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 Système International (SI) units and customary units.

ASCE

SEI

Structural Engineering Institute

SEI/ASCE 8-02

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SEI 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 7-88 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

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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The American Society of Civil Engineers (ASCE) acknowledges the devoted efforts of Wei-Wen Yu, Theodore V. Galambos, and Shin-Hua Lin for developing this Standard Specification. Appreciation is expressed to the American Iron and Steel Institute for relinquishing to ASCE the 1974 edition of AISI Specification for the Design of Cold-Formed Stainless Steel Structural Members for revision and publication as an ASCE standard.

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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* Reproduced with permission of the American Iron and Steel Institute from AISI Specification for the Design of Cold-Formed Steel Structural Members with Commentary (1986).

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

* Reproduced with permission of the American Iron and Steel Institute from AISI Specification for the Design of Cold-Formed Steel Structural Members with Commentary (1986).

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

Specification for the Design of Cold-Formed Stainless 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: 1.3*F*, 1.6*H*, 1.2*P*, and 1.2*T*, 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

* 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: 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:

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

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

where:

w = flat width as shown in Figure 1

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

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.

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:

$$k = 4 + 2(1 - \psi)^3 + 2(1 - \psi) \qquad (2.2.2\text{-}4)$$

$\psi = f_2/f_1$

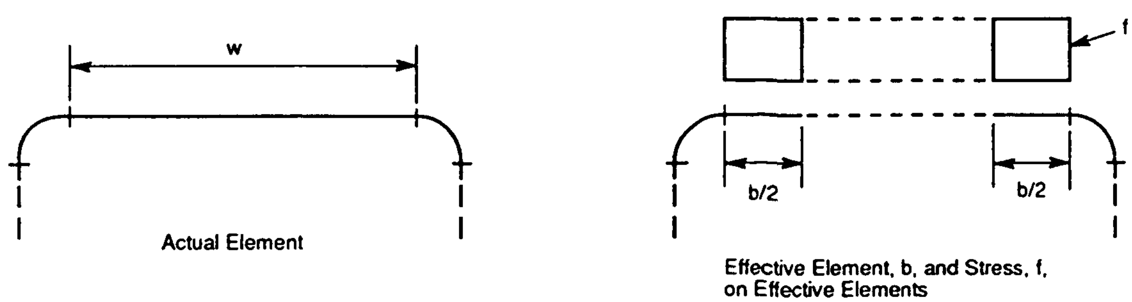


FIGURE 1. Stiffened Elements with Uniform Compression

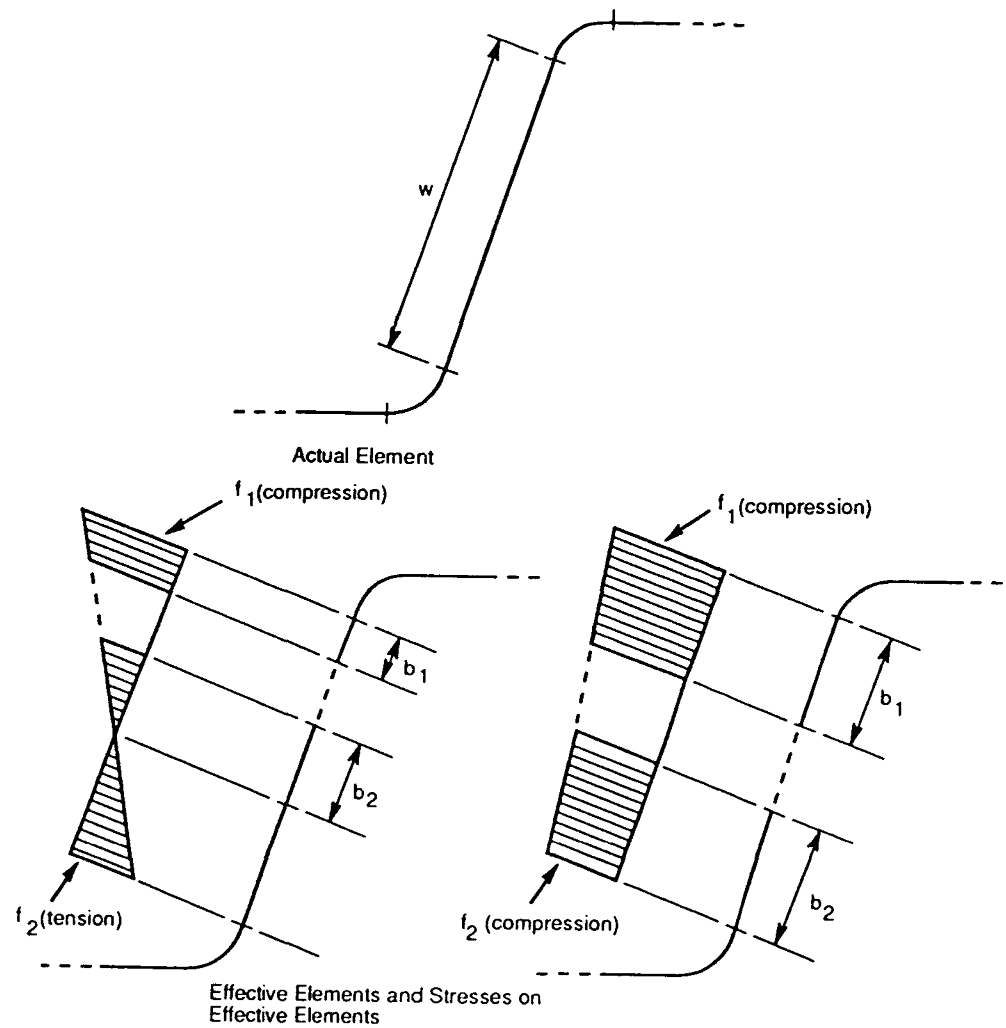


FIGURE 2. Stiffened Elements with Stress Gradient and Webs

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-

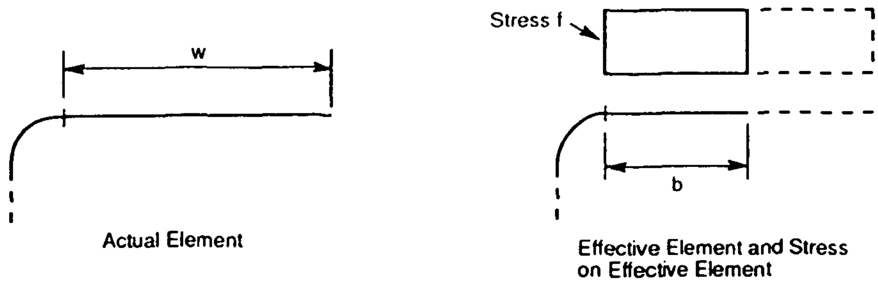


FIGURE 3. Unstiffened Elements with Uniform Compression

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:

$S = 1.28\sqrt{E_o/f}$; (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.

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

$$A'_s = d'_s t \quad (2.4\text{-}3)$$

2.4.1 Uniformly Compressed Elements with Intermediate Stiffener

1. *Load Capacity Determination.*

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

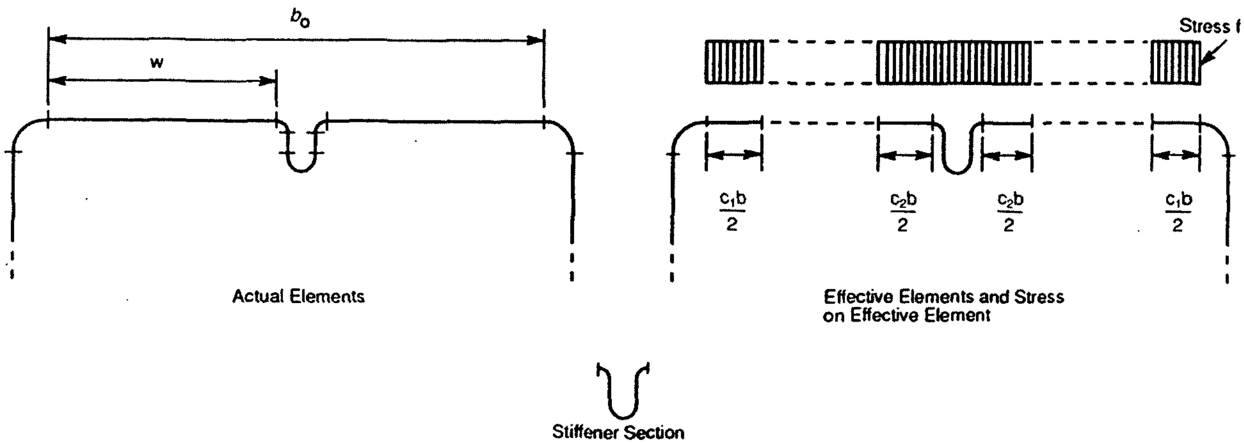


FIGURE 4. Elements with Intermediate Stiffener

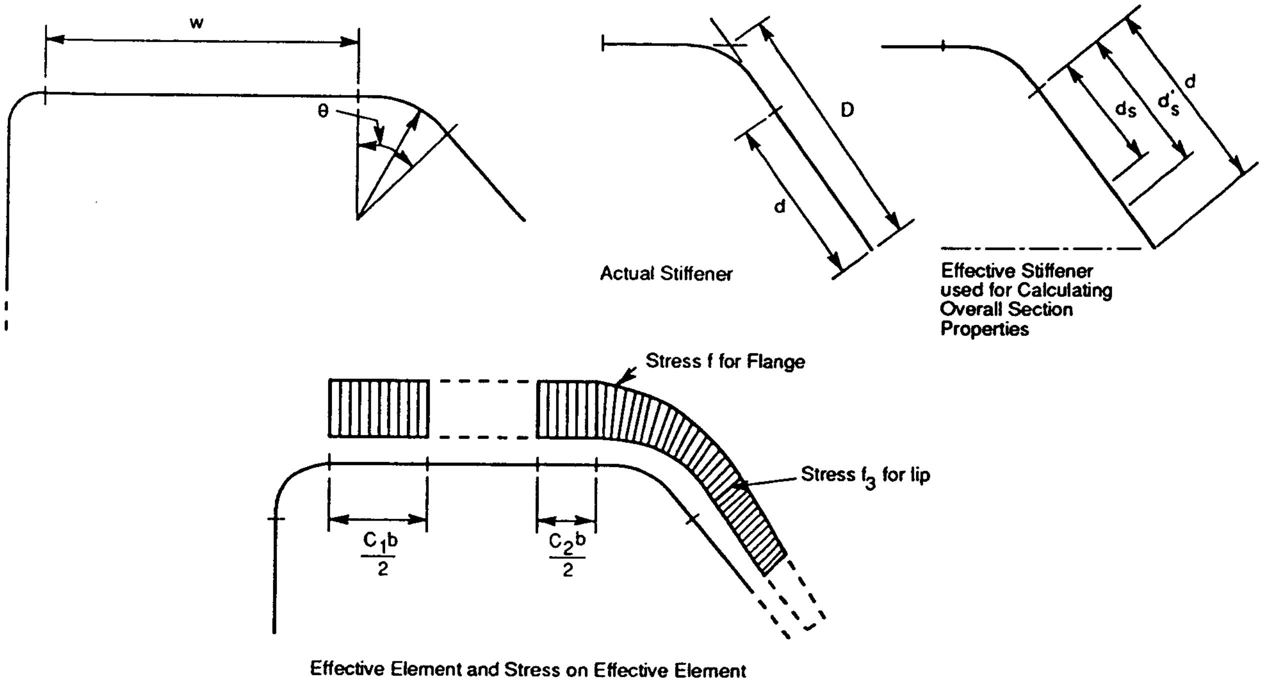


FIGURE 5. Elements with Edge Stiffener

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

$$b = w \quad (2.4.1\text{-}3)$$

$$A_s = A'_s \quad (2.4.1\text{-}4)$$

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

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

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: $$\frac{b_o}{t} \geq 3S$$

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

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: $$\frac{w}{t} \leq \frac{S}{3} \quad (2.4.2\text{-}1)$$

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

$$b = w \quad (2.4.2\text{-}3)$$

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

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

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

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

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

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

b shall be calculated according to Section 2.2.1 where

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

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:

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

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

$$A_{ef} = \alpha A_{st} \tag{2.5-4}$$

where:

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

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

$$A_{ef} = \frac{b_e}{w} A_{st} \tag{2.5-6}$$

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:

$$\phi_t = 0.85$$
$$T_n = A_n F_y \tag{3.2-1}$$

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:

$$M_n = S_e F_y \tag{3.3.1.1-1}$$

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:

$$\lambda_1 = \frac{1.11}{\sqrt{\dfrac{F_{yc}}{E_o}}} \qquad (3.3.1.1\text{-}2)$$

$$\lambda_2 = \frac{1.28}{\sqrt{\dfrac{F_{yc}}{E_o}}} \qquad (3.3.1.1\text{-}3)$$

(b) Unstiffened compression elements

$$C_y = 1$$

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

$$C_y = 1$$

When applicable, effective design widths shall be used in calculating section properties. M_n shall be calculated considering equilibrium of stresses, assuming an ideally elastic-plastic stress-strain curve 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:

$$\phi_d = 1.0 \qquad M_{ld} = S_f f_b \qquad (3.3.1.1\text{-}4)$$

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

$$f_b = 1.2\,F_{cr} \qquad (3.3.1.1\text{-}5)$$

For unstiffened compression elements

$$f_b = F_{cr} \qquad (3.3.1.1\text{-}6)$$

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

$$f_b = 0.9\,F_{cr} \qquad (3.3.1.1\text{-}7)$$

For unstiffened compression elements

$$f_b = 0.75\,F_{cr} \qquad (3.3.1.1\text{-}8)$$

where:

F_{cr} = critical buckling stress

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

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:

$$= \sqrt{r_x^2 + r_y^2 + x_o^2} \qquad (3.3.1.2\text{-}9)$$

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:

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

3.3.2 Strength for Shear Only

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

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

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

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

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

$$0.6\frac{M_u}{\phi_b M_n} + \frac{V_u}{\phi_v V_n} \le 1.3 \qquad (3.3.3\text{-}2)$$

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 $N/t \leq 210$, $N/h \leq 3.5$, and for beams with $R/t \leq 6$ and deck with $R/t \leq 7$.

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.

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

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

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

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

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

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

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

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 $\leq 1.5h$[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 $F_y/(91.5C_t) \leq 1.0$ (3.3.4-10)
$= 1.69$ when $F_y/(91.5C_t) > 1.0$

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

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

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

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

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

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

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

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

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 $\geq 45°$, 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:

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

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);

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

Exception: When $h/t \leq 2.33/\sqrt{(F_y/E_o)}$ and $\lambda \leq 0.673$, 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:

$$\phi_c = 0.85 \qquad P_n = A_e F_n \tag{3.4-1}$$

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:

$$\phi_d = 1.0 \qquad P_{ld} = A f_b \tag{3.4-2}$$

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:

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

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:

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

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:

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

* 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
   $C_m = 0.6 - 0.4(M_1/M_2)$ (3.5-6)

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:

$$s_{max} = \frac{L}{6} \qquad (4.1.1\text{-}2)$$

In no case shall spacing exceed value

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

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 mid-plane of its web;

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

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

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

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

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, $\alpha = 0$; 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

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

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 material 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 $\frac{L}{t} < 30$:

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

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

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

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)	1/4 Hard kips (kN)	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)	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:

$$\phi = 0.70 \qquad P_n = teF_u \tag{5.3.1-1}$$

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 ½ d)
(12.7)	(d + .794)	(d + 1.588)	(d + .794) (d + 6.35)	(d + .793)
≥ 1/2	d + 1/16	d + 1/8	(d + 1/16) by (d + 1/4)	(d + 1/16) by (2 ½ 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:

$$\phi = 0.70 \qquad P_n = A_n F_t \tag{5.3.2-1}$$

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

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:

$$\phi = 0.85 \qquad P_n = F_y A_n \tag{5.3.2-4}$$

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:

$$\phi = 0.65 \qquad P_n = F_p dt \tag{5.3.3-1}$$

where:

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

1. For single shear connection:
$$F_p = 2.00\,F_u \tag{5.3.3-2}$$

2. For double shear connection:

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

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

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

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:

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

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]	$\leq$ 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]	$\leq$ 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]	$1/4 \leq d \leq 1\cdot1/2$ $(6.4 \leq d \leq 38.1)$	42.0 (289.6)	31.5 (217.2)	52.5 (361.9)
304,316[e]	$1/4 \leq d \leq 1\cdot1/2$ $(6.4 \leq d \leq 38.1)$	42.0 (289.6)	31.5 (217.2)	52.5 (362.0)
304,316[f]	$1/4 \leq d \leq 5/8$ $(6.4 \leq d \leq 15.9)$	57.0 (393.0)	42.8 (295.1)	71.2 (490.9)
	$3/4 \leq d \leq 1\cdot1/2$ $(19.1 \leq d \leq 38.1)$	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 $d \geq 1/2$ 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.

APPENDICES

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.

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

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

	F_y, ksi (MPa)									
	Types 201, 301, 304, 316						S20400			
Type of Stress	Annealed		1/16 Hard		1/4 Hard	1/2 Hard	Annealed	1/4 Hard	Type 409	Types 430, 439
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 × 10^3 (MPa × 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 × 10^3 (MPa × 10^3)	27.0 (186.2)	29.0 (200)
Initial shear modulus: G_o, ksi × 10^3 (MPa × 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		Transverse compression	
	Annealed	1/4 Hard	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		Types 430, 439	
	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)	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)

	E_s/E_o			
	Longitudinal compression		Transverse compression	
Stress ksi (MPa)	Annealed	1/4 Hard	Annealed	1/4 Hard
0	1.00	1.00	1.00	1.00
4 (27.6)	1.00	0.99	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)

	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 (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 ksi (MPa)	E_t/E_o			
	Longitudinal compression		Transverse compression	
	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)

	E_t/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)	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

	G_s/G_o				
	Types 201, 301, 304, 316				
Stress ksi (MPa)	Annealed and 1/16 Hard	1/4 Hard	1/2 Hard	Type 409	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 × 10^3 (MPa × 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 × 10^3 (MPa × 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

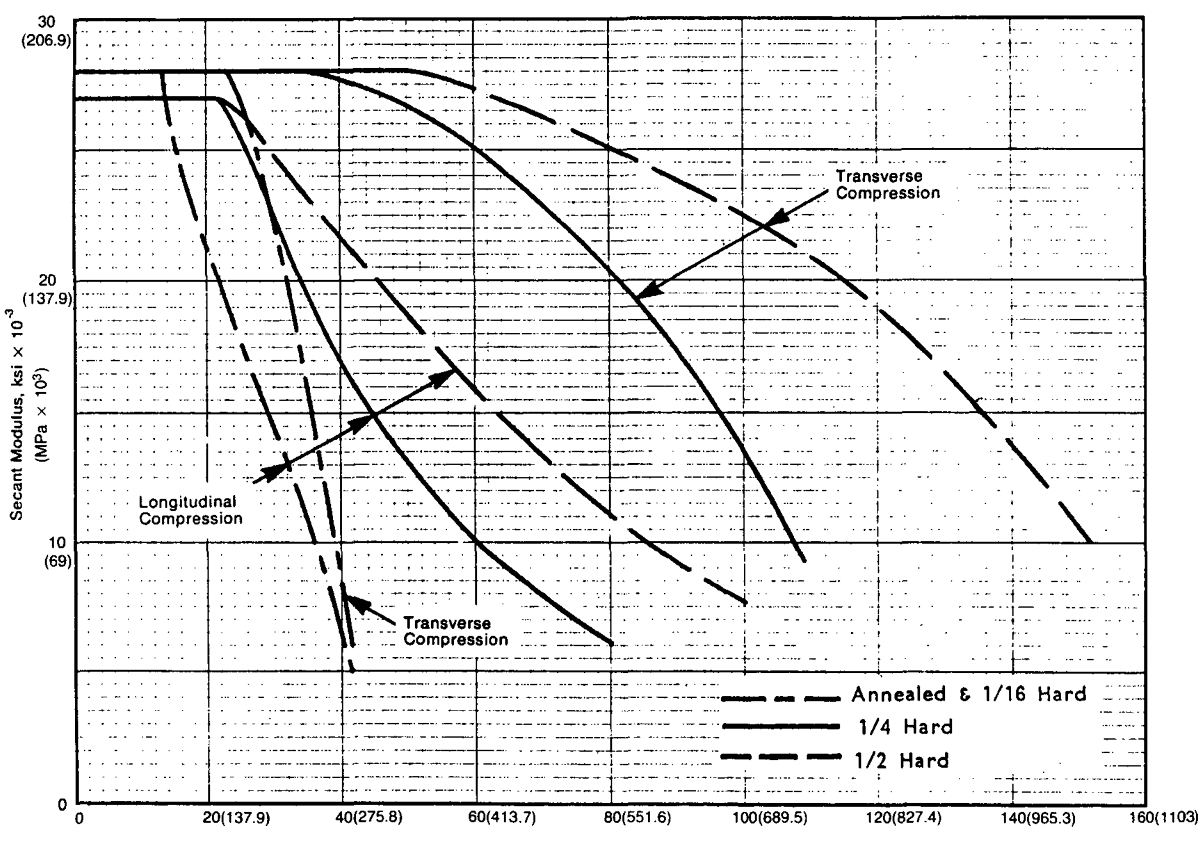


FIGURE A1a. Secant Moduli for Deflection Calculations (Types 201, 301, 304, and 316)

