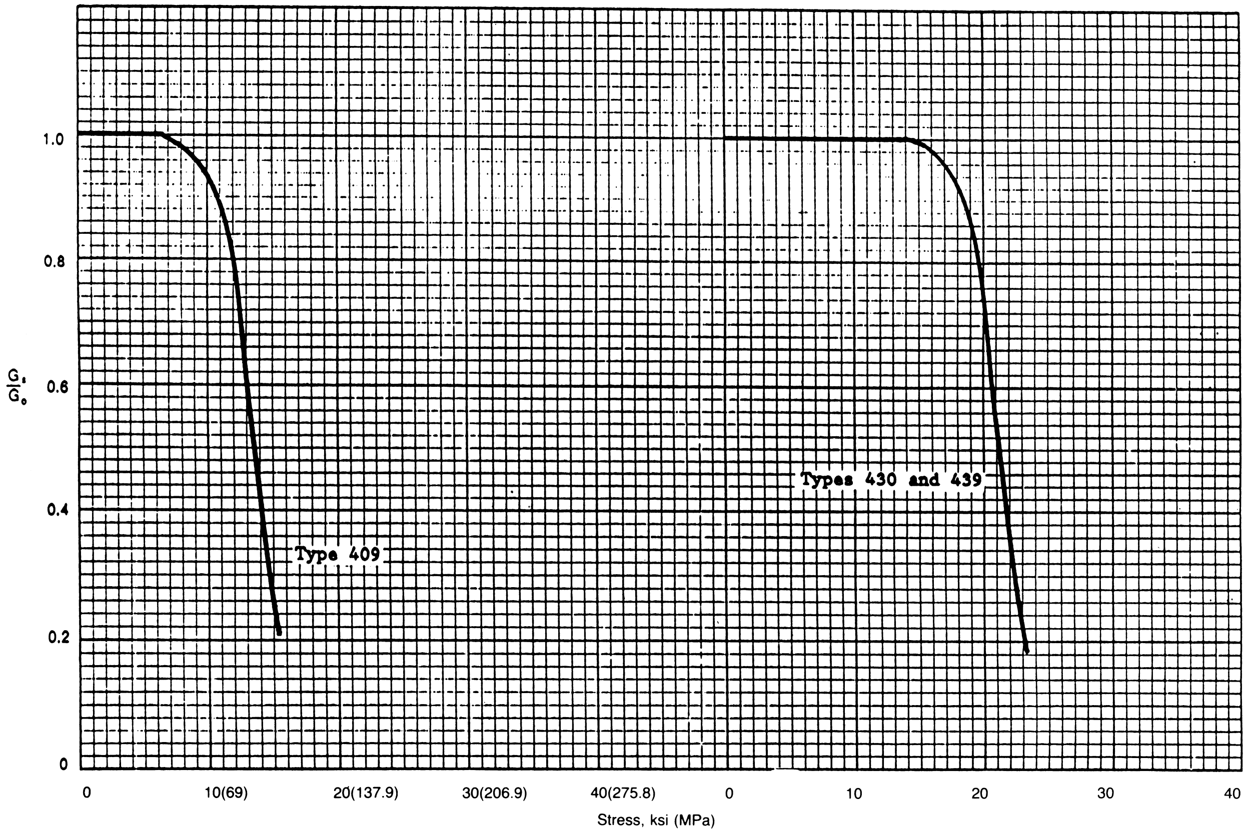

FIGURE A10. Plasticity Reduction Factors for Shear Stresses in Webs (Types 409, 430, and 439)