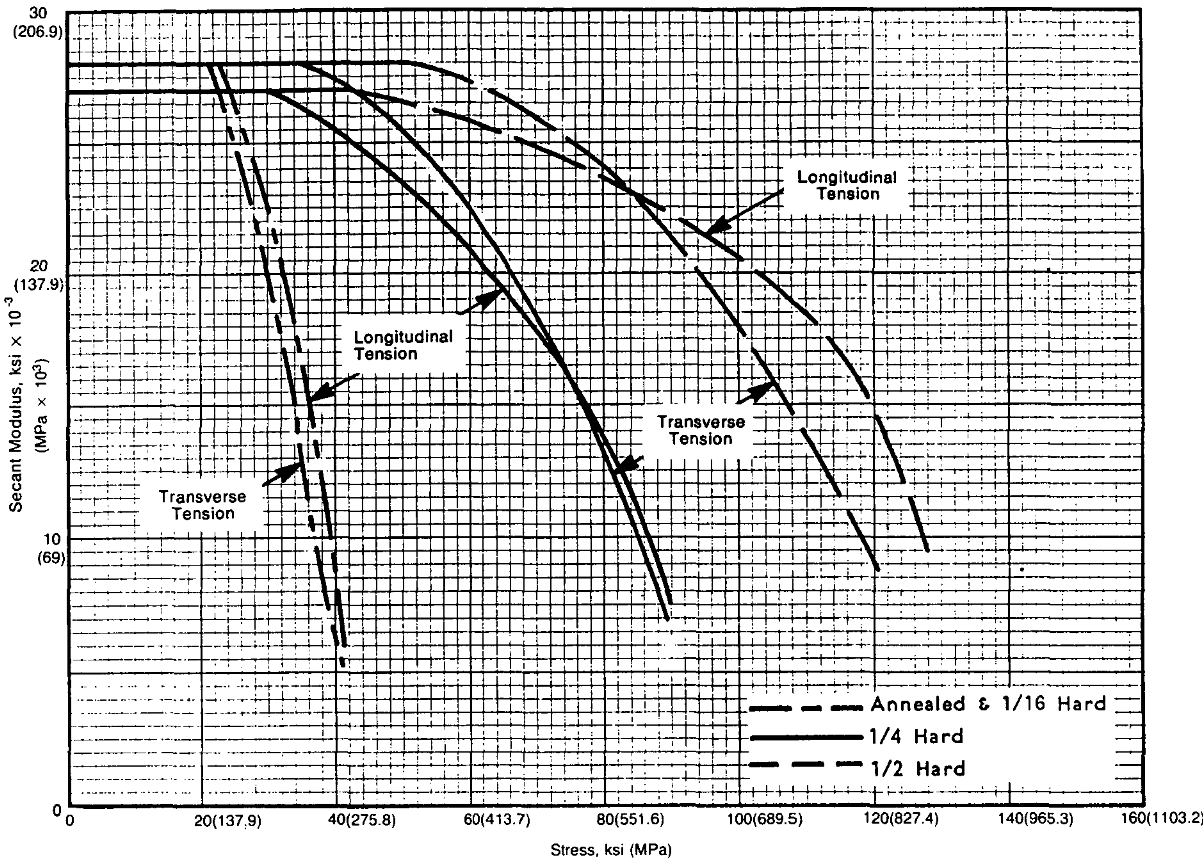


FIGURE A1a. *Continued*

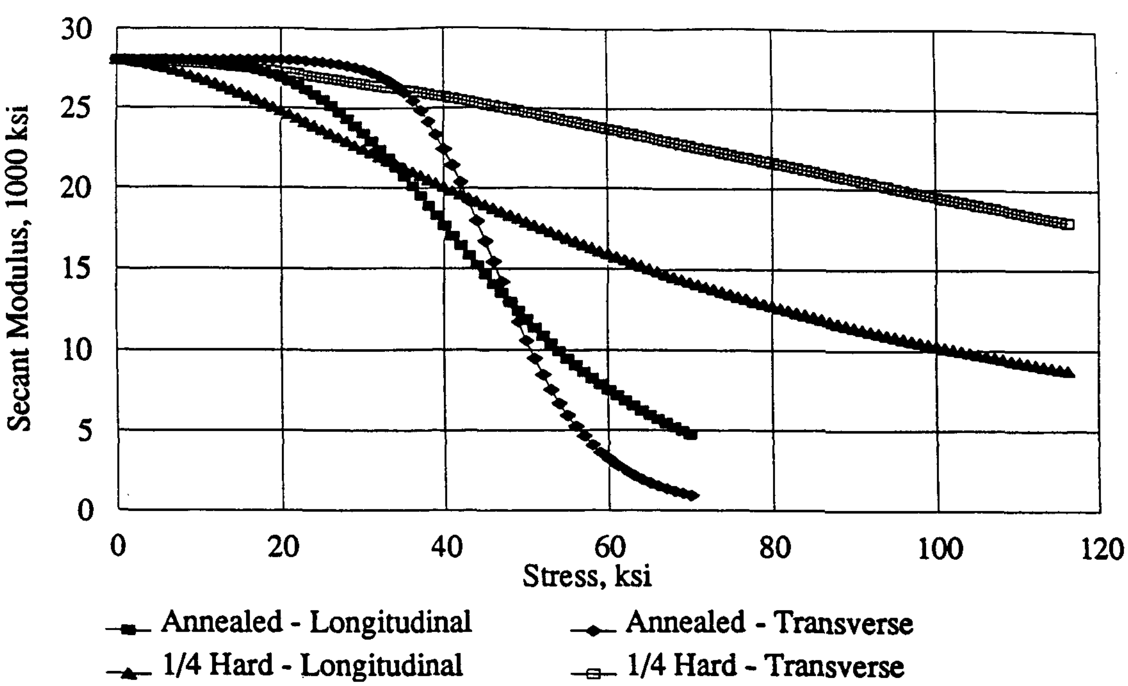


FIGURE A1b. Secant Moduli for Deflection Calculations (S20400, Compression)

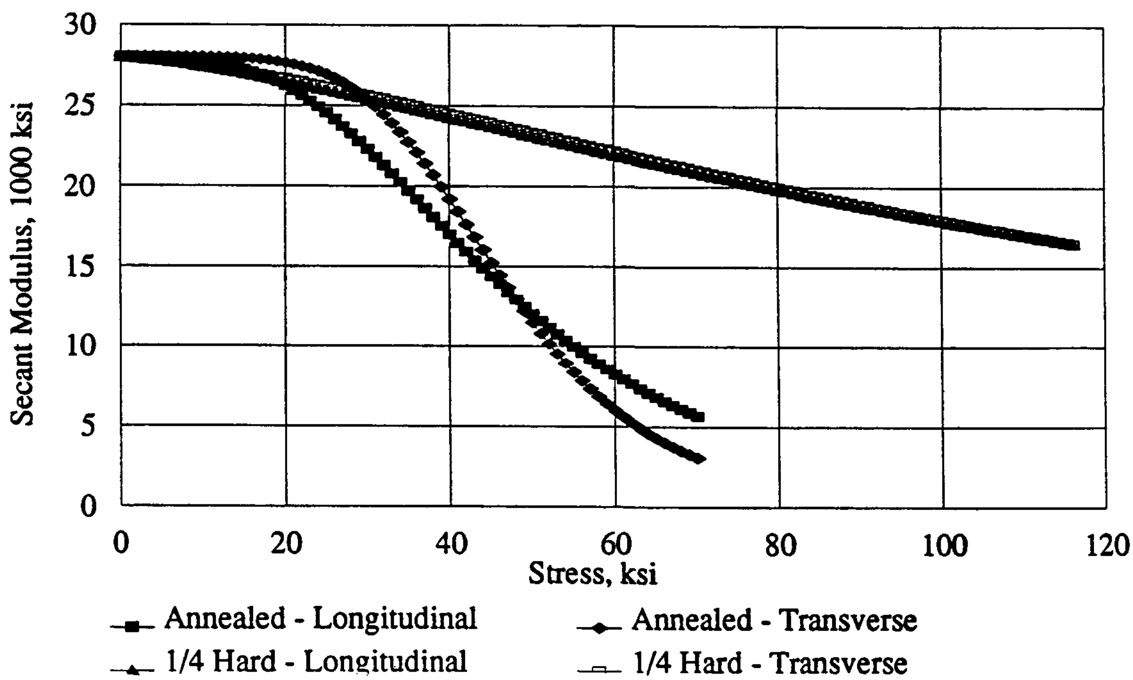


FIGURE A1b. ***Continued*** **(S20400, Tension)**

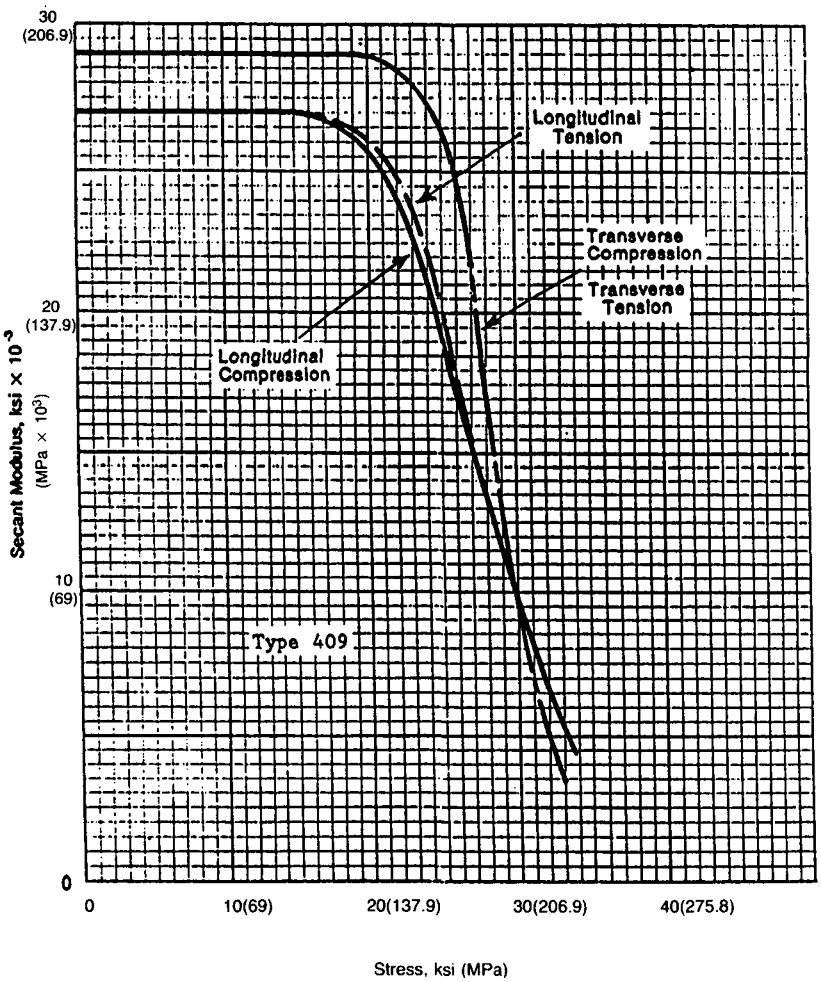


FIGURE A2. Secant Moduli for Deflection Calculations (Types 409, 430, and 439)

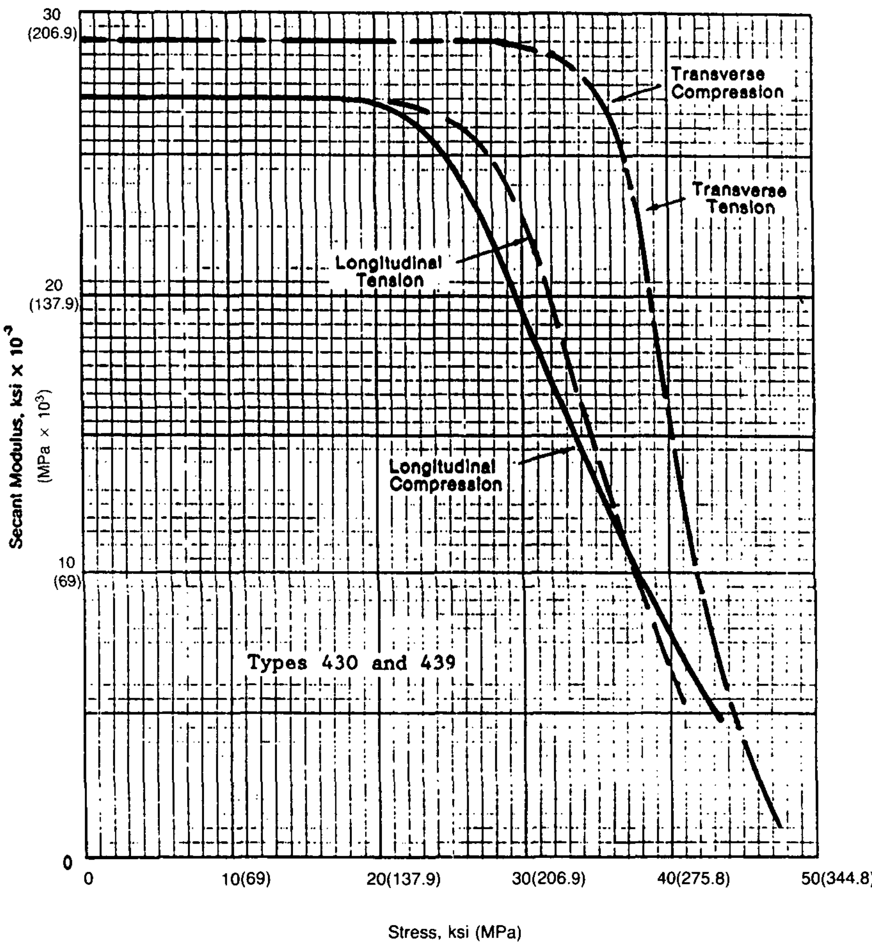


FIGURE A2. ***Continued***

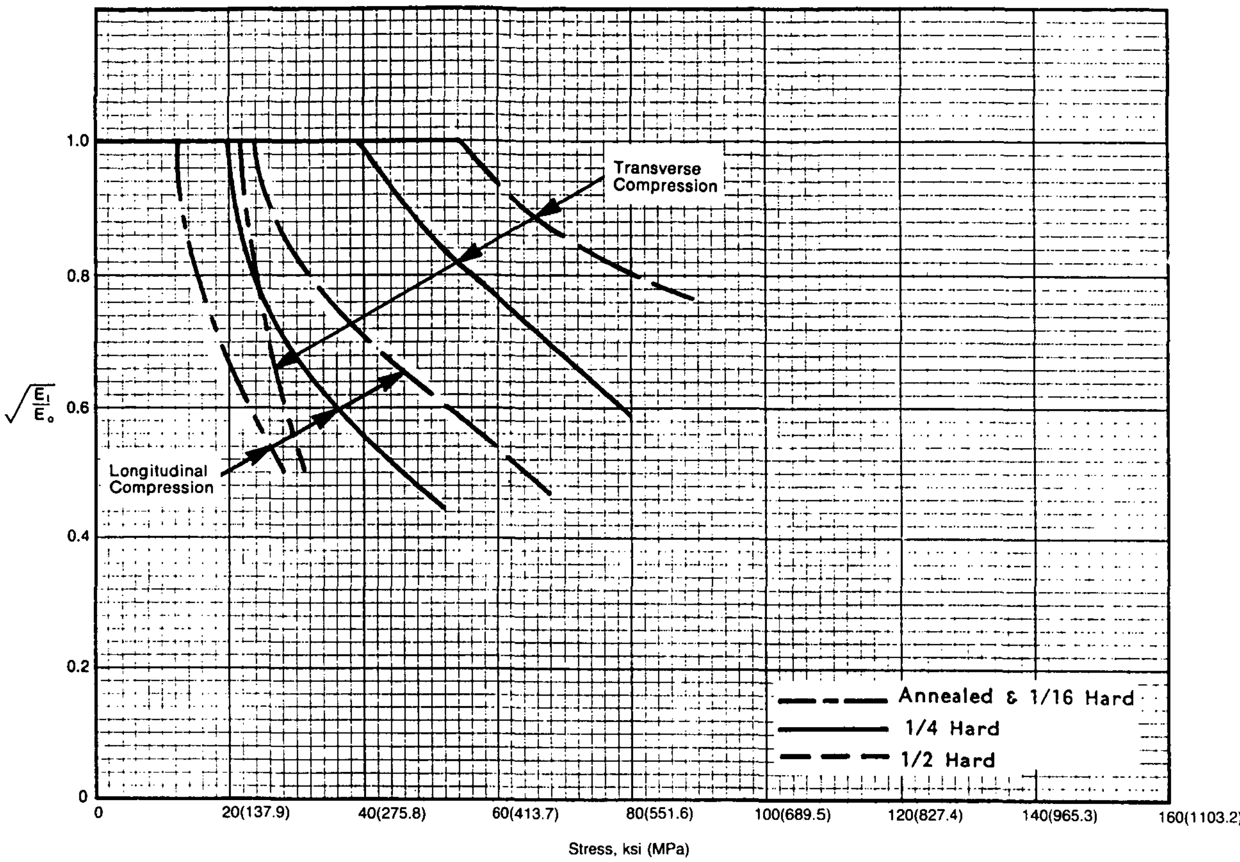


FIGURE A3a. Plasticity Reduction Factors for Stiffened Compression Elements (Types 201, 301, 304, and 316)

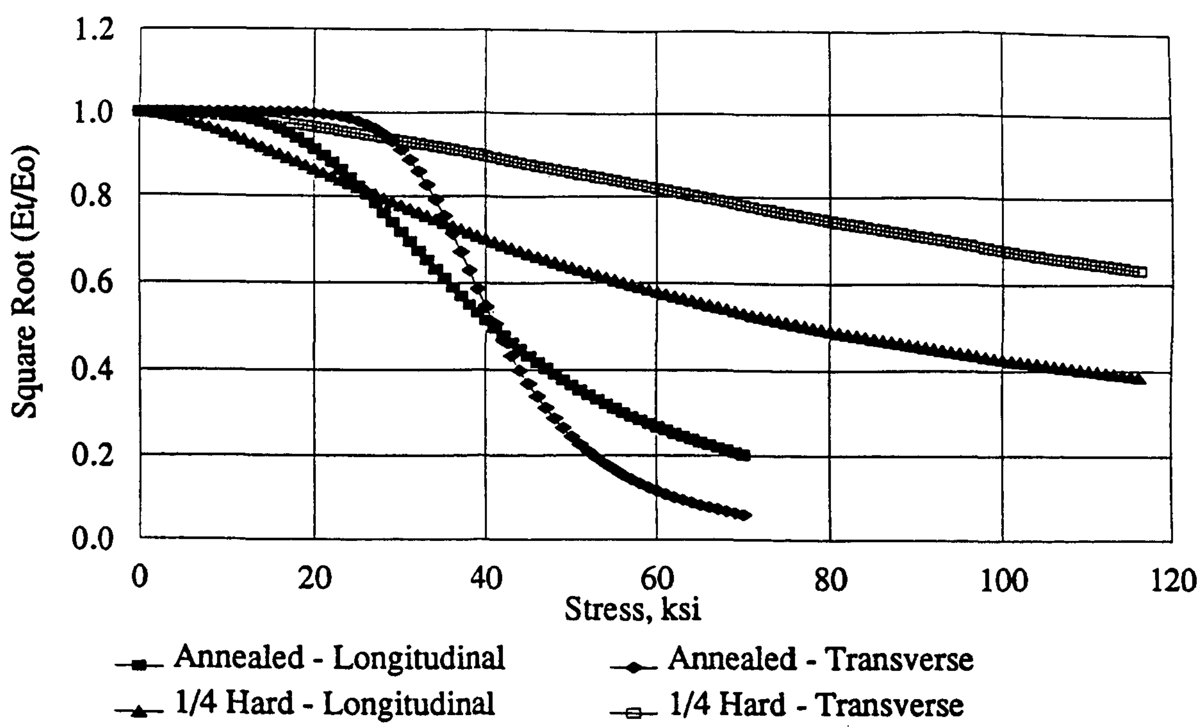


FIGURE A3b. Plasticity Reduction Factors for Stiffened Compression Elements (S20400)

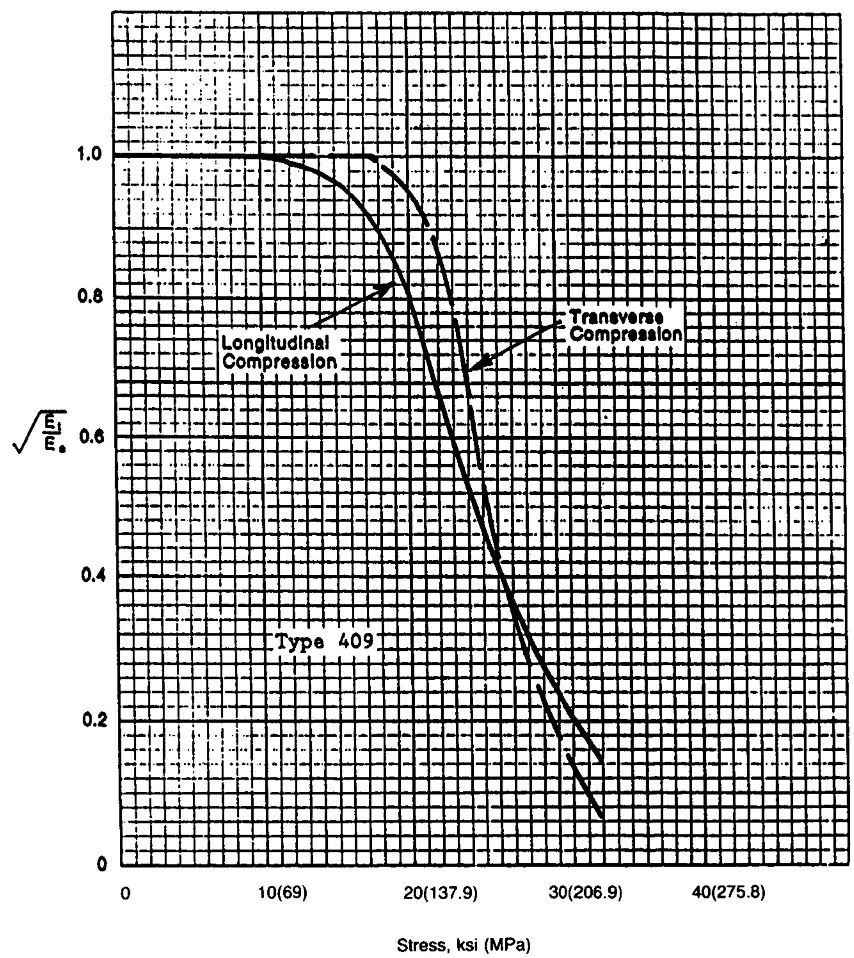


FIGURE A4. Plasticity Reduction Factors for Stiffened Compression Elements (Types 409, 430, and 439)

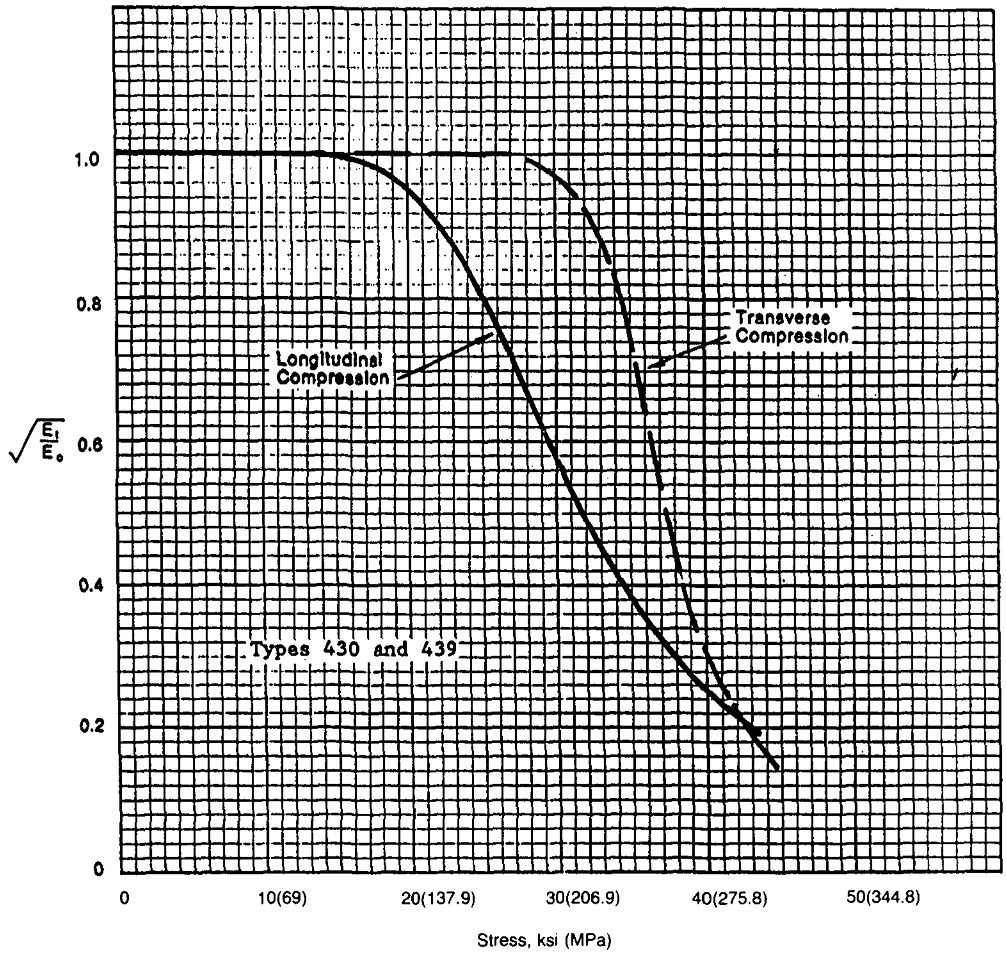


FIGURE A4. *Continued*

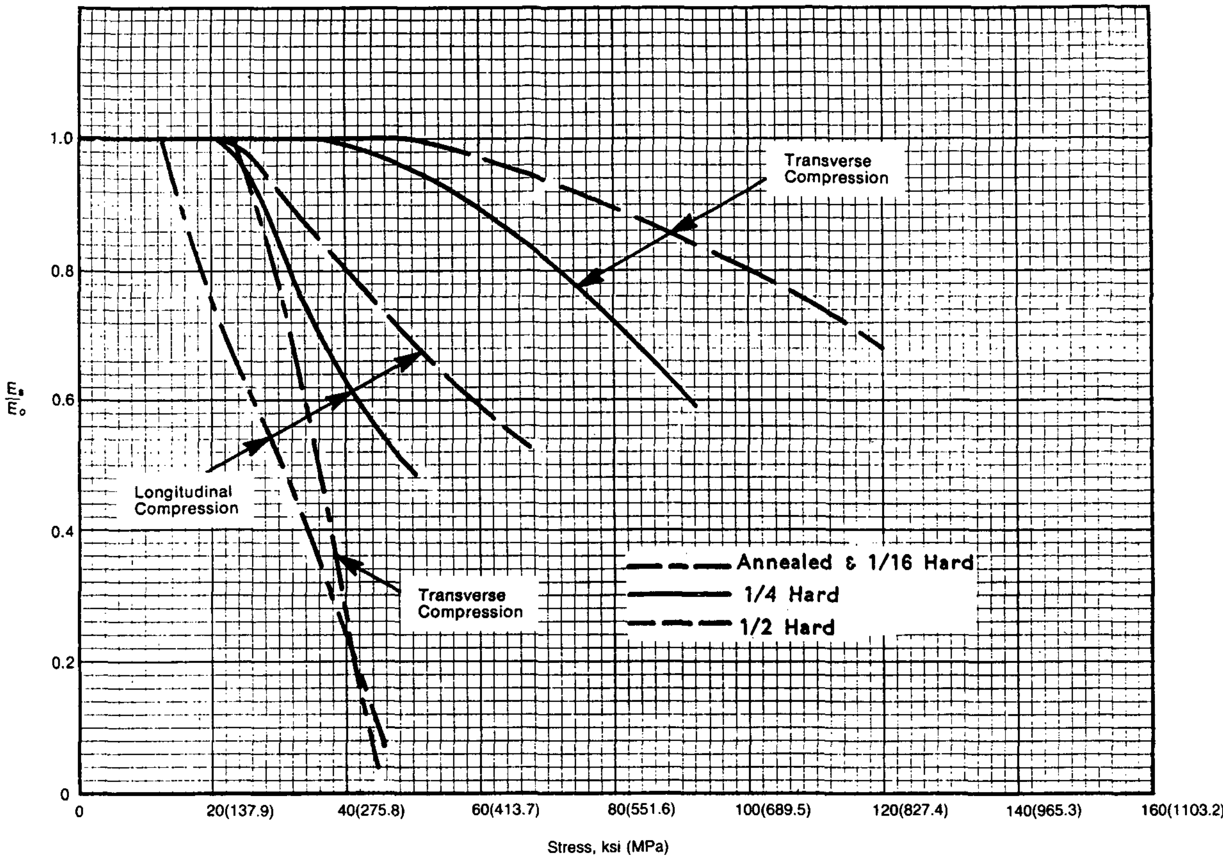


FIGURE A5a. Plasticity Reduction Factors for Unstiffened Compression Elements (Types 201, 301, 304, and 316)

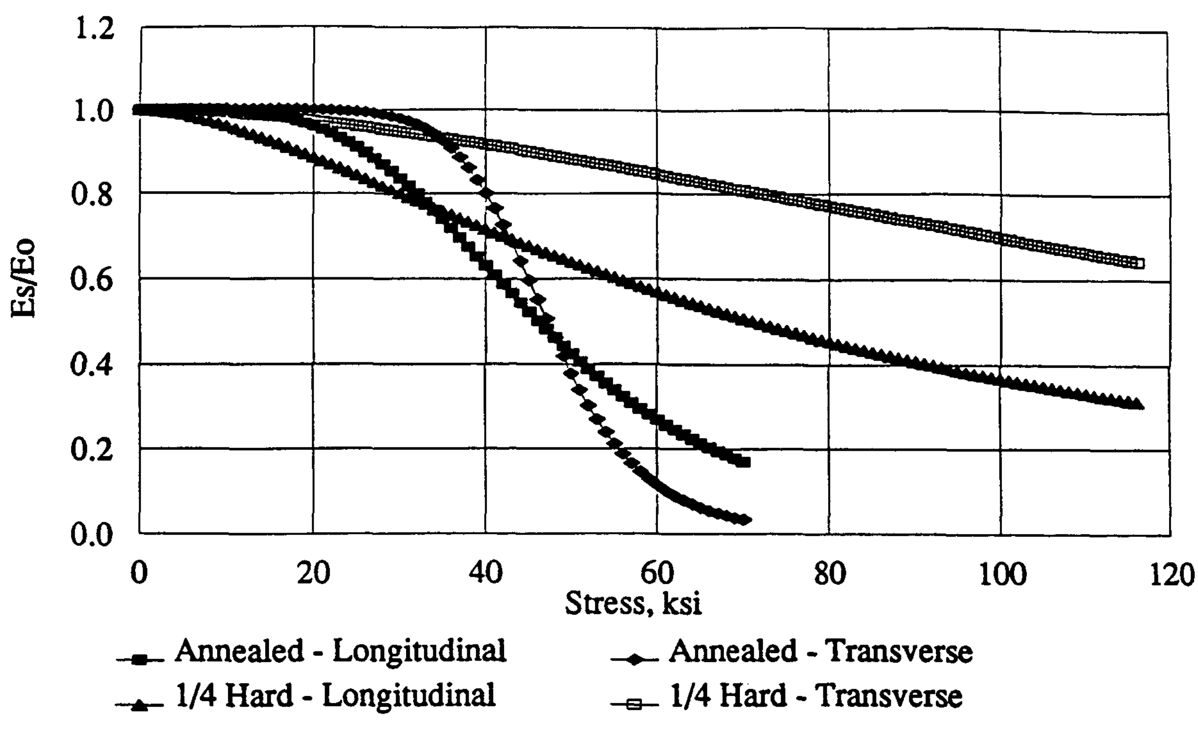


FIGURE A5b. Plasticity Reduction Factors for Unstiffened Compression Elements (S20400)

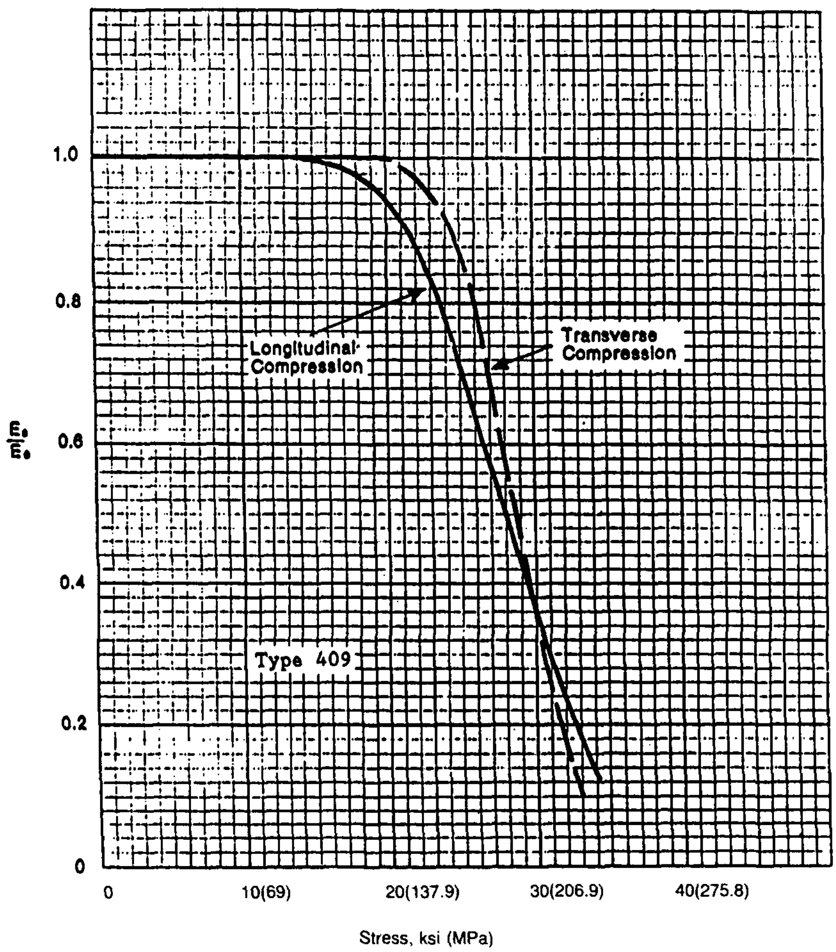


FIGURE A6. Plasticity Reduction Factors for Unstiffened Compression Elements (Types 409, 430, and 439)

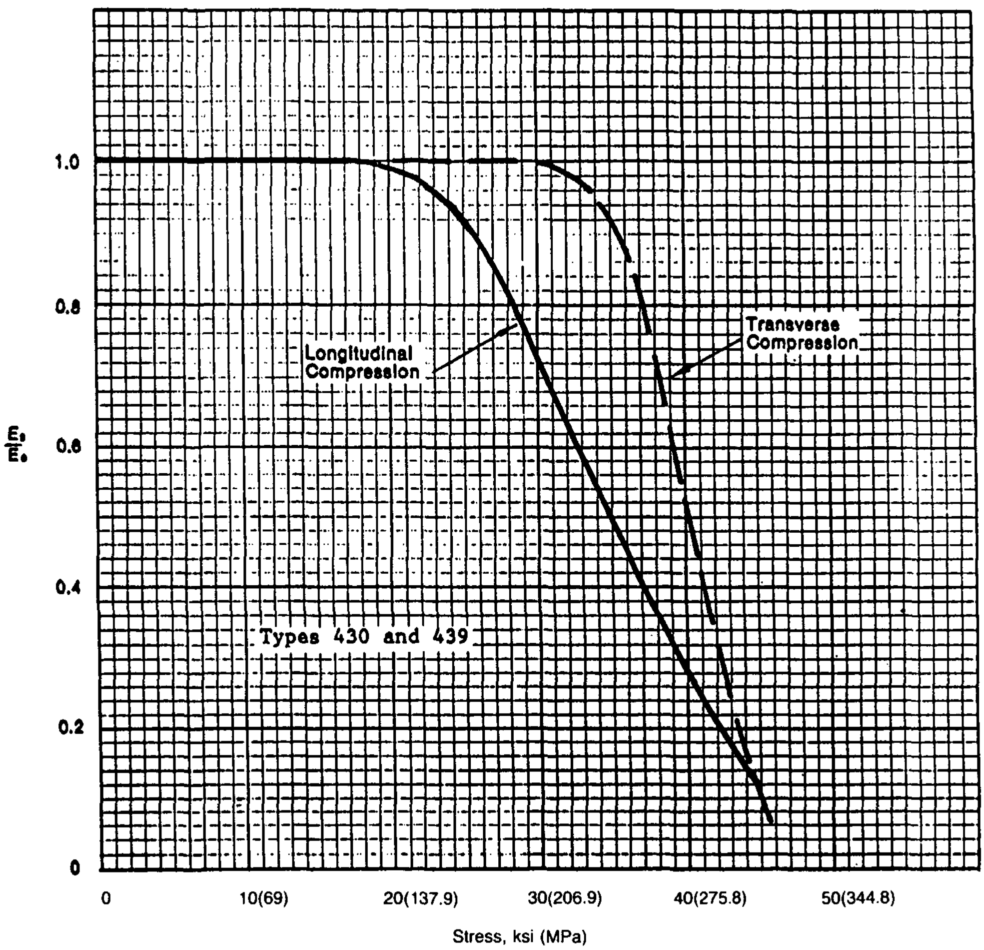


FIGURE A6. ***Continued***

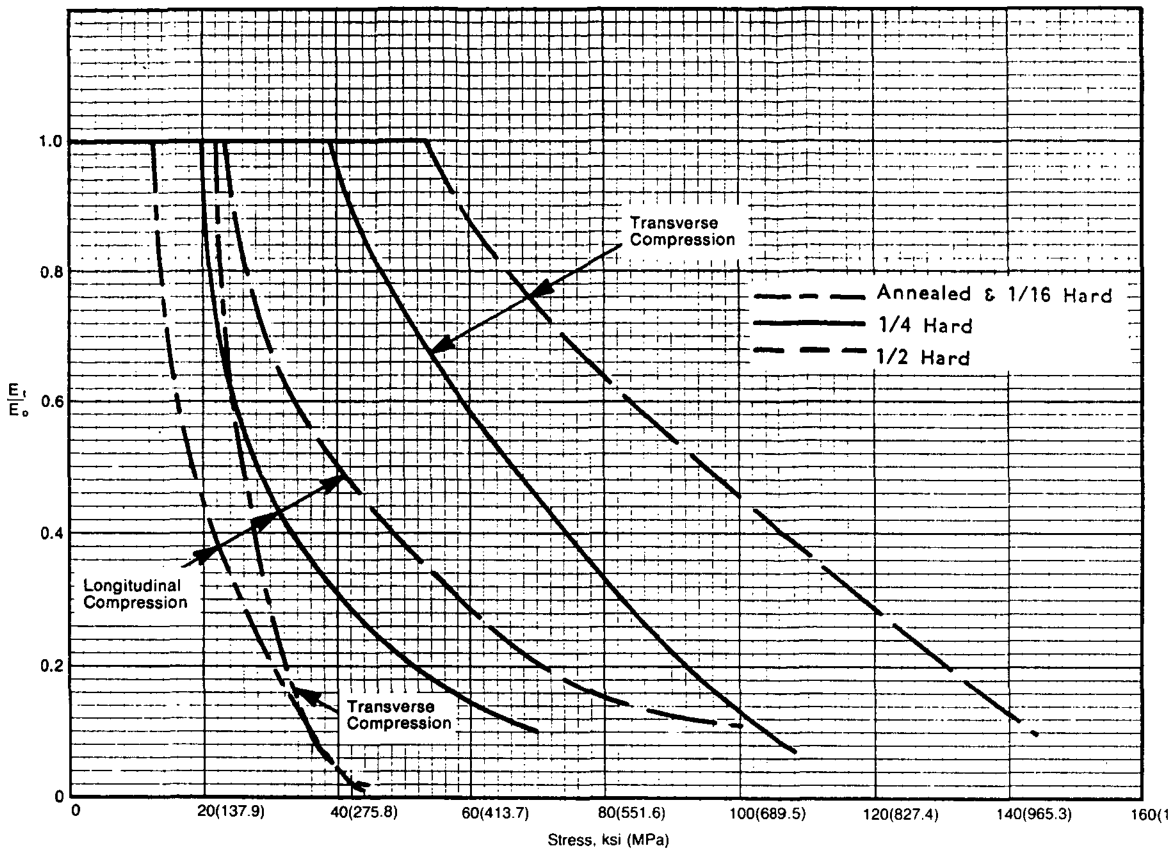


FIGURE A7a. Plasticity Reduction Factors For Design of Laterally Unbraced Single Web Beams (Types 201, 301, 304, and 316)