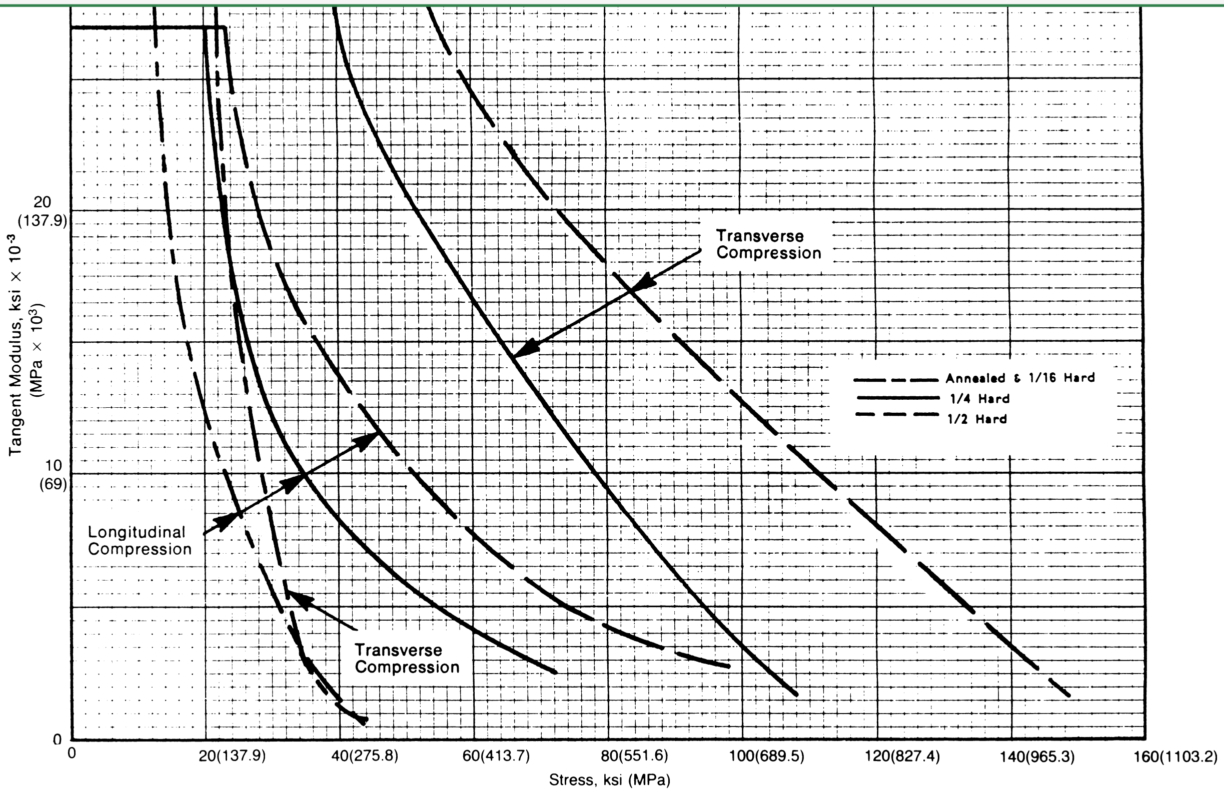

FIGURE A11a. Tangent Moduli for Design of Columns (Types 201, 301, 304, and 316)