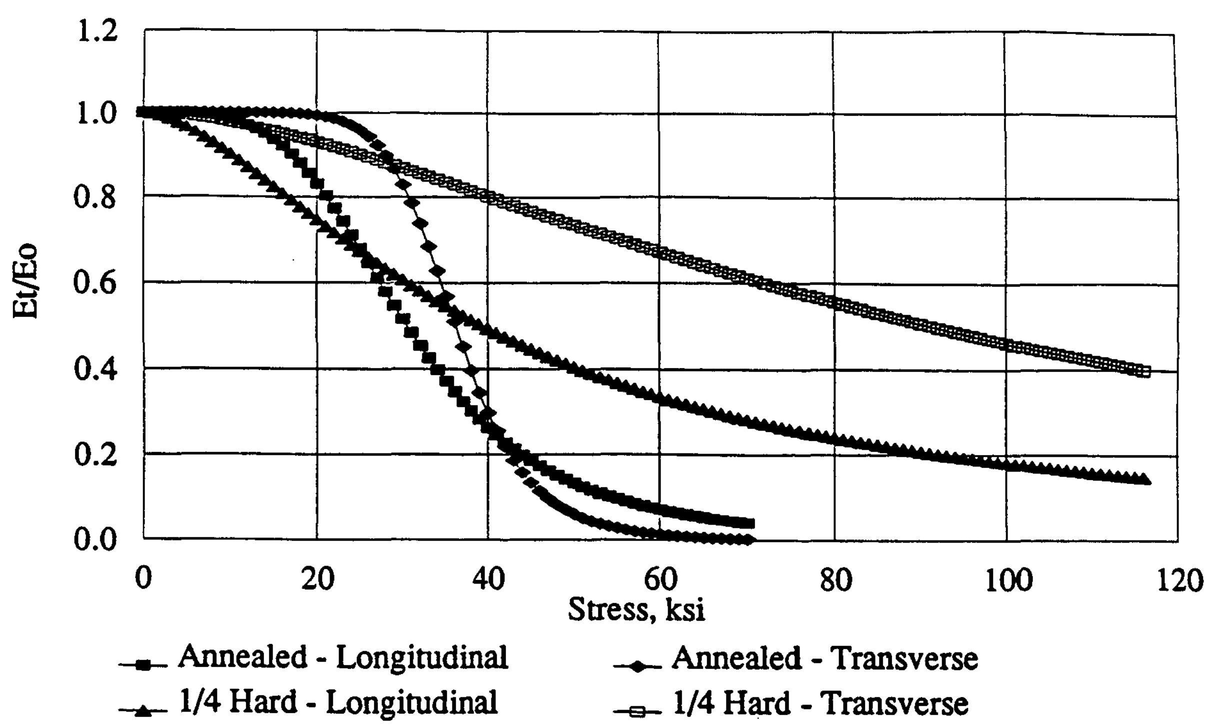


FIGURE A7b. Plasticity Reduction Factors For Design of Laterally Unbraced Single Web Beams (S20400)

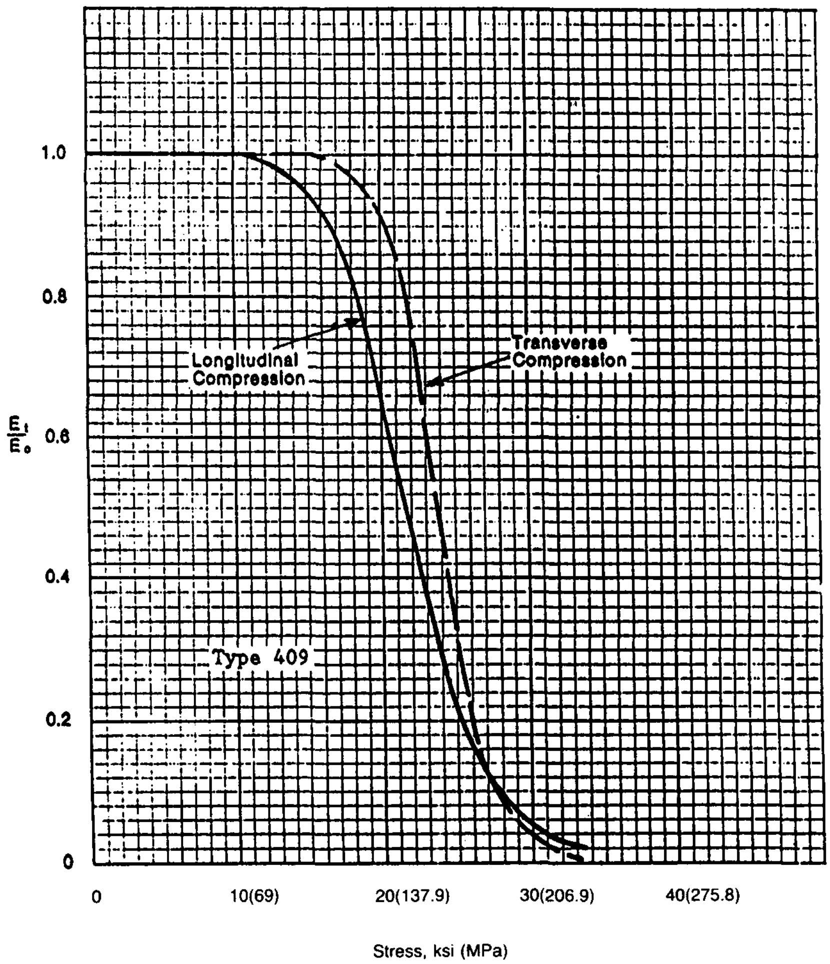


FIGURE A8. Plasticity Reduction Factors for Design of Laterally Unbraced Single Web Beams (Types 409, 430, and 439)

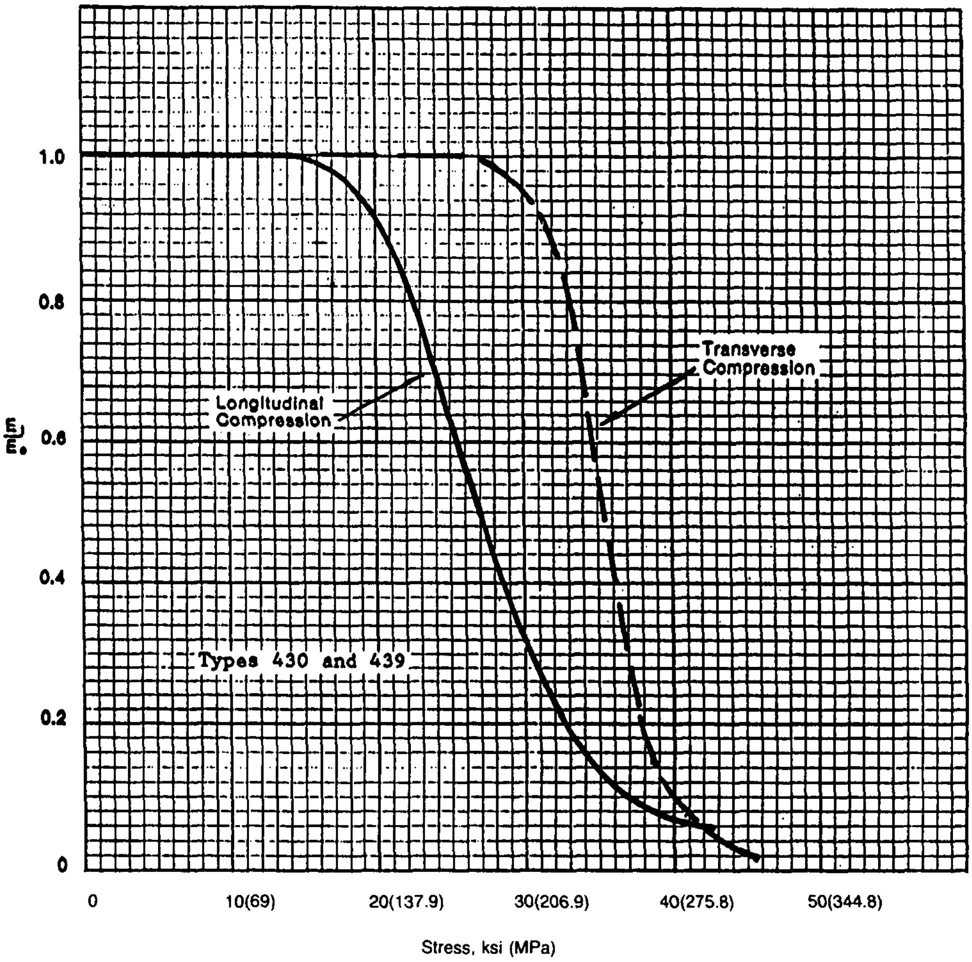


FIGURE A8. ***Continued***

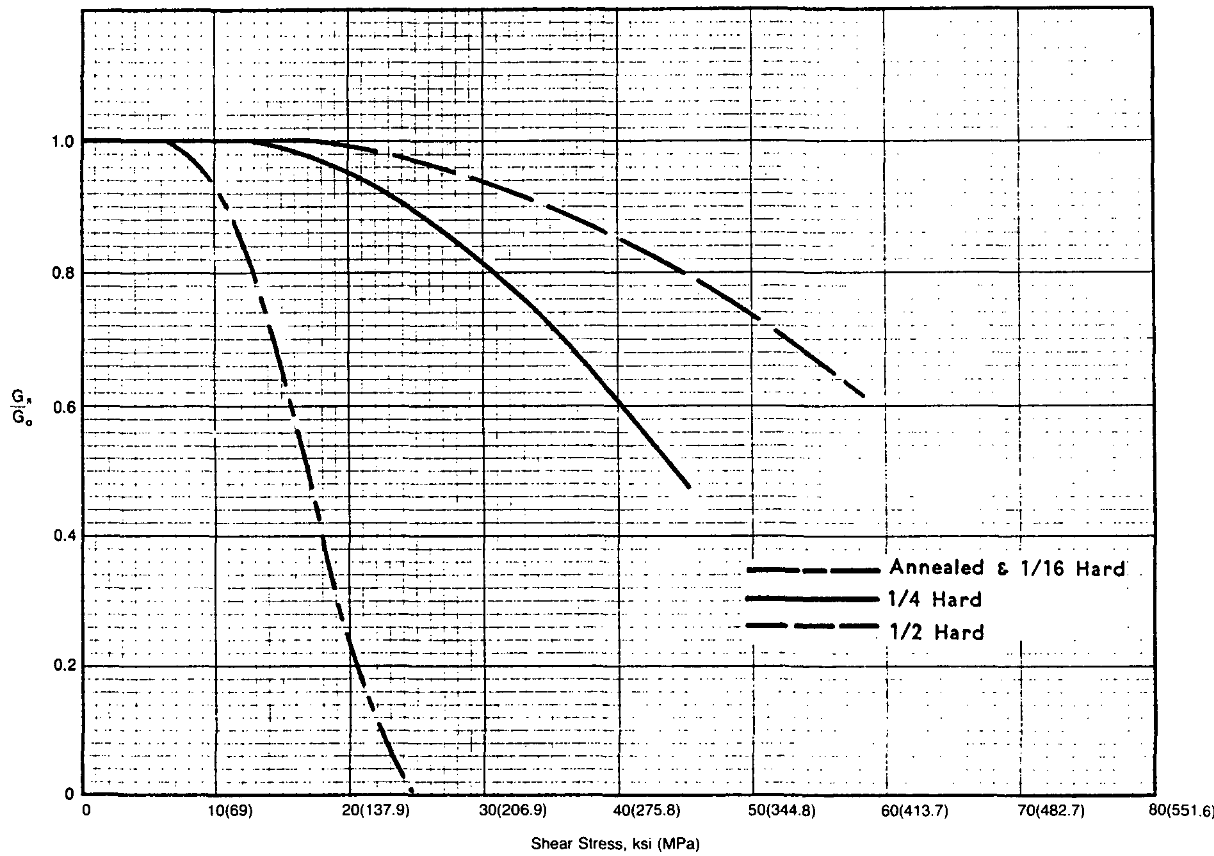


FIGURE A9. Plasticity Reduction Factors for Shear Stresses in Webs (Types 201, 301, 304, and 316)

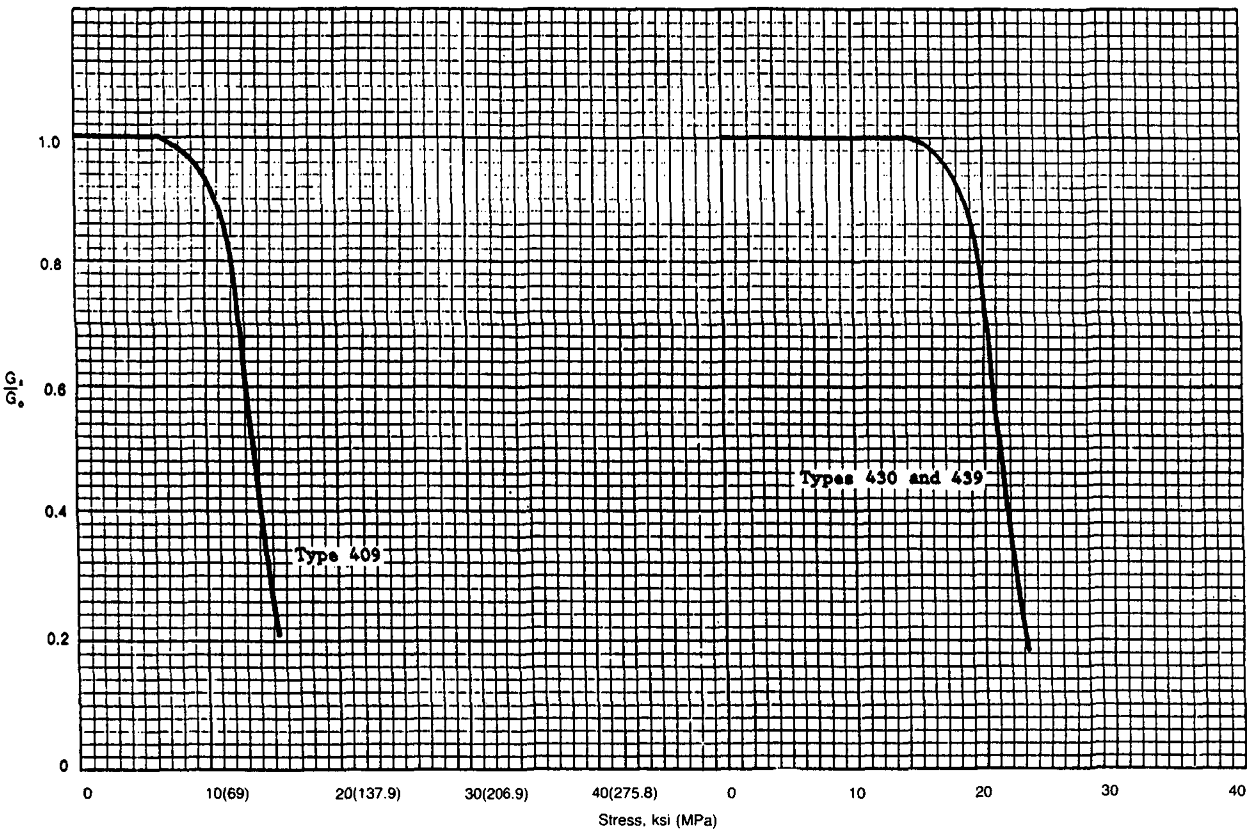


FIGURE A10. Plasticity Reduction Factors for Shear Stresses in Webs (Types 409, 430, and 439)

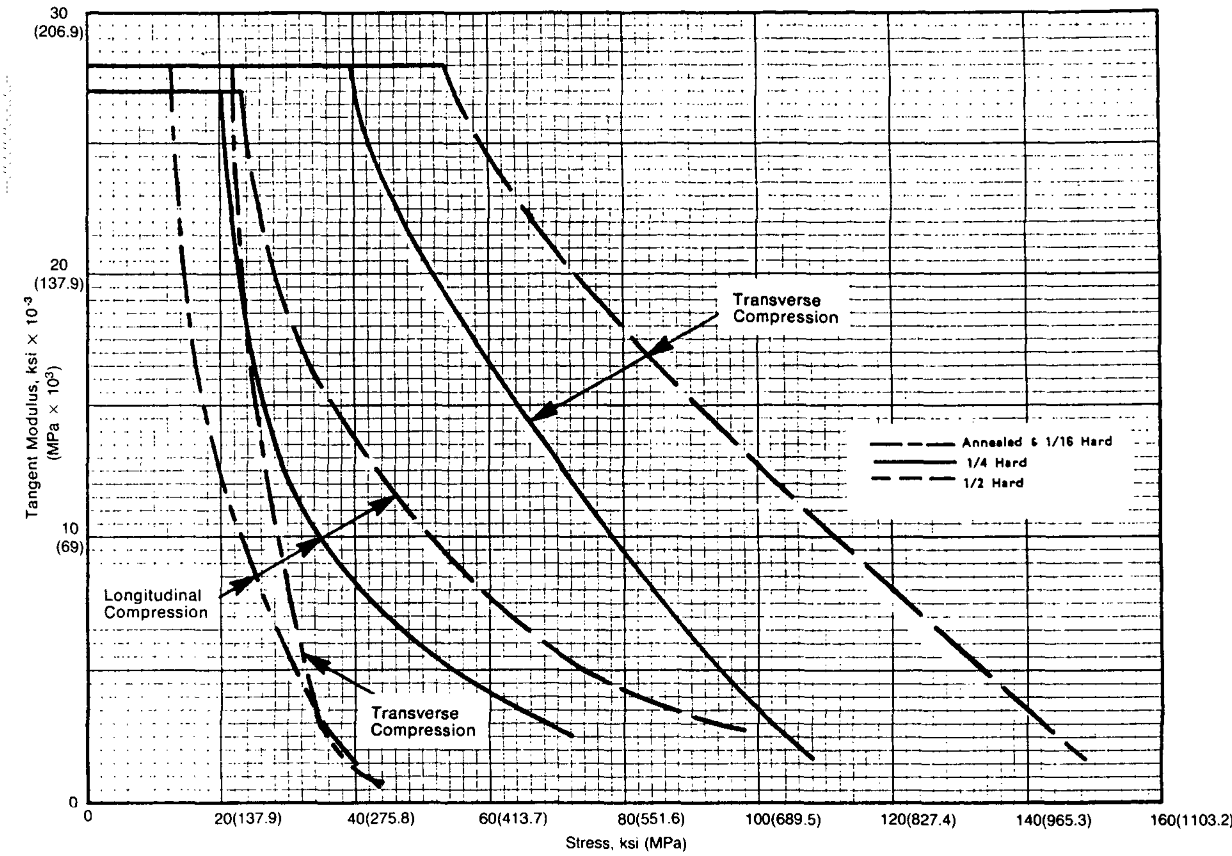


FIGURE A11a. Tangent Moduli for Design of Columns (Types 201, 301, 304, and 316)