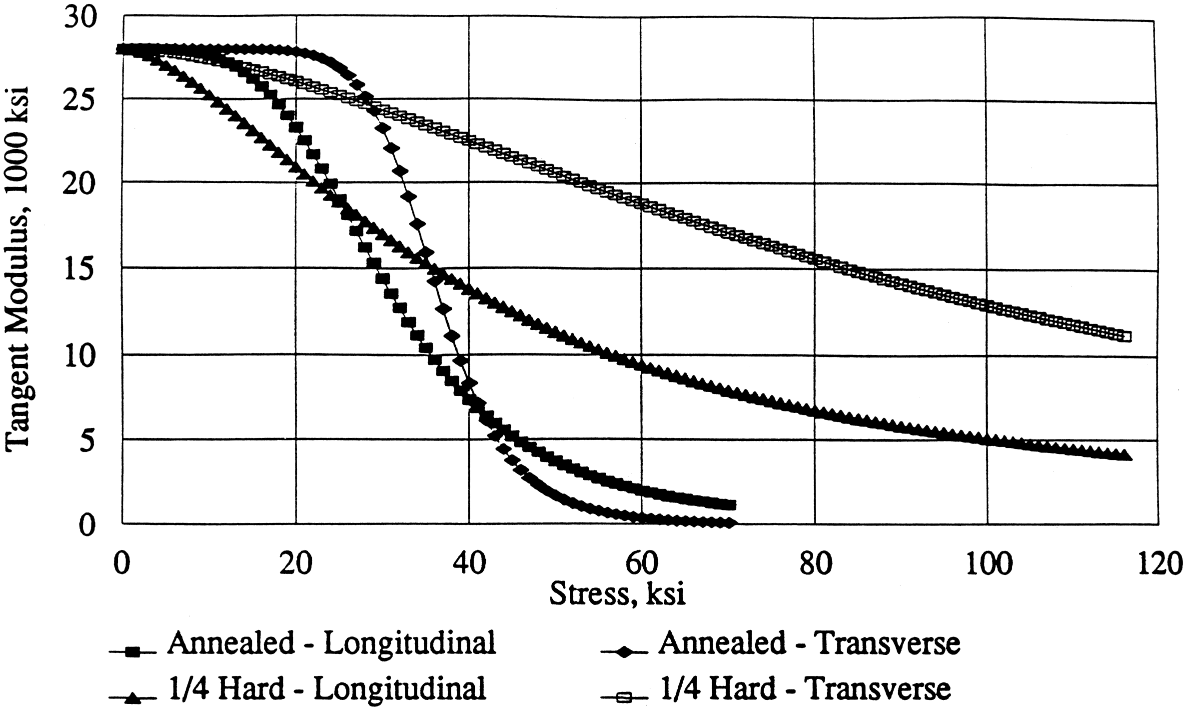

FIGURE A11b. Tangent Moduli for Design of Columns (Type S20400)

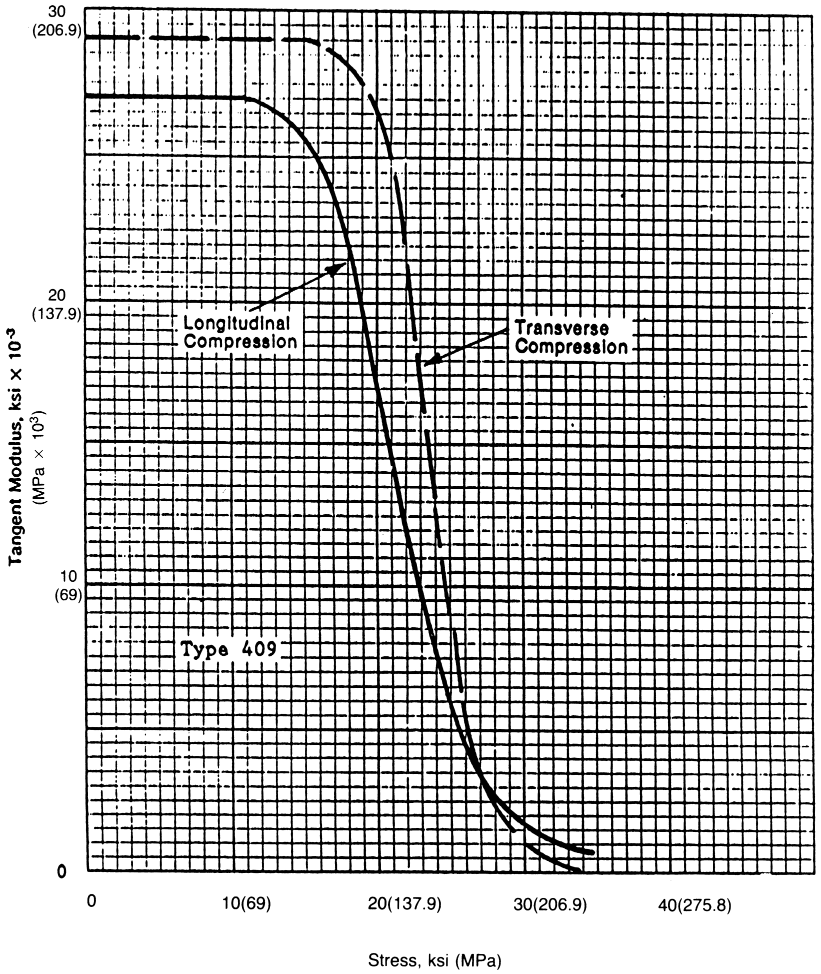

FIGURE A12. Tangent Moduli for Design of Columns (Types 409, 430, and 439)

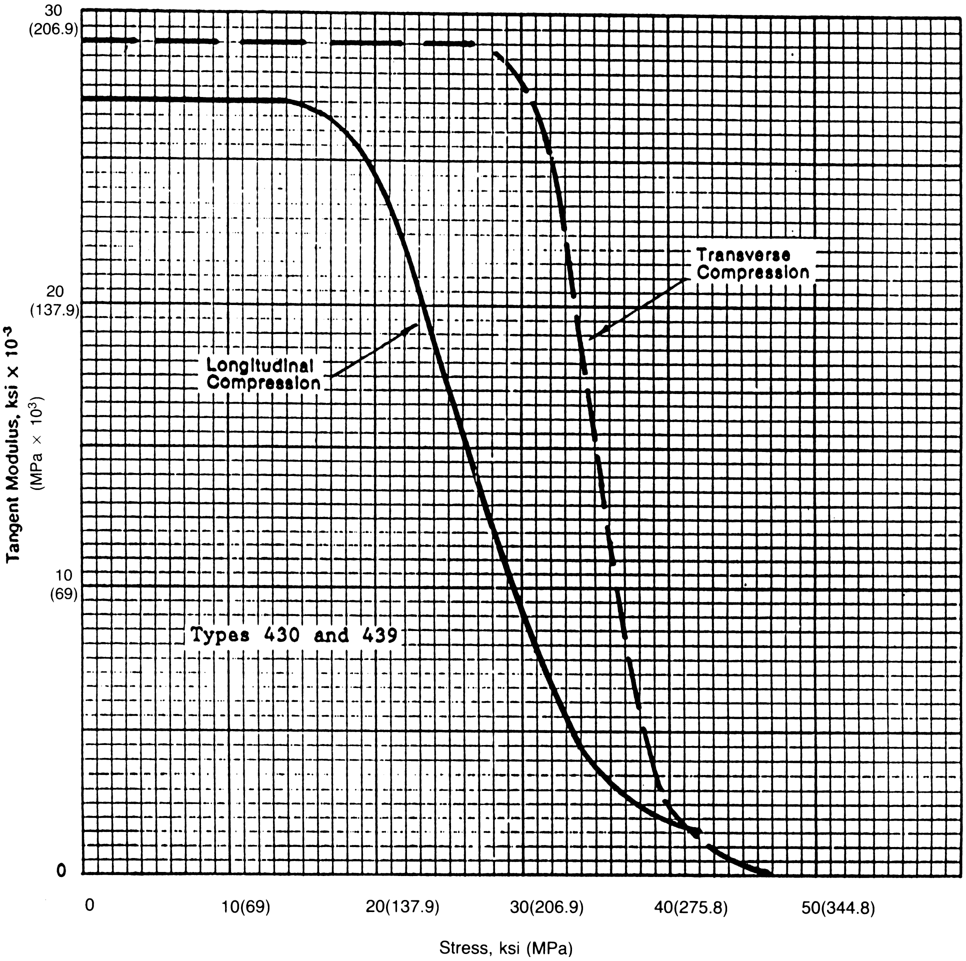

FIGURE A12. ***Continued***

TABLE B. Coefficient n Used for Modified Ramberg-Osgood Equation

Types of Stress	Types 201, 301, 304, 316 Annealed and 1/16 Hard	1/4 Hard	1/2 Hard	UNS S20400 Annealed	1/4 Hard	Type 409	Types 430 and 439
Longitudinal tension	8.31	4.58	4.21	4.24	2.42	10.77	8.43
Transverse tension	7.78	5.38	6.71	6.14	2.49	15.75	14.13
Transverse compression	8.63	4.76	4.54	9.49	2.70	15.76	14.30
Longitudinal compression	4.10	4.58	4.22	4.79	2.61	9.70	6.25

For buckling stress of columns and lateral buckling strength of beams

$$\eta = E_t/E_o = \frac{F_y}{F_y + 0.002n\, E_o \left(\frac{\sigma}{F_y}\right)^{n-1}} \quad \text{(B-5)}$$

In these equations:

σ = normal stress;
ε = normal strain;
E_o = initial modulus of elasticity;
F_y = 0.2% offset yield strength; and
n = coefficient, as given in Table B.