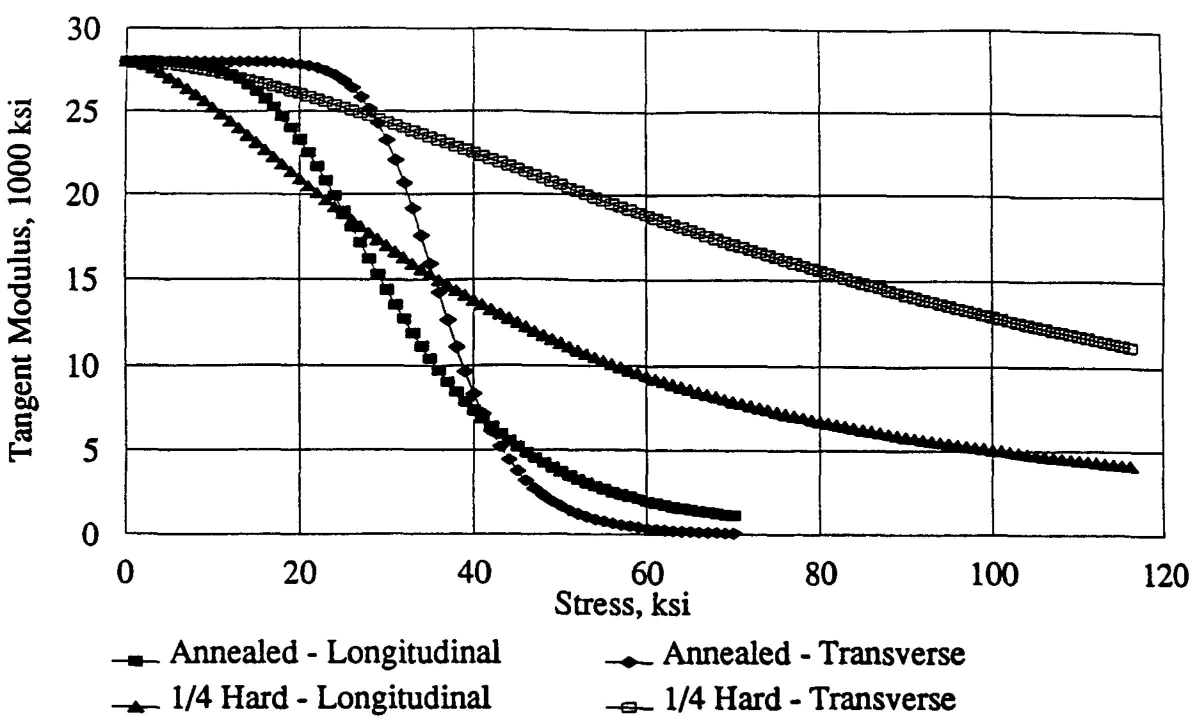


FIGURE A11b. Tangent Moduli for Design of Columns (Type S20400)

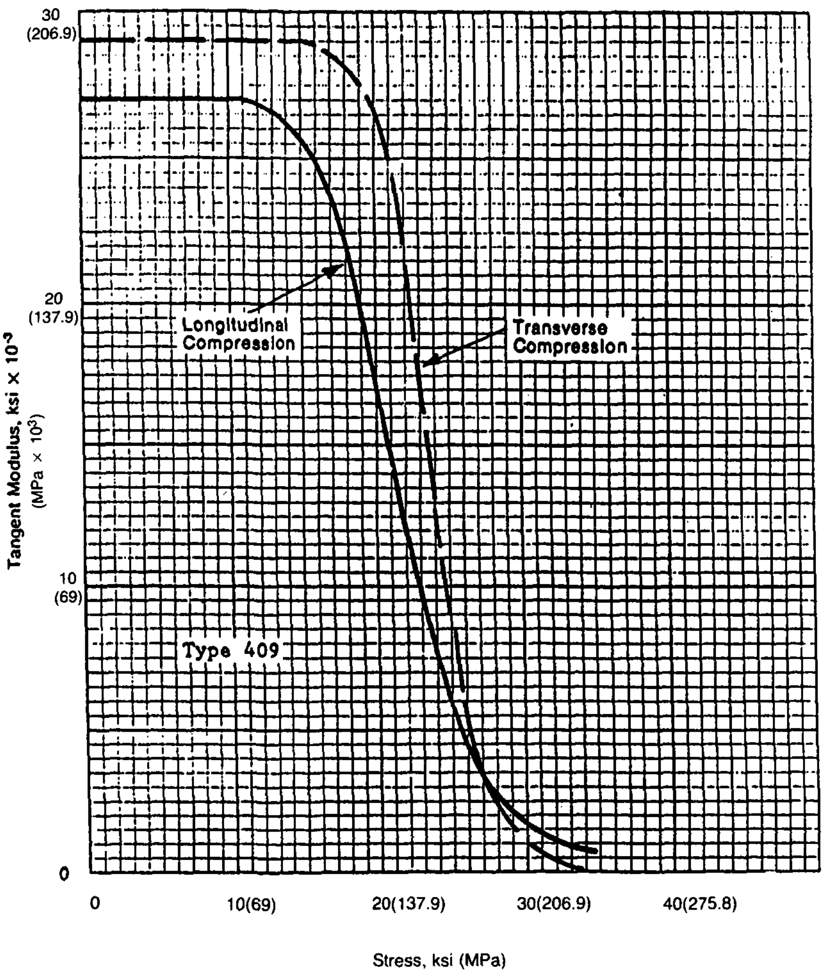


FIGURE A12. Tangent Moduli for Design of Columns (Types 409, 430, and 439)

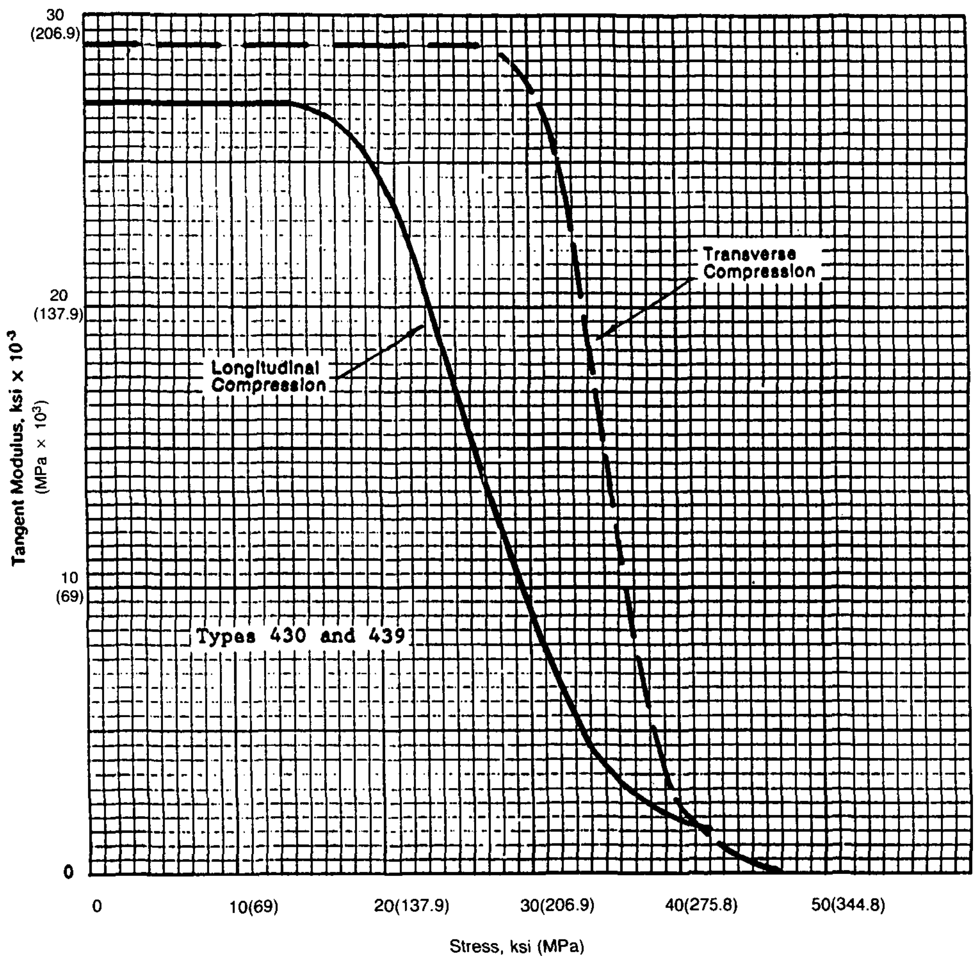


FIGURE A12. *Continued*

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

Types of Stress	Types 201, 301, 304, 316			UNS S20400			
	Annealed and 1/16 Hard	1/4 Hard	1/2 Hard	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}
$A_b = b_1t + A_s$, for transverse stiffeners at interior support and under concentrated load (C-4)
$A_b = b_2t + A_s$, 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

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

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:

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

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

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

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

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

COMMENTARY

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

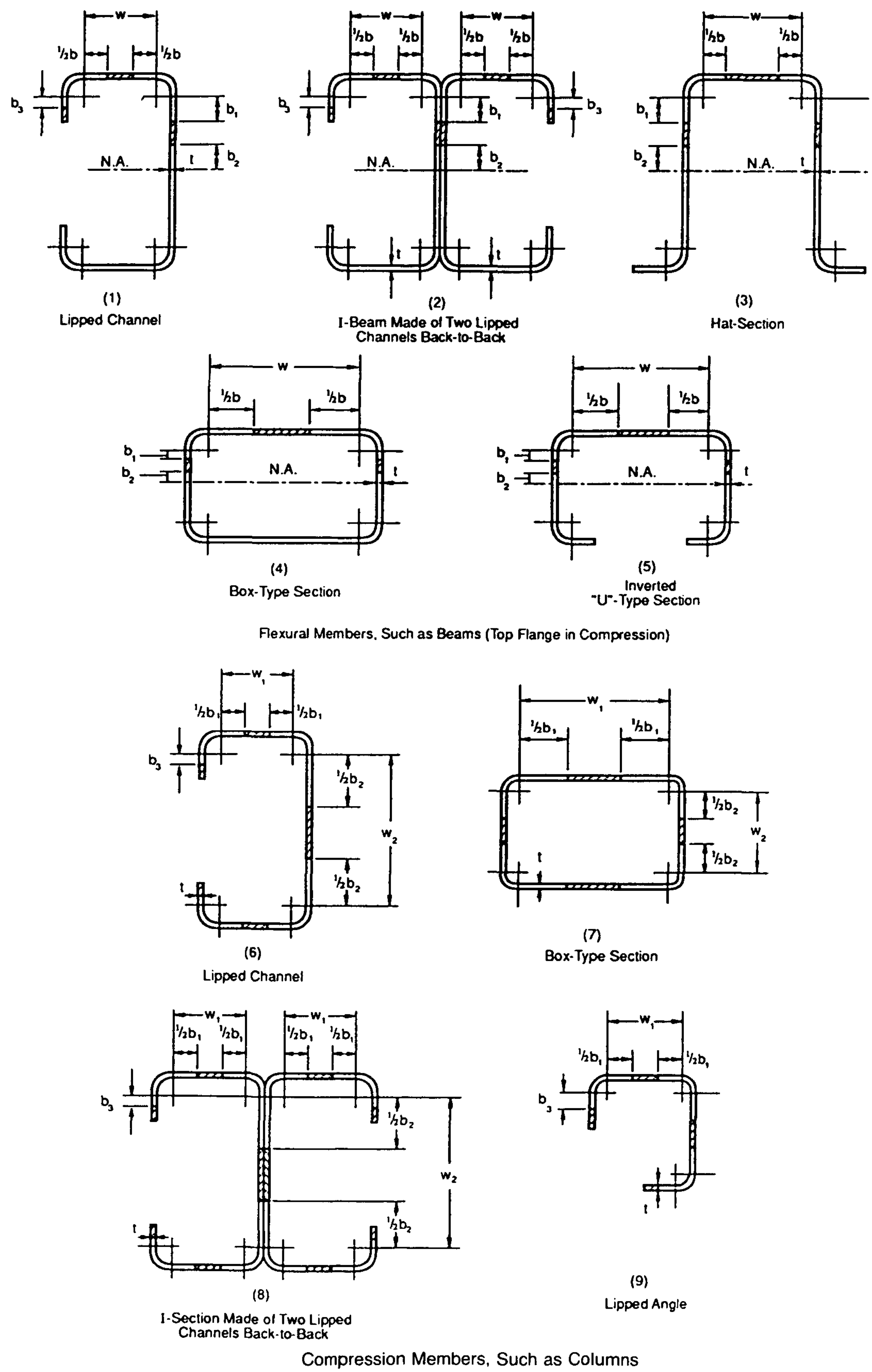


FIGURE C1. Members with Stiffened and Partially Stiffened Compression Elements

webs. Shapes 6 and 8 show edge-stiffened flange elements that have a vertical element (web) and an edge stiffener (lip) to stiffen the elements while the web itself is stiffened by the flanges. Shape 7 has four compression elements stiffening each other, and Shape 9 has two stiffened leg elements stiffened by a lip and the other stiffened element.

b. *Unstiffened Compression Elements.* "Unstiffened compression element" is a flat compression element supported along only one edge parallel to the direction of stress. Figure C2 shows various shapes used as flexural members and compression members which contain unstiffened compression elements. Shapes 1, 2, and 3 have only a web to support the compression flange element. The legs of Shape 4 provide mutual stiffening action to each other along their common edge. Shapes 5, 6, and 7 used as columns have vertical stiffened elements (webs) which provide support for one edge of the unstiffened flange elements. The legs of Shape 8 provide mutual stiffening action to each other.

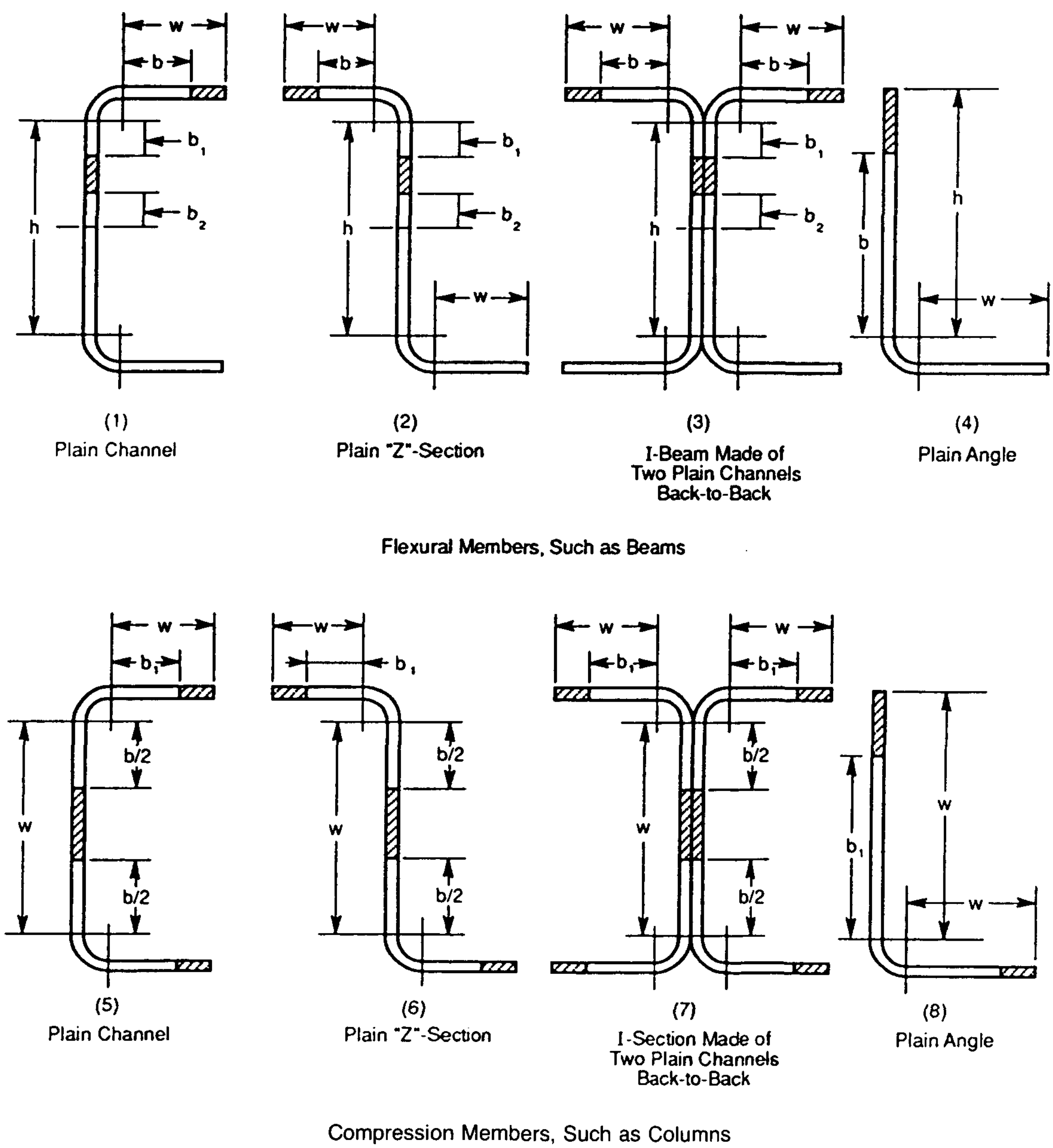
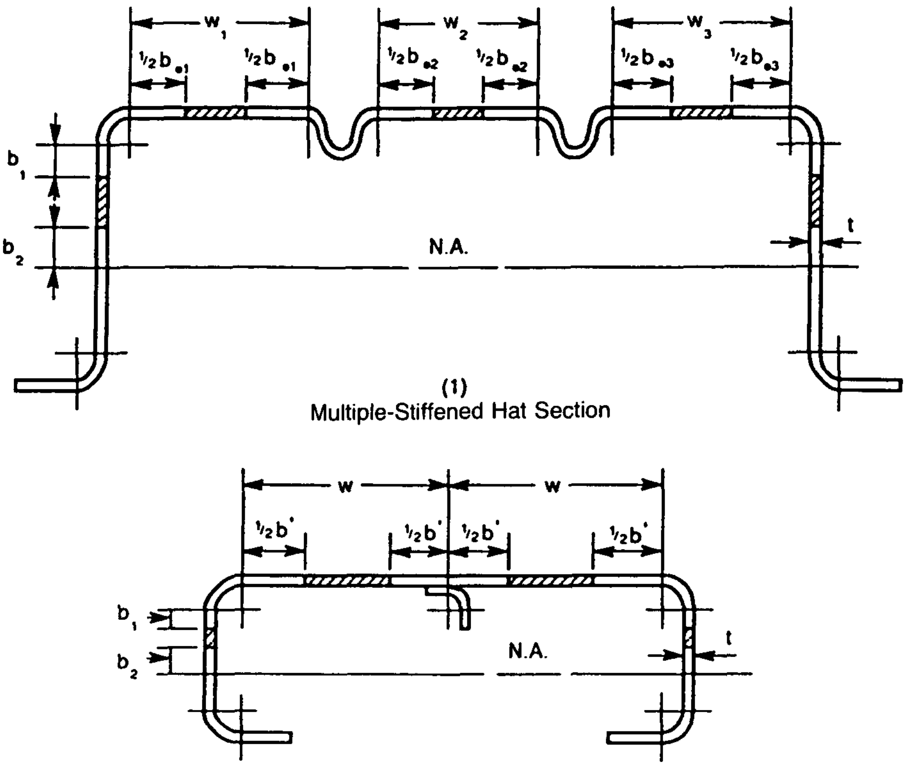


FIGURE C2. Members with Unstiffened Compression Elements



(1)
Multiple-Stiffened Hat Section

(2)
Multiple-Stiffened Inverted "U"-Type Section

Flexural Members, Such as Beams

FIGURE C3. Members with Multiple-Stiffened Compression Elements

c. *Multiple-Stiffened Elements.* Multiple-stiffened elements of two shapes are shown in Figure C3. Each of the two outer sub-elements of Shape 1 are stiffened by a web and an intermediate stiffener while the middle sub-element is stiffened by two intermediate stiffeners. The two sub-elements of Shape 2 are stiffened by a web and the attached intermediate middle stiffener.

d. *Effective Design Width.* The effective design width is a concept taking account of local buckling and post-buckling strength for compression elements. In Figures C1, C2 and C3, "w," "w_1," "w_2" and "w_3" represent the flat widths and "b," "b_1," "b_2," "b_3," and "b'" represent the effective design widths. The effect of shear lag on short, wide flanges is also handled by using an effective design width. These matters are treated in Section 2.1.1 of the Specification, and the corresponding effective widths are discussed in the Commentary on that Section.

1.1.3 Units of Symbols and Terms

The non-dimensional character of the majority of the design provisions in this Specification is intended to facilitate the design in any compatible system of units (U.S. customary, SI, or metric).