APPENDIX C: STIFFENERS

C.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 Section 5. 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 (1) and (2) as follows:

1. $P_n = F_{yw} A_c$ (C-1)
2. P_n = nominal axial strength evaluated according to Section 3.4 with A_e replaced by A_b

where:

$A_c = 18t^2 + A_s$, for transverse stiffeners at interior support and under concentrated load (C-2)
$A_c = 10t^2 + A_s$, for transverse stiffeners at end support (C-3)
F_{yw} = lower value of yield strength in beam web F_y or stiffener section F_{ys}
$A_b = b_1 t + A_s$, for transverse stiffeners at interior support and under concentrated load (C-4)
$A_b = b_2 t + A_s$, for transverse stiffeners at end support (C-5)
A_s = cross-sectional area of transverse stiffeners
$b_1 = 25t\,(0.0024\,(L_{st}/t) + 0.72) \leq 25t$ (C-6)
$b_2 = 12t\,(0.0044\,(L_{st}/t) + 0.83) \leq 12t$ (C-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 $1.28\sqrt{(E_o/F_{ys})}$ and $0.37\sqrt{(E_o/F_{ys})}$, respectively, where F_{ys} = the yield strength of stiffener steel and t_s the thickness of the stiffener steel.

C.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 3.3.2, 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

$$I_{smin} = 5ht^3(h/a - 0.7\,(a/h)) \geq (h/50)^4 \quad \text{(C-8)}$$

The gross area of shear stiffeners shall be not less than

$$A_{st} = \left(\frac{(1 - C_v)}{2}\right) \times \left[\frac{a}{h} - \frac{(a/h)^2}{(a/h) + \sqrt{1 + (a/h)^2)}}\right] \times YDht \quad \text{(C-9)}$$

where:

$$C_v = \frac{45{,}000 k_v}{F_y \left(\frac{h}{t}\right)^2} \quad \text{when} \quad C_v \leq 0.8 \quad \text{(C-10)}$$

$$C_v = \frac{190}{\left(\frac{h}{t}\right)} \sqrt{\frac{k_v}{F_y}} \quad \text{when} \quad C_v > 0.8 \tag{C-11}$$

$$k_v = 4.00 + \frac{5.34}{\left(\frac{a}{h}\right)^2} \quad \text{when} \quad \frac{a}{h} \leq 1.0 \tag{C-12}$$

$$k_v = 5.34 + \frac{4.00}{\left(\frac{a}{h}\right)^2} \quad \text{when} \quad \frac{a}{h} > 1.0 \tag{C-13}$$

a = distance between transverse stiffeners;
Y = ratio of yield strength in web to yield strength in stiffener;
D = 1.0 for stiffeners furnished in pairs;
D = 1.8 for single-angle stiffeners;
D = 2.4 for single-plate stiffeners;
h = depth of flat portion of web measured along plane of web; and
t = web thickness.

C.3 Nonconforming Stiffeners

The design strength of members with transverse stiffeners that do not meet the requirements of Appendices C.1 and C.2, such as stamped or rolled-in transverse stiffeners shall be determined by tests in accordance with Section 6 of this Specification.

APPENDIX D: ALLOWABLE STRESS DESIGN (ASD)

Allowable stress design shall be based on the allowable design strength determined in accordance with Equation D-1:

$$R_a = \frac{R_n}{\Omega} \tag{D-1}$$

where:

R_a = allowable design strength;
R_n = nominal strength specified in Sections 2 through 5; and
Ω = safety factors specified in Table D.

All resistance factors, ϕ, specified in Sections 2 through 5 shall be taken as unity. For combined bending and shear (Section 3.3.3), combined bending and web crippling (Section 3.3.5), and combined axial load and bending (Section 3.5), the nominal strength used in the interaction formulas shall be replaced by the allowable design strength.

The required strength for allowable stress design shall be determined for the load combinations as specified in Section 2.3 of the American Society of Civil Engineers Standard ANSI/ASCE 7-88 Minimum Design Loads for Buildings and Other Structures but using the corresponding nominal load symbols D_n, L_n, L_{rn}, S_n, R_{rn}, W_n, E_n, T_n of this standard.