1.2 Nonconforming Shapes and Constructions

The official having jurisdiction (authority) may approve any alternate shape of constructions provided the proposed alternate is satisfactory and complies with the provisions of Section 6 of the Specification and the particular building code.

If there is insufficient evidence of compliance with the requirements of the particular building code, the authority administering the code may require tests, at the applicant's expense, as proof of compliance. Test procedures shall be as stipulated by Section 6 of the Specification. If there is no recognized or accepted test method, the authority may prescribe appropriate test procedures.

1.3 Material

1.3.1 Applicable Stainless Steels

The American Society for Testing and Materials (ASTM) is the basic source of stainless steel designations for this Specification. Section 1.3.1 contains a list of ASTM Standards for stainless steels that are designed by using this Specification. The types of stainless steels covered in various ASTM specifications with the corresponding UNS designations are listed in the following table:

ASTM Specification	Types Covered in ASTM Specification	UNS Designation[a]
A176	409	S40900
	430	S43000
A240	201	S20100
		S20400
	304	S30400
	316	S31600
	409	S40900
	430	S43000
	439	S43035
A276	201	S20100
	304	S30400
	316	S31600
	430	S43000
A666	201	S20100
		S20400
	301	S30100
	304	S30400
	316	S31600

[a] New designation established in accordance with ASTM Practice E527.

This Specification is applicable for S20400 (annealed and 1/4 hard) and for Types 201, 301, 304, and 316 (annealed, 1/16, 1/4, and 1/2 Hard) austenitic stainless steels and Types 409, 430, and 439 (annealed) ferritic stainless steels. The maximum thicknesses for Types 409, 430, and 439 stipulated in this Specification are based on their Ductile-to-Brittle Transition Temperature (DBTT) at room temperature or lower.

The important mechanical properties of cold-formed stainless steels for structural design are: yield strength, tensile strength, proportional limit, initial modulus of elasticity, tangent modulus, secant modulus, and ductility. Standard methods for testing of steels and steel products are given in ASTM A370. Determination of yield strengths or proportional limits for gradual yielding type of stainless steels can be done simply by using the offset method as shown in Figure C4. The distance om equals 0.002 in./in. (0.002 mm/mm) and 0.0001 in./in. (0.0001 mm/mm) for determining yield strength and proportional limit, respectively.

The ASTM Specification A176-85a[8] contains Types 409 and 430 stainless and heat-resisting chromium steel plate, sheet, and strip. The mechanical properties of Types 409 and 430 stainless steels specified in ASTM A176-85a are given in Table C1 of the Commentary. The mechanical properties of Type 439 stainless steel obtained from the ASTM Specification A240-86[9] are also included in Table C1. Type 439 is a stabilized version of Type 430 for weldability.

The ASTM Specification A276-85a[10] covers stainless and heat-resisting steel hot-finished or cold-finished bars and hot-rolled or extruded shapes. This ASTM Specification includes AISI Types 201, 304, 316, and 430 stainless steels used in this Specification.

The ASTM Specification A666-84[11] covers four types of austenitic stainless steels (Types 201, 301, 304, and 316) in the annealed, 1/16, 1/4, and 1/2 Hard-temper conditions. These stainless steels include sheet, strip, plate, and flat bar which are used primarily for architectural and structural applications. The mechanical properties of these stainless steels specified in ASTM A666-84 are given in Table C2.

The austenitic stainless steels used in architectural applications usually require a flattening operation as a last step in processing. This operation is almost always accomplished by a light cold-rolling pass. Sometimes

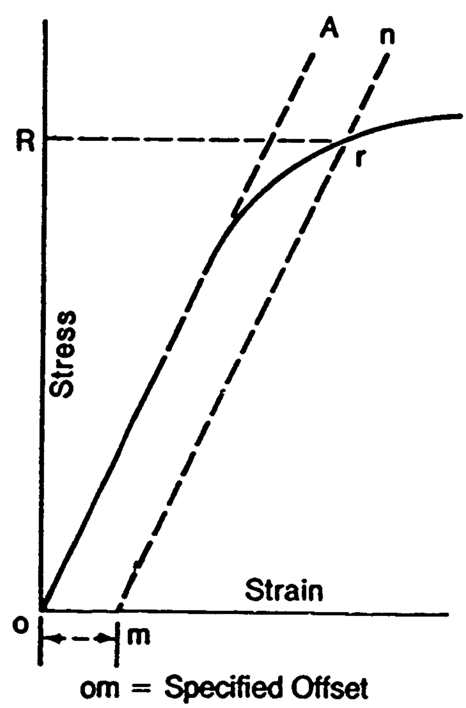


FIGURE C4. Stress-Strain Diagram Showing Yield Strength Determination

TABLE C1. ASTM Requirements for Mechanical Properties of Types 409, 430, and 439[8,9]

UNS designation	AISI type	Tensile strength min. ksi (MPa)	Yield[a] strength min. ksi (MPa)	Elongation in 2 in. (50.8 mm) min. (%)
S40900	409	55 (379.2)	30 (206.9)	22.0
S43000	430	65 (448.2)	30	22.0
S43035	439	65	30	22.0

[a] Yield strength is determined by 0.2% offset method described in Methods and Definitions, ASTM A370.

TABLE C2. ASTM Requirements for Mechanical Properties of Types 201, S20400, 301, 304, and 316[11]

UNS Designation	AISI type	Tensile strength min. ksi (MPa)	Yield strength min. ksi (MPa)	Elongation in 2 in. (50.8 mm) min. (%)
		Annealed		
S20100 Class 1	201-1	90 (620.6)	30 (206.9)	40.0
S20100	201-2	95 (655)	45 (310.3)	40.0
S20400		95	48 (330)	35.0
S30100	301	90	30	40.0
S30400	304	75 (517.1)	30	40.0
S31600	316	75	30	40.0
		1/16 Hard		
S20100 PSS[a]	201	95	45	40.0
FB[b]		75	40 (275.8)	40.0
S30100	301	90	45	40.0
S30400 PSS	304	80 (551.6)	45	35.0
FB		90	45	40.0
S31600 PSS	316	85 (586.1)	45	35.0
FB		90	45	40.0
		1/4 Hard		
S20100	201	125 (861.9)	75	25.0
S20400		140 (965)	100 (690)	20.0
S30100	301	125	75	25.0
S30400	304	125	75	10.0
S31600	316	125	75	10.0
		1/2 Hard		
S20100	201	150 (1,034.3)	110 (758.5)	15.0
S30100	301	150	110	15.0
S30400	304	150	110	6.0
S31600	316	150	110	6.0

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

TABLE C3. Tested Mechanical Properties of Annealed and Strain-Flattened Types 201-2 and 304 Austenitic Stainless Steels, Adapted from Reference 13

Type of Stainless Steels	Yield Strength F_y Mean ksi (MPa)	Yield Strength F_y St. dev. ksi (MPa)	Tensile Strength F_u Mean ksi (MPa)	Tensile Strength F_u St. dev. ksi (MPa)	No. of Tests
Type 201-2					
LT	52.16 (359.64)	1.05 (7.24)	108.05 (745)	3.63 (25.03)	6
TT	55.56 (383.08)	2.68 (18.48)	105.83 (729.7)	3.10 (21.37)	6
TC	55.14 (380.19)	2.41 (16.62)			5
LC	42.90 (295.79)	0.62 (4.27)			6
Type 304					
LT	42.10 (290.27)	1.28 (8.82)	98.08 (676.26)	2.08 (14.34)	32
TT	42.05 (289.93)	1.76 (12.14)	94.35 (650.54)	2.22 (15.31)	35
TC	44.68 (308.06)	1.17 (8.07)			24
LC	42.88 (295.66)	1.27 (8.76)			27

Note: St. dev. = Standard Deviation.
LT = Longitudinal Tension; TT = Transverse Tension.
LC = Longitudinal Compression; TC = Transverse Compression.

this operation is done by roller leveling or, in the case of cut length sheets, by stretching. This flattening operation results in a small reduction in thickness. When No. 2B finish is required, the stainless steel sheet should receive a light pass on bright, polished rolls.

Because austenitic stainless steels are very sensitive to cold-working, this will result in a slight directionality increasing the yield and tensile strengths and producing changes in the shape of stress-strain curves. To take this improvement of mechanical properties into account, experimental studies have been made on sheets in the strain-flattened condition.[12] Table C3 lists the tested yield and tensile strengths of annealed and strain-flattened Types 201-2 and 304 austenitic stainless steels based on Reference 13.

For 1/4 and 1/2 Hard-temper austenitic stainless steels, experimental studies have also been made to establish the mechanical properties for various types of stress in various directions.[14] Tables C4 and C5 list the tested yield and tensile strengths for 1/4 Hard and 1/2

TABLE C4. Tested Mechanical Properties of 1/4 Hard and 1/2 Hard Type 301 Austenitic Stainless Steels, Adapted from Reference 14

Type of stainless steels	Yield Strength F_y Mean ksi (MPa)	Yield Strength F_y St. dev. ksi (MPa)	Tensile Strength F_u Mean ksi (MPa)	Tensile Strength F_u St. dev. ksi (MPa)	No. of tests
Type 301, 1/4-Hard					
LT	85.9 (592.28)	5.92 (40.81)	137.9 (950.82)	—	17
TT	88.3 (608.83)	6.39 (44.06)	137.0 (944.61)	—	81
TC	100.0 (689.5)	5.31 (36.61)			17
LC	59.7 (411.63)	4.26 (29.37)			17
Type 301, 1/2-Hard					
LT	121.1 (834.98)	8.66 (59.71)	167.0 (1,151.47)	—	29
TT	113.9 (785.34)	8.58 (59.16)	168.1 (1,158.36)	—	93
TC	136.7 (942.54)	9.35 (64.47)			29
LC	82.1 (566.1)	9.13 (62.95)			29

TABLE C5. Tested Mechanical Properties of 1/4 Hard and 1/2 Hard Types 201 and 301 Austenitic Stainless Steels, Adapted from Reference 15

Type of stainless steels	Yield Strength F_y		Tensile Strength F_u		No. of tests
	Mean ksi (MPa)	St. dev. ksi (MPa)	Mean ksi (MPa)	St. dev. ksi (MPa)	
Type 201, 1/4-Hard					
LT	78.08 (538.36)	7.12 (49.48)	138.33 (953.79)	6.91 (47.64)	12
TT	85.50 (589.52)	4.24 (28.96)	130.00 (896.35)	1.41 (9.72)	2
Type 201, 1/2-Hard					
LT	139.58 (962.4)	8.89 (61.3)	172.71 (1,190.84)	7.00 (48.27)	118
TT	103.01 (710.25)	3.43 (23.65)	143.09 (986.61)	4.08 (28.13)	96
Type 301, 1/4-Hard					
LT	91.34 (629.79)	7.66 (52.82)	138.20 (952.89)	7.10 (49)	112
TT	87.06 (600.28)	7.24 (49.92)	130.44 (899.38)	3.36 (23.17)	9
Type 301, 1/2-Hard					
LT	142.95 (985.64)	12.81 (88.32)	166.73 (1,149.6)	9.17 (63.23)	244
TT	121.20 (835.67)	6.41 (44.2)	158.70 (1,094.24)	3.81 (26.25)	20
LT	116.92 (806.16)	10.12 (69.73)	169.63 (1,169.6)	6.09 (41.99)	487

Hard austenitic stainless steels obtained from References 14 and 15, respectively.

From Tables C3, C4, and C5, it can be seen that for the same stainless steel the yield strength varies with the direction and the type of stress. However, in the ASTM A666 Specification referred to in Section 1.3.1 of the Specification, only the minimum values for transverse tension are provided. A comparison of the tested yield strengths in Table C4 indicates that the tested yield strengths in transverse compression (TC) are the highest, while the values in the longitudinal compression (LC) are the lowest. Similar results can also be found in Table C3.

The tested yield and tensile strengths of Types 409, 430, and 439 annealed and strain-flattened ferritic stainless steels given in Table C6 were established on the basis of Reference 21. From this table it is noted that for these ferritic stainless steels, especially for Types 430 and 439, the tested-yield strengths are excessively larger than the ASTM specified values as given in Table C1. In order to obtain a reasonable material factor on yield strength to be used in the LRFD

TABLE C6. Tested Mechanical Properties of Types 409, 430, and 439 Ferritic Stainless Steels[21]

Type of stainless steels	Yield Strength F_y		Tensile Strength F_u		No. of tests
	Mean ksi (MPa)	St. dev. ksi (MPa)	Mean ksi (MPa)	St. dev. ksi (MPa)	
Type 409					
LT	34.2 (235.81)	2.50 (17.24)	58.6 (404.04)	1.93 (13.31)	15
TT	39.7 (273.73)	3.33 (22.96)	64.3 (443.35)	2.70 (18.62)	4977
TC	36.8 (253.74)	3.05 (21.03)			12
LC	34.9 (240.64)	2.06 (14.2)			14
Types 430 and 439					
LT	45.8 (315.79)	1.74 (12)	74.7 (515.06)	0.82 (5.65)	26
TT	52.2 (359.92)	3.92 (27.03)	78.4 (540.57)	4.23 (29.17)	4209
TC	52.3 (360.6)	1.99 (13.72)			27
LC	45.6 (314.41)	2.92 (20.13)			29

criteria for cold-formed stainless steels, the adjusted values of yield strength for Types 409, 430, and 439 ferritic stainless steels are needed to reflect the tested results. Accordingly, the adjusted yield strengths were used in this Specification on the basis of the test values. The ratios of the tested-to-specified yield strength for Types 409, 430, and 439 ferritic stainless steels are given in Table C7. Based on these modifications of the specified yield strength for Types 409, 430, and 439 ferritic stainless steels, the mean values and coefficients of variation of the ratios of tested to specified yield strengths are given in this table.

1.3.2 Other Stainless Steels

Although the use of ASTM-designated stainless steels listed in Specification Section 1.3.1 is encouraged, other stainless steels may also be used in cold-formed stainless steel structures, provided they satisfy the requirements stipulated in this provision.

1.3.3 Ductility

Ductility is the ability of a steel to undergo sizable plastic or permanent strains before fracturing and is important both for structural safety and for cold-forming. It is measured by the elongation in a 2-in. (50 mm) gage length. The ratio of the tensile strength-to-yield strength is also an important material property; this is an indication of the ability of the material to redistribute stress.

The requirements specified in Section 1.3.3 are adopted from Reference 16 for cold-formed carbon steels due to the lack of available research information for cold-formed stainless steels. Because the yield strength of stainless steels varies with the direction and the type of stress, the ratio of tensile strength-to-yield strength and the total elongation should meet the requirements of Section 1.3.3 in both longitudinal and transverse directions.

TABLE C7. Statistics on Yield Strengths of Types 409, 430, and 439 Ferritic Stainless Steels[21]

Type of stainless steels	Specified yield strength ksi (MPa)	(Tested F_y)/(Specified F_y) Mean	C.O.V.	Number of tests used in analysis
Type 409				
LT	30 (206.9)	1.1406	0.0727	15
TT	35 + (241.3)	1.1340	0.0844	4977
TC	35 +	1.0510	0.0829	12
LC	30	1.1615	0.0589	14
Total		1.1339	0.0844	5018
Types 430 & 439				
LT	40 + (275.8)	1.1459	0.0382	26
TT	45 + (310.3)	1.1613	0.0752	4209
TC	45 +	1.1631	0.0375	27
LC	40 +	1.1414	0.0642	29
All		1.1611	0.0748	4291

Note: For Types 409, 430, and 439 stainless steels, ASTM specified yield strength is 30 ksi, as given in Table C1.
C.O.V. = coefficient of variation.
+ Adjusted values.

1.3.4 Delivered Minimum Thickness

Sheet and strip stainless steels may be ordered to nominal or minimum thickness. If the stainless steel is ordered to minimum thickness, all thickness tolerances are over (+) and nothing under (−). If the stainless steel is ordered to nominal thickness, the thickness tolerances are divided equally between over and under. Therefore, in order to provide equity between the two methods of ordering sheet and strip stainless steels, the delivered thickness of a cold-formed product must be at least 95% of the design thickness. Thus, a portion of the factor of safety may be considered to cover minor negative thickness tolerances.

Generally, thickness measurements should be made in the center of elements. For decking and siding, measurements should be made as close as practical to the center of the first full flat of the section. Thickness measurements should not be made closer to edges than the minimum distances specified in ASTM A568.

The responsibility of meeting this requirement for a cold-formed product is clearly that of the manufacturer of the product, not the steel producer.

1.4 Loads

The general requirements for loads included in Specification Section 1.4 are the same as those used for the design of cold-formed carbon steels. This provision is adopted from Reference 2. The Specification does not establish the dead-, live-, or snow-loading requirements for which a structure should be designed. In most cases, these loads are adequately specified by the applicable building code or design standard. See Specification Section 1.6.

When a structure is subject to live loads which induce impact, recognized engineering procedures should be employed to reflect the effect of impact loads.

When the deflection of structural members is a critical factor in design, the designed stress should not exceed the proportional limit of stainless steels included in Table A17 of the Specification. Otherwise, permanent deflection may result from the inelastic behavior of members. When gravity and lateral loads produce forces of opposite sign in members, consideration should be given to the minimum gravity loads in combination with wind or earthquake loads.

When calculating the load on a relatively flat roof resulting from ponding of rainwater or snowmelt, the final deflected shape of the member should be considered. With regard to ponding, design guidance can be found from Section K2 of the AISC LRFD Specification for Structural Steel Buildings (Ref. 4) and Section 1110.4 of the BOCA National Building Code (Ref. 61).

1.5 Structural Analysis and Design

1.5.1 Design Basis

The current method of designing cold-formed stainless steel structural members, as presented in Reference 6, is based on the allowable stress design method. In this approach, the forces (bending moments, axial forces, shear forces) in structural members are computed by accepted methods of structural analysis for the specified working loads. These member forces or moments should not exceed the allowable values permitted by the Specification. The allowable load or moment is determined by dividing the nominal load or moment at a limit state by a factor of safety. The factors of safety used in the ASD Specification for the design of cold-formed stainless steel structural members are 1.85 for beams, 2.15 for columns, 2.4 for bolted connections, and 2.5 for welded connections.

A limit state is the condition at which the structural usefulness of a load-carrying element or member is impaired to such an extent that it becomes unsafe for the occupants of the structure, or the element no longer performs its intended function. Typical limit states for cold-formed steel members are excessive deflection, yielding, buckling, and attainment of maximum strength after local buckling (i.e., post-buckling strength). Reference 5 indicated that for cold-formed carbon steel members, these limit states have been established through experience in practice or in the laboratory, and they have been thoroughly investigated through analytical and experimental research.

The factors of safety are provided to account for the uncertainties and variabilities inherent in the loads, the analysis, the limit state model, the material properties, the geometry, and the fabrication. The allowable stress design method employs only one factor of safety for a limit state. The use of multiple load factors provides a refinement in the design which can account for the different degrees of the uncertainties and variabilities of the design parameters. Such a design method is called Load and Resistance Factor Design (LRFD), and its format is expressed by the following criterion:

$$\phi R_n \geq \Sigma \gamma_i Q_i \qquad \text{(C-1)}$$

where:

R_n = nominal resistance;
ϕ = resistance factor corresponding to R_n;
γ_i = load factor corresponding to Q_i;
Q_i = nominal load effect;
ϕR_n = design strength; and
$\Sigma \gamma_i Q_i$ = required resistance.

The left side of Eq. C-1, which is the design strength ϕR_n, represents a limiting structural capacity provided by the selected members. The nominal resistance R_n = the strength of the element or member for a given limit state computed for nominal section properties and for minimum specified material properties according to the appropriate analytical model which defines the strength. The resistance factor ϕ accounts for the uncertainties and variabilities inherent in the R_n, and it is usually less than unity. The right side of Eq. C-1 represents the required resistance, in which the nominal load effect Q_i = the forces on the cross section (bending moment, axial force, shear force) determined from the specified nominal loads by structural analysis, and y_i = the corresponding load factors which account for the uncertainties and variabilities of loads. The load factors are greater than unity.

The advantages of LRFD are: (1) Uncertainties and variabilities of different types of loads and resistances are different (e.g., dead load is less variable than wind load), and so these differences can be accounted for by use of multiple factors; and (2) by using probability theory, all designs can achieve ideally a uniform reliability. Thus LRFD provides the basis for a more rational and refined design method than is possible with the Allowable Stress Design method.

Probabilistic concepts. Factors of safety or load factors are provided against the uncertainties and variabilities which are inherent in the design process. Structural design consists of comparing nominal load effects Q to nominal resistances R, but both Q and R are assumed to be statistically independent random variables. Figure C5 represents frequency distributions for Q and R as separate curves for a hypothetical case. As long as the resistance R is greater than the effects of the loads Q, a

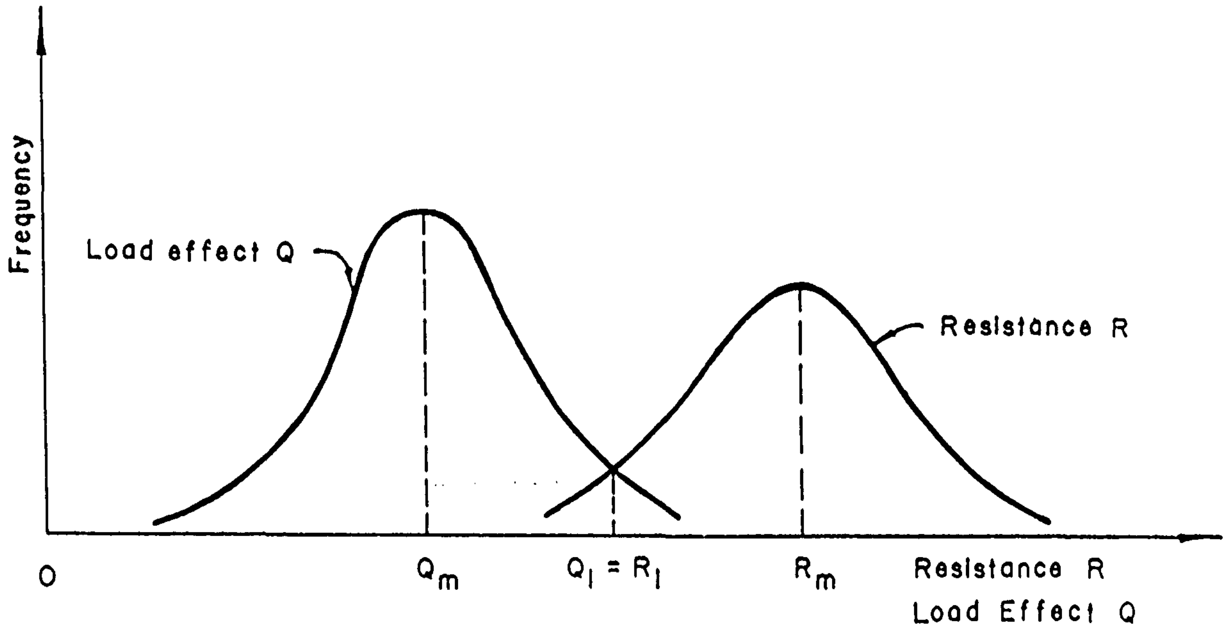


FIGURE C5. Frequency Distributions of Resistance and Load Effect[5]

margin of safety for the particular limit state exists. However, because Q and R are random variables, a limit state may be violated if $R < Q$ as indicated by the overlapping area of the distribution curves. While the possibility of this event ever occurring is never zero, a successful design should, nevertheless, have only an acceptably small probability of exceeding the limit state.

If the exact probability distributions of Q and R were known, then the probability of $(R - Q) < 0$ could be exactly determined for any design. In general the distributions of Q and R are not known, and only the means, Q_m and R_m, and the standard deviations, σ_Q and σ_R, can be estimated. Nevertheless, it is possible to determine relative reliabilities of several designs by an equivalent situation as illustrated in Figure C6. The distribution curve shown is for $\ln(R/Q)$, and the probability of exceeding a limit state is equal to the probability that $\ln(R/Q) \leq 0$ and is represented by the shaded area in the figure. The size of this area is dependent on the distance between the origin and the mean of $\ln(R/Q)$. For the given statistical data R_m, Q_m, σ_R and σ_Q, the area under $\ln(R/Q) \leq 0$ can be varied by changing the value of β (Figure C6). Since $\beta\sigma_{\ln(R/Q)} = \ln(R/Q)_m$, then the safety index can be expressed as follows:[5]

$$\beta = \frac{\ln\left(\dfrac{R_m}{Q_m}\right)}{\sqrt{V_R^2 + V_Q^2}} \qquad \text{(C-2)}$$

where: $V_R = \sigma_R/R_m$ and $V_Q = \sigma_Q/Q_m$, which are the coefficients of variation of R and Q, respectively. The factor β is called the "reliability index" or "safety index," and it is a relative measure of the safety of the design. When two designs are compared, the one with the larger β is more reliable. The detailed discussion on the determination of the safety index can be found in Reference 21.

The concept of the reliability index can be used in determining the relative reliability inherent in current design, and it can be used in testing out the reliability of new design formats, as illustrated by the following example of simply supported, braced, cold-formed stainless steel beams subjected to dead and live loads.

The design requirement of Reference 6 for such a beam is:

$$\frac{S_e F_y}{FS} = \left(\frac{L^2 s}{8}\right)(D_n + L_n) \qquad \text{(C-3)}$$

where:

S_e = elastic section modulus based on the effective section;
FS = 1.85 = factor of safety for bending;
F_y = specified yield strength;
L = span length; and s = beam spacing; and
D_n and L_n = respectively, code-specified dead and live load intensities.

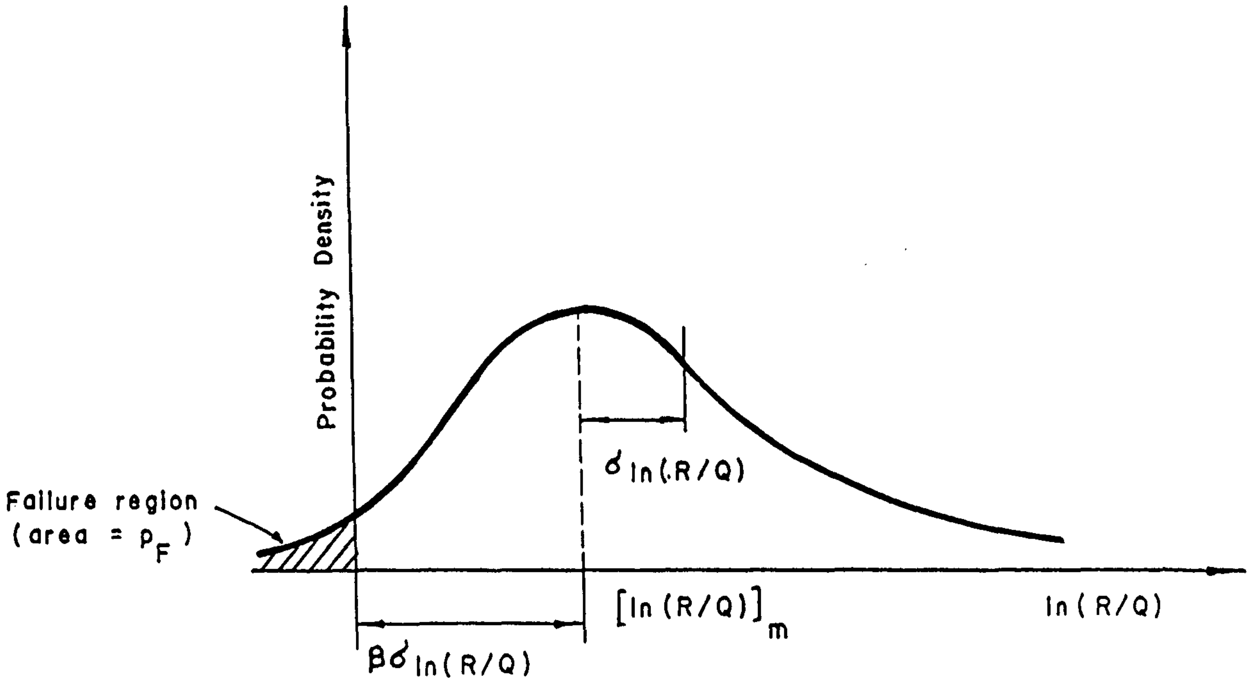


FIGURE C6. Probability Distribution of ln(R/Q)[5]

The mean resistance is defined as follows (Reference 16):

$$R_m = R_n(P_m M_m F_m) \qquad \text{(C-4)}$$

In this equation R_n = the nominal resistance, which in this case is

$$R_n = S_e F_y \qquad \text{(C-5)}$$

that is, the nominal moment predicted on the basis of the post-buckling strength of the compression flange. The mean values P_m, M_m, and F_m, and the corresponding coefficients of variation V_P, V_M and V_F, are the statistical parameters which define the variability of the resistance:

P_m = mean ratio of experimentally determined ultimate moment to predicted ultimate moment for actual material and cross-sectional properties of test specimens;
M_m = mean ratio of yield strength to minimum specified value;
F_m = mean ratio of section modulus to published (nominal) value;

The coefficient of variation of R equals

$$V_R = \sqrt{V_P^2 + V_M^2 + V_F^2} \qquad \text{(C-6)}$$

These parameters were obtained from examining the available tests on cold-formed stainless steel beams having different compression flanges with effective flanges and webs, and from analyzing test data on yield strengths and cross-sectional dimensions from many measurements. The statistical data used for this example are given below (Reference 21):

P_m = 1.19, V_p = 0.06; M_m = 1.10, V_M = 0.10; F_m = 1.0, V_F = 0.05. Thus, based on Eqs. C-4 and C-6, $R_m = 1.31R_n$ and $V_R = 0.13$.

The mean load effect is equal to

$$Q_m = \frac{L^2 s}{8}(D_m + L_m) \qquad \text{(C-7)}$$

and

$$V_Q = \frac{\sqrt{(D_m V_D)^2 + (L_m V_L)^2}}{D_m + L_m} \qquad \text{(C-8)}$$

where: D_m and L_m = the mean dead and live load intensities, respectively; and V_D and V_L = the corresponding coefficients of variation.

Load statistics have been analyzed in Reference 18, where it was shown that $D_m = 1.05D_n$, $V_D = 0.1$, $L_m = L_n$, and $V_L = 0.25$. The mean live load intensity equals the code live load intensity if the tributary area is small enough so that no live load reduction is included. Substitution of the load statistics into Eqs. C-7 and C-8 gives

$$Q_m = \frac{L^2 s}{8}\left(\frac{1.05D_n}{L_n} + 1\right)L_n \tag{C-9}$$

$$V_Q = \frac{\sqrt{(1.05D_n/L_n)^2 V_D^2 + V_L^2}}{\left(1.05\dfrac{D_n}{L_n} + 1\right)} \tag{C-10}$$

Q_m and V_Q thus depend on the dead-to-live-load ratio. Cold-formed beams typically have small D_n/L_n, and for the purposes of checking the reliability of these LRFD criteria it will be assumed that $D_n/L_n = 1/5$, and so $Q_m = 1.21L_n\ (L^2s/8)$ and $V_Q = 0.21$.

From Eqs. C-3 and C-5 we obtain the nominal design capacity, R_n, for $D_n/L_n = 1/5$ and $FS = 1.85$. Thus

$$\frac{R_m}{Q_m} = \frac{1.31 \times 1.85 \times 1.2 \times L_n\left(\dfrac{L^2 s}{8}\right)}{1.21L_n\left(\dfrac{L^2 s}{8}\right)} = 2.4$$

and, from Eq. C-2

$$\beta = \frac{ln(2.40)}{\sqrt{0.13^2 + 0.21^2}} = 3.58$$

Of itself, $\beta = 3.58$ for cold-formed stainless steel beams having different compression flanges with effective flanges and webs designed by the proposed ASCE ASD Specification means nothing. However, when this is compared to β for other types of cold-formed carbon steel members, and to β for designs of various types of hot-rolled steel shapes or even for other materials, then it is possible to say that this particular cold-formed stainless steel beam has a relatively high reliability (References 19, 20).

Basis for LRFD of Cold-Formed Stainless Steel Structures. A great deal of work has been performed for determining the values of the reliability index β inherent in traditional design, as exemplified by the current structural design specifications, such as the AISC Specification for hot-rolled steel, the AISI Specification for cold-formed steel, the ACI Code for reinforced concrete members, etc. The studies for hot-rolled and cold-formed carbon steels are summarized in References 16 and 17, respectively, where many further papers are also referenced which contain additional data.

The determination of β for cold-formed stainless steel elements or members is studied in Reference 21, where both the basic research data as well as the β's determined from the calibrations of the LRFD provisions are presented in great detail. The β's computed in the referenced publications noted were developed with slightly different load statistics than those of this Commentary, but the essential conclusions remain the same.

The entire set of data for hot-rolled steel and cold-formed carbon steel designs, as well as data for reinforced concrete, aluminum, laminated timber, and masonry walls was reanalyzed in References 18, 19, and 20 by using: (1) updated load statistics; and (2) a more advanced level of probability analysis which was able to incorporate probability distributions which describe the true distributions more realistically. The details of this extensive reanalysis are presented in the aforementioned references and also only the final conclusions from the analysis are summarized here:

1. The values of the reliability index β vary considerably for the different kinds of loading, the different types of construction, and the different types of members within a given material design specification. In order to achieve more consistent reliability, it was suggested in Reference 20 that the following values of β would provide this improved consistency while at the same time give, on the average, essentially the same design by the new LRFD method as is obtained by current design for all materials of construction. These target reliability indices β_o for use in LRFD are:

Basic case:	Gravity loading, $\beta_o = 3.0$
For connections:	$\beta_o = 4.5$
For wind loading:	$\beta_o = 2.5$

These target reliability indices are the ones inherent in the load factors recommended in the ANSI/ASCE 7-88 Load Code (Ref. 22).

For simply supported, braced cold-formed stainless steel beams with stiffened flanges, which were designed according to this Specification, it was shown that for the representative dead-to-live load ratio of 1/5 the computed reliability index $\beta = 3.04$ (Ref. 21). Considering the fact that for other such load ratios, or for other types of members, the reliability index inherent in the current cold-formed stainless steel construction could be more or less than this value of 3.04, the basic target reliability index of $\beta_o = 3.0$ is recommended as a lower limit for the new LRFD criteria. The resistance factors ϕ were selected such that $\beta_o = 3.0$ is essentially the lower bound of the actual β's for members. In order to assure that failure of a structure is not initiated in the connections, a higher target reliability index of $\beta_o = 4.0$ is recommended for joints and fasteners. These two targets of 3.0 and 4.0 used for cold-formed stainless steel members and connections, respectively, are relatively higher than those recommended for the design of cold-formed carbon steel structural members and connections.

2. The following load factors and load combinations were developed in References 18 and 20 to give essentially the same β's as the target β_o's, and are recommended for use with the 1988 ANSI/ASCE Load Code (Ref. 22) for all materials:

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

where:

D_n = nominal dead load;
E_n = nominal earthquake load;
L_n = nominal live load due to occupancy;
L_{rn} = nominal roof life load;
R_{rn} = nominal roof rain load;
S_n = nominal snow load; and
W_n = nominal wind load.

In view of the fact that the dead load of cold-formed stainless steel structures is usually smaller than that of heavy construction, the first case of load combinations included in Section 1.5.2 of the Specification is $(1.4D_n + L_n)$ instead of the ANSI value of $1.4D_n$. This requirement is identical with the ANSI/ASCE Code when $L_n = 0$.

It should be noted that for the third case of load combinations, the load factor used for the nominal roof live load, L_{rn}, in Section 1.5.2 of this Specification is 1.4 instead of the ANSI/ASCE value of 1.6. A relatively small load factor is used because the roof live load is due to the presence of workmen and materials during repair operations and therefore can be considered a type of construction load.

For roof and wall construction, it is recommended that the load factor for the nominal wind load W_n to be used for the design of individual purlins, girts, wall panels, and roof decks be multiplied by a reduction factor of 0.9 because these elements are secondary members subjected to a short duration of wind load and thus can be designed for a smaller reliability than primary members such as beams and columns. For example, the reliability index of a wall panel under wind load alone is approximately 1.5 with this reduction factor. Deflection calculations for serviceability criteria should be made with the appropriate unfactored loads.

The load factors and load combinations given are recommended for use with the LRFD criteria for cold-formed stainless steel structural members. The following portions of this Commentary present the background for determining the following resistance factors ϕ which are recommended for the design strength of members and connections in Sections 3 and 5:

Type of Strength	*Resistance Factor, ϕ*
1. Stiffeners	
Transverse stiffeners	0.85
Shear stiffeners	0.85
2. Tension members	0.85
3. Flexural members	
Bending strength:	
For sections with stiffened or partially stiffened compression flanges	0.90
For sections with unstiffened compression flanges	0.85
Laterally unbraced beams	0.85
Web design:	
Shear yielding strength	0.95
Shear buckling strength	0.85
Web crippling	
For single unreinforced webs	0.70
For I-sections	0.70
4. Concentrically loaded compression members	0.85
5. Combined axial load and bending:	
ϕ_c for compression	0.85
ϕ for bending	
Using Section 1.3.1.1	0.85 or 0.90
Using Section 1.3.1.2	0.85
6. Cylindrical tubular members	
Bending strength	0.90
Axial compression	0.80
7. Welded connections	
Groove welds	
Tension or compression	0.60
Longitudinal fillet welds	
Connected part	0.55
Welds	0.55
Transverse fillet welds	
Connected part	0.55
Welds	0.65
Resistance welds	0.60
8. Bolted connections	
Minimum spacing and edge distance	0.70

Tension strength on net section	0.70
Bearing strength	0.65
Shear strength of bolts	0.65
Tensile strength of bolts	0.75

These ϕ factors are determined in conformance with the load factors given above to approximately provide a target β_o of 3.0 for members and 4.0 for connections, respectively, for the load combination of $1.2D_n + 1.6L_n$. For practical reasons it is desirable to have relatively few different resistance factors, and so the actual values of β will differ from the derived targets. This means that

$$\phi R_n = c(1.2D_n + 1.6L_n) = \left(\frac{1.2D_n}{L_n} + 1.6\right)cL_n \qquad \text{(C-11)}$$

where c is the deterministic influence coefficient translating load intensities to load effects.

By assuming $D_n/L_n = 1/5$, Eqs. C-11 and C-9 can be rewritten as follows:

$$R_n = 1.84\left(\frac{cL_n}{\phi}\right) \qquad \text{(C-12)}$$

$$Q_m = \left(\frac{1.05D_n}{L_n} + 1\right)cL_n = 1.21cL_n \qquad \text{(C-13)}$$

The mean resistance, R_m, can be obtained by substituting Eq. C-12 into Eq. C-4 as

$$R_m = 1.84\frac{cL_n}{\phi}M_mF_mP_m \qquad \text{(C-14)}$$

Therefore:

$$\frac{R_m}{Q_m} = \frac{1.521}{\phi}M_mF_mP_m \qquad \text{(C-15)}$$

Alternatively, based on Eq. C-2, the ratio of R_m/Q_m can be expressed as

$$\frac{R_m}{Q_m} = \exp\left(\beta\sqrt{V_R^2 + V_Q^2}\right) \qquad \text{(C-16)}$$

Thus, the resistance factors ϕ can be computed by equating Eqs. C-15 and C-16 as follows:

$$\phi = \frac{1.521M_mF_mP_m}{\exp\left(\beta\sqrt{V_R^2 + V_Q^2}\right)} \qquad \text{(C-17)}$$

1.5.4 Yield Strength and Strength Increase from Cold Work of Forming

For yield strengths and tensile strengths of various types of stainless steels, see Section 1.3 of the Commentary on Material.

The mechanical properties, such as yield strength, tensile strength, and elongation, of the cold-formed stainless steel section may differ from the properties exhibited by the flat material prior to forming. This difference can be attributed to cold working of the steel sheet during the forming process. A combination of strain hardening, resulting from stretching of the sheet during the forming process, and strain aging causes an increase in the yield strength and tensile strength and a decrease in ductility.

Previous study indicated that the strengthening effect produced by cold-forming in corners is the largest in the annealed state and decreases with increasing hardness of the material, becoming almost negligible for the Full-Hard grades.[14] Tables 4 and 7 of Reference 23 revealed that for annealed and strain-flattened stainless steels, the potential increase in member strength caused by cold-forming ranges from 5 to 11% for flexural strength, and 14 to 24% for column strength for the particular sections investigated.

Section 1.5.4 of the Specification permits utilization of the cold work of forming under certain conditions of Sections 1.5.4.2.1 and 1.5.4.2.2. Because Karren's formula (Reference 24) was originally developed on the basis of the test data for carbon steels, the equation may not be applicable for determining the yield strength of corners for stainless steels. Appropriate modifications to this formula are given in Reference 64 for stainless steel Types 304, 409, and 430. Any use of the strength increase from cold work must be based on full section tests. Results of such tests are reported for annealed AISI Types 304 and 316 stainless steels in Reference 65.

The following statistical data (mean values and coefficients of variation) on material and cross-sectional properties were obtained from Reference 21 for use in the derivation of the resistance factors ϕ:

$$(F_y)_m = 1.10F_y; \quad M_m = 1.10; \quad V_{Fy} = V_M = 0.10$$

$$(F_u)_m = 1.10F_u; \quad M_m = 1.10; \quad V_{Fu} = V_m = 0.05$$

$$F_m = 1.00; \quad V_F = 0.05$$

The subscript m refers to mean values. The symbol V stands for coefficient of variation. The symbols M and F are, respectively, the ratios of the mean-to-nominal material property and cross-sectional property; and F_y and F_u are, respectively, the specified minimum yield strength and the specified minimum tensile strength.

These data are based on the analysis of many samples, and they are representative properties of materials and cross sections used in the industrial applications of cold-formed stainless steel structures.

1.5.5 Design Tables and Figures

The design tables and figures given in Specification Appendix A provide the information needed for the design of seven types of stainless steels used in this Specification. These technical data deal with yield strength, secant modulus, initial modulus of elasticity, tangent modulus, and plasticity reduction factors. In addition, design tables for tensile strengths of weld metals and base metals for welded connections are also included. For more detailed discussion, see Appendix A of the Commentary.

The secant modulus, tangent modulus, and plasticity reduction factor can be determined by using the analytical expressions as given in Appendix B. The modified Ramberg-Osgood equation is used in this Specification. For more discussions on this subject, see Appendix B of the Commentary.

1.6 Reference Documents

The references listed in Section 1.6 of the Specification pertain to various aspects of cold-formed stainless steel design and provide useful information to aid the design engineer. If conflict occurs between the ASCE Standard and reference documents, the design of cold-formed stainless steel structural members and connections should be governed by the ASCE Standard.

2. ELEMENTS

In this LRFD Specification, the effective design width approach is applied to stiffened and unstiffened compression elements.

2.1 Dimensional Limits and Considerations

Because of the relatively large flat-width-to-thickness ratios (w/t) that are possibly used in cold-formed stainless steel construction, dimensional limits are established in the Specification. Other phenomena, e.g., flange curling and shear lag effects, are also considered in the Specification.

2.1.1 Flange Flat-Width-to-Thickness Considerations

a. *Maximum Flat-Width-to-Thickness Ratios.* The limits imposed in Specification Section 2.1.1a are the same as those used in Reference 6. These limitations on maximum allowable overall w/t ratios are selected on the basis of experience and general practice and are proved to be reasonable for the cold-formed stainless steel construction. The additional requirements for stiffeners based on the values of I_s and I_a and the ratio of D/w according to Section 2.4.2 are the same as those specified in Reference 2 for cold-formed carbon steel members.

The limitation on a maximum w/t ratio of 50 for compression elements stiffened by a simple lip is based on the fact that the stiffening lip itself is an unstiffened element. The limitation to $w/t = 90$ for flanges with stiffeners other than simple lips is to prevent possible damage of such flexible flanges in transportation, handling, and erection.

The maximum allowable overall w/t ratio for stiffened compression elements with both longitudinal edges connected to a web or a stiffened flange element is limited to 400. It has been shown that this limit is reasonable and achievable for cold-formed stainless steel members. In such cases where the limits are exceeded, tests in accordance with Specification Section 6 are required.

The note regarding noticeable deformation for large flat-width-to-thickness ratios is a caution and is not intended to prevent the use of such compression elements. However, when it is necessary to use these elements with large w/t ratios, the design requirements of local distortions should be used for flexural and compression members as specified in Specification Sections 3.3.1.1 and 3.4, respectively.

In addition, Reference 25 indicates that buckling of unbacked sheet may be developed due to thermal effect if the w/t ratio of stiffened elements exceeds 150 to 200, depending on the surface finish of the sheet.

b. *Flange Curling.* In order to limit the maximum amount of curling or movement of unusually wide, thin flanges toward the neutral axis of beams, a formula (Eq. 2.1.1-1) is included in Specification Section 2.1.1 to determine the maximum permissible flange width, w_f, for a given amount of tolerable curling, c_f. This formula is the same as that included in Reference 6. The modulus of elasticity used for stainless steel members should be based on the value given in Tables A4 and A5 of the Specification.

It should be noted that this provision does not stipulate the amount of curling which can be regarded as tolerable. For carbon and low alloy steels, Reference 6 suggests that curling on the order of 5% of the depth is usually not considered excessive. The designer must establish the amount of curling on the basis of the kind of section used and good engineering practice. In general, it is essential to control closely out-of-plane distortions of unusually wide flanges for the sake of appearance.

c. *Shear Lag Effects.* The Specification contains the design provision for beams having relatively small span-to-width ratio, L/w_f, and subject to concentrated loads. The specified reduction of flange width is due to the effect of shear lag.

The phenomenon of shear lag is described by Winter[26] as follows: "In metal beams of the usual shapes, the normal stresses are induced in the flanges through shear stresses transferred from the web to the flange. These shear stresses produce shear strains in the flange which, for ordinary dimensions, have negligible effects. However, if flanges are unusually wide (relative to their length) these shear strains have the effect that the normal bending stresses in the flanges decrease with increasing distance from the web. This phenomenon is known as shear lag. It results in a nonuniform stress distribution across the width of the flange, similar to that in stiffened compression elements, though for entirely different reasons. As in the latter case, the simplest way of accounting for this stress variation in design is to replace the nonuniformly stressed flange of actual width w_f by one of reduced, effective width subject to uniform stress."

Previous study has indicated that the effect of shear lag depends upon the ratio of E_o/G_o, where E_o is the modulus of elasticity and G_o is shear modulus. In view of the fact that the E_o/G_o ratio is about the same for all grades of stainless steels, the design provision of the proposed ASCE ASD Specification[6] is retained in this LRFD Specification. It should be noted that the flange width, w_f, in this case is the projection beyond the web, not the flat portion of the flange, as is the case in subsequent sections of Section 2.

For a uniform load the width reduction due to shear lag is practically negligible except that the span length is extremely short. No provision is included in Section 2.1.1 for reduction of flange width for such a case.

2.1.2 Maximum Web Depth-to-Thickness Ratio

The limits prescribed in Specification Section 2.1.2 are the same as those used in the proposed ASCE ASD Specification.[6] The maximum ratio of h/t for flexural members having unreinforced webs is limited to a value of 200. This is based on the vertical buckling of webs subjected to transverse flange forces developed from the curvature of the beam. With webs provided with transverse stiffeners which satisfy the requirements of Specification Appendix C.1, the limits of the allowable depth-to-thickness ratio are stipulated as 260 for webs using bearing stiffeners only and as 300 for webs using bearing stiffeners and intermediate stiffeners. These limits are the same as those used in the current AISI Specification[2] for the design of cold-formed carbon steel sections.

It should be noted that the definition for h is the depth of the flat portion of the web measured along the plane of the web.

2.2 Effective Widths of Stiffened Elements

It is well known that plate and sheet elements possess a large strength reserve after buckling, unless buckling occurs at stresses approaching the yield strength for sharp yielding materials or at large inelastic strains for materials such as stainless steels which do not have a definite yield strength. For example, Figure C7 shows the buckled form of a stiffened compression element (a sheet which is supported along both unloaded edges by thin webs and can be regarded as simply supported), uniaxially loaded by a compression force. Although the element has buckled, and out-of-plane waves have developed, it is still capable of sustaining additional load, and the member of which the element is a part does not collapse. This behavior is a result of the membrane stresses (post-buckling strength) which are developed in the element transverse to the direction of loading. Unstiffened compression elements (sheets which are supported along one

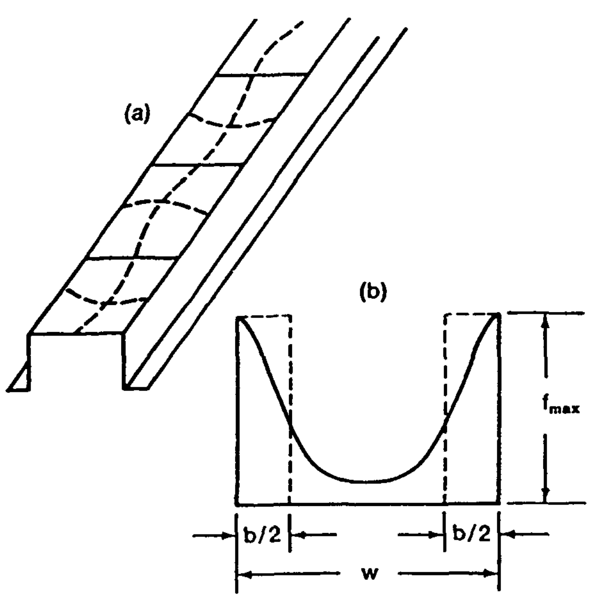


FIGURE C7. Local Buckling and Post-Buckling Strength of Stiffened Compression Element

unloaded edge only, the other unloaded edge being unsupported) behave in a similar manner, except that the strength reserve after buckling is relatively small because less membrane action is possible. The strength of partially stiffened stainless steel compression elements is studied in Reference 77 by experimental and theoretical means.

The general equation for the critical buckling stress of isotropic sheet element is

$$\sigma_{cr} = \frac{\pi^2 \eta k E_o}{12(1-\mu^2)\left(\frac{w}{t}\right)^2} \quad \text{(C-18)}$$

where:

k = buckling coefficient;
E_o = initial modulus of elasticity;
μ = Poisson's ratio in the elastic range;
η = plasticity reduction factor;
w = flat width; and
t = thickness.

To keep the width-to-thickness ratio reasonably small, thus maintaining larger critical stresses, compression elements are frequently provided with intermediate longitudinal stiffeners between a web and an edge stiffener (Figure C3).

In practical design the effective width concept is widely used for taking the post-buckling strength of compression elements into account. Figure C7(b) shows the post-buckling stress distribution in a stiffened compression element. The solid line is the actual stress distribution over the actual element width, w. The dashed line is the equivalent uniform stress distribution, equal in intensity to the edge stress of the actual distribution, but only applied over an effective width b. The total load carried by the element is the same for both distributions.

The effective width concept is now used explicitly in computing the properties of sections which contain stiffened and unstiffened compression elements. Because the effective width is a function of the element edge stress, it follows that the properties of the section are also functions of the stress level. For this reason, when computing the effective area, moment of inertia, and section modulus, proper recognition must be given to the effective width of stiffened and unstiffened compression elements as a function of the edge stress and the flat-width-to-thickness ratio.

In this LRFD Specification, the effective design width approach is adopted for: (1) Uniformly compressed stiffened elements; and (2) webs and stiffened elements with stress gradients.

2.2.1 Uniformly Compressed Stiffened Elements

a. *Load Capacity Determination.* The effective design width expression as given in Eq. C-19 was originally derived by Winter for the design of cold-formed carbon steel members. It has been verified by Johnson and Wang for using stainless steel members.[12,14]

$$\frac{b}{t} = 1.9\sqrt{\frac{E_o}{f_{max}}}\left[1 - \frac{0.475\sqrt{E_o/f_{max}}}{w/t}\right] \quad \text{(C-19)}$$

where:

b = effective design width;
E_o = initial modulus of elasticity;
f_{max} = maximum stress at edge;
w = flat width; and
t = thickness.

Based on a long-time accumulated experience in the design of cold-formed carbon steel structural members, a more realistic equation as follows was used in the AISI Specification for the design of cold-formed carbon steel structural members since 1968:

$$\frac{b}{t} = 1.9\sqrt{\frac{E_o}{f_{max}}}\left[\frac{1 - 0.415\sqrt{E_o/f_{max}}}{w/t}\right] \quad \text{(C-20)}$$

Consequently, an additional study has been made by Wang on the suitability of Eq. C-20 for stainless steel structural members.[14] Because Eq. C-20 compares favorably with the experimental data obtained from numerous beam and column tests as shown in Figure C8, this equation was used in the 1974 Edition of the AISI Specification for the design of stainless steel structural members.[3]

In Reference 6, Eq. C-20 is retained for determining the effective design width for uniformly compression stiffened elements. However, this equation is expressed in terms of the ratio of b/w as follows:

$$\frac{b}{w} = \frac{1}{\lambda}\left(1 - \frac{0.22}{\lambda}\right) \quad \text{(C-21)}$$

where λ is a slenderness factor determined as

$$\lambda = \sqrt{\frac{f_{max}}{f_{cr}}} \quad \text{(C-22)}$$

f_{cr} is the critical buckling stress of an isotropic sheet element calculated from Eq. C-18.

In this LRFD Specification, the effective design-width formulas are the same as those used in the proposed ASCE ASD Specification.[6] In Section 2.2, Eq. 2.2.1-4 for calculating the slenderness factor, λ, is ob-

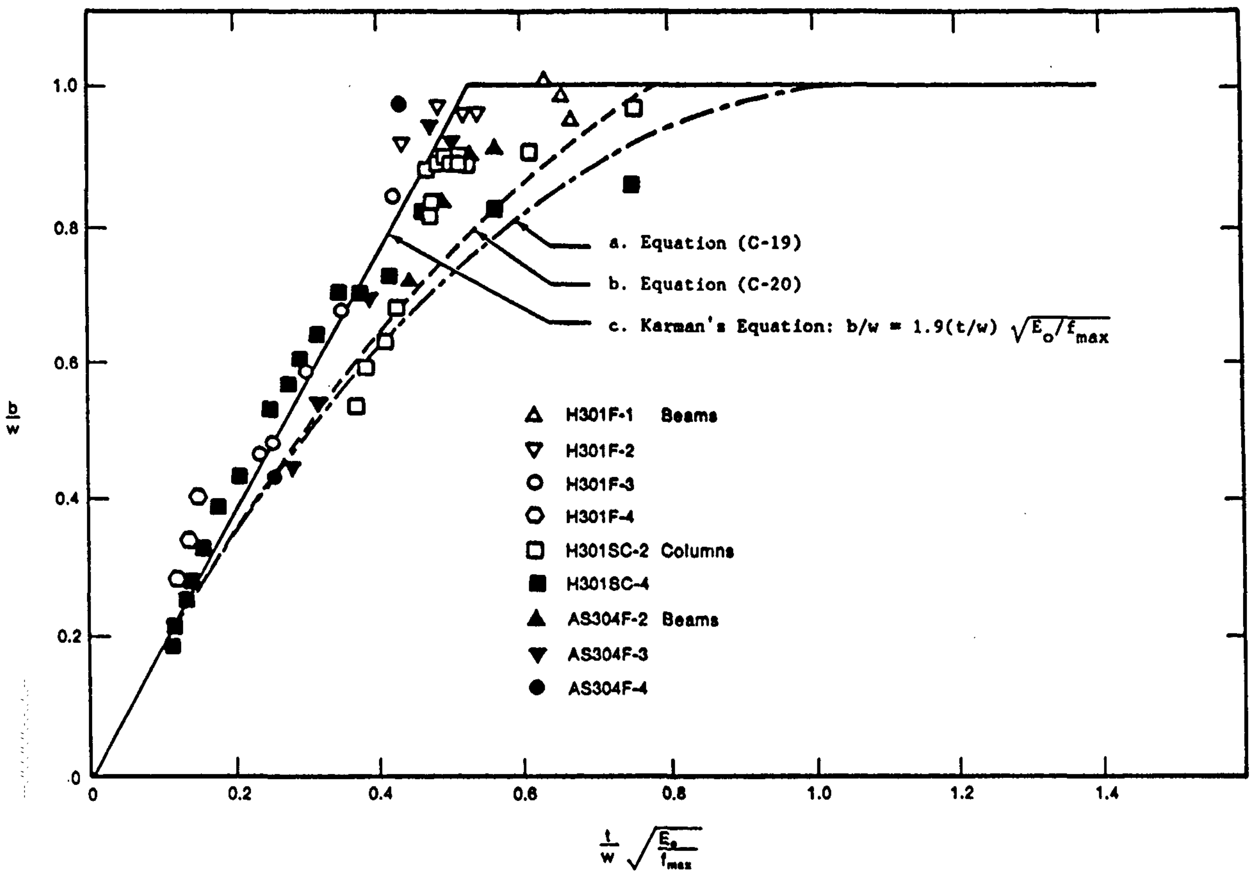


FIGURE C8. Correlation Between Effective Width Formula and Test Data[3]

tained from Eq. C-22 by substituting the value of f_{cr} from Eq. C-18 with $\eta = 1.0$. The use of $\eta = 1.0$ for the effective design width has been verified for cold-formed stainless steel members having stiffened compression elements.[26] For the design of local distortions, the plasticity reduction factor, η, may be taken as $\sqrt{E_t/E_o}$ for uniformly compressed stiffened elements of stainless steel.[12,27,28] The buckling coefficient, k, depends upon the boundary conditions and the aspect ratio of the plate element. It is taken as 4.0 for long stiffened elements supported by a web on each longitudinal edge.

b. *Deflection Determination.* The effective design width used for deflection calculation can be determined by the same equation from Specification Section 2.2.1a except that the stress is evaluated on the basis of the actual stress level and the reduced modulus of elasticity, E_r, instead of E_o. For beam sections generally used in cold-formed stainless steel construction, the average of the secant moduli corresponding to the stresses in both tension and compression flanges is recommended for the reduced modulus. Experimental verification of this provision is given in Reference 23. The Ramberg-Osgood equation used to determine the secant modulus is discussed in Appendix B of the Commentary.

2.2.2 Effective Widths of Webs and Stiffened Elements with Stress Gradient

Due to the lack of sufficient test results of stiffened elements with stress gradient on cold-formed stainless steel members, this Section is adopted from the AISI 1986 Specification for the design of cold-formed carbon steel sections.[2]

The use of effective design widths for web elements subjected to a stress gradient is a deviation from the past practice of using a full area of web in conjunction with a reduced stress to account for local buckling. The effective widths are based on Winter's effective width equation distributed as shown in Figure 2 of the Specification.

2.3 Effective Widths of Unstiffened Elements

The effective design width approach is also applied to members consisting of unstiffened compression elements in the current Specification because the research results reported in Reference 29 for cold-formed carbon steel structural members indicate that Winter's effective design width equation is an adequate predictor of section capacity if the appropriate buckling coefficient, *k,* is employed.

The provisions included in Specification Section 2.3 are the same as those used in Reference 2 except that the buckling coefficient, *k,* is taken as 0.5 for cold-formed stainless steel members.

2.3.1 Uniformly Compressed Unstiffened Elements

The theoretical buckling coefficient for an ideally flat, long unstiffened element having hinged and fixed edge conditions is 0.425 and 1.277, respectively.[28] Previous test results of cold-formed stainless steel members indicated that the buckling coefficient can be taken as 0.85 for I-sections made by two channels connected back to back.[27] In Reference 6, the buckling coefficient was conservatively taken as 0.5 for unstiffened compression elements. This value has been retained in this LRFD Specification. The plasticity reduction factor, η, used for the design of local distortions may be taken as E_s/E_o for members having uniformly compressed unstiffened elements.[12,14,30,31]

2.3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

Reference 29 shows that by using Winter's effective design width equation in conjunction with a $k = 0.43$, good correlation can be achieved between test and calculated capacity for unstiffened elements and edge stiffeners with stress gradient using cold-formed carbon steel. This same trend is also true for deflection determination. Because no test results are available for cold-formed stainless steel members regarding this topic, the provisions included in Specification Section 2.3.2 are similar to those used in Reference 2 except that the buckling coefficient, *k,* is equal to 0.5 for stainless steel members.

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

Because the effective design width of stainless steel elements with an edge stiffener or one intermediate stiffener has not been investigated in the past, Section 2.4 of this Specification is adopted from the 1986 Edition of the AISI Specification.[2]

2.4.1 Uniformly Compressed Elements with Intermediate Stiffener

This LRFD Specification contained provisions for the minimum required moment of inertia, which was based on the assumption that an intermediate stiffener needed to be twice as rigid as an edge stiffener. This provision is adopted from Reference 2 which includes expressions for evaluating the required stiffener rigidity based on the geometry of the contiguous flat elements. By using the ratio of actual stiffener moment of inertia, I_s, to adequate stiffener moment of inertia, I_a, (i.e., I_s/I_a) to evaluate the buckling coefficient and the stiffener area, a partially stiffened compression flange can be evaluated.

In this provision, three cases are identified, i.e.: (1) Element is fully effective as a stiffened element and does not need an intermediate stiffener; (2) element can be fully effective as a stiffened element when it has an adequate