TABLE D. Safety Factors by Subjects and Sections of the Specification for the Design of Cold-Formed Stainless Steel Structural Members

Subject	Section	Safety Factor, Ω
Tension member	3.2	1.85 against yielding
Flexural members		
Bending only	3.3.1	1.85 against yielding and buckling
Shear only	3.3.2	1.64 against shear yielding 1.85 against shear buckling
Web crippling	3.3.4	2.0 for single unreinforced webs 2.20 for I-beams
Concentrically loaded compression members	3.4	2.15 against column buckling
Arc-and-tangent corrugated sheets	3.7	1.85 against yielding
Fusion welds	5.2	1.85 against yielding of base metal 2.50 against ultimate test value of welds
Resistance welds	5.2.3	2.50 against ultimate test value of welds
Bolted connections	5.3	
Spacing and edge distance	5.3.1	2.40 against sheet shearing
Tension on net section	5.3.2	2.40 when washers are provided under bolt head and nut
Bearing	5.3.3	2.40 for single and double shear connections
Bolt shear	5.3.4	3.0 against shear failure of bolt
Bolt tension	5.3.4	3.0 against tensile strength of bolt

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INTRODUCTION

The "Allowable Stress Design" method has long been used for the design of steel structures in the United States.[1,2,3] Recently, the probability-based Load and Resistance Factor Design (LRFD) criteria have been successfully applied to the structural design of hot-rolled steel shapes and built-up members.[4] The AISI LRFD Specification is being developed as well for the design of structural members cold-formed from carbon and low alloy steels.[5] The LRFD criteria offer an improved approach for the design of steel structures because they involve probabilistic considerations for uncertain variables in the design formulas.

Due to the significant difference in material properties between stainless and carbon steels and the recent development of the design methodology, a new specification for load and resistance factor design of cold-formed stainless steel structural members is needed. Since July 1986, a research project entitled "Load and Resistance Factor Design of Cold-Formed Stainless Steel" has been conducted at the University of Missouri-Rolla to develop the new design criteria for cold-formed stainless steel structural members and connections based on the probabilistic approach.[76]

The Load and Resistance Factor Design criteria included in this ASCE Standard were developed on the basis of the first-order probabilistic theory by using only the mean value and coefficient of variation of load effects, material factors, fabrication factors, and professional factors. They can provide a more uniform overall safety and structural reliability. The design provisions for cold-formed stainless steel structural members, assemblies, and connections are included in Sections 3 through 5 of this Specification, respectively.

This Commentary contains a brief explanation 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.

In order to permit the use of the allowable stress design method for the design of cold-formed stainless steel structural members, appropriate safety factors are provided in Appendix D of this Standard Specification.

1. GENERAL PROVISIONS

1.1 Limits of Applicability and Terms

1.1.1 Scope and Limits of Applicability

This Specification is limited to the use of four types of austenitic stainless steels (Types 201, 301, 304, and 316, annealed and cold-rolled in 1/16, 1/4, and 1/2 Hard) and three types of ferritic stainless steels (Types 409, 430, and 439, annealed) for structural members cold-formed to shapes from sheet, strip, plate, or flat bar. The forming process is carried out at, or near, room temperature by the use of bending brakes, press brakes, or roll-forming machines.

The design provisions are developed primarily for cold-formed stainless steel structural members used for buildings. They may also be used for structures other than buildings, provided that appropriate allowances are made for thermal and/or dynamic effects. The general information on impact-loading of thin-walled beams and columns can be found in Reference 3.

Some of the significant differences in material properties of cold-formed stainless steels as compared with carbon steels are: (1) Pronounced anisotropic characteristics; (2) difference in stress-strain relationships for different grades of stainless steels; (3) low proportional limits; and (4) pronounced response to cold work. Due to these significant differences, the LRFD Specifications for hot-rolled steel sections[4] and cold-formed steel members[5] do not apply to the design of cold-formed stainless steel structural members.

1.1.2 Terms

Many of the definitions used in Section 1.1.2 are self-explanatory. The following discussion intends to clarify the meaning of some terms used in the Specification.

a. *Stiffened or Partially Stiffened Compression Elements.* "Stiffened compression element" is a flat compression element supported along both edges parallel to the direction of stress. Figure C1 shows various shapes used as flexural members and compression members which contain stiffened compression elements. Shapes 1 and 2 each have a web and a lip to stiffen the compression flange. For design purpose, the ineffective portion is shown shaded. For the explanation of these ineffective portions, see Item *d* below on Effective Design Width and Section 2. Shapes 3, 4, and 5 show compression flanges stiffened by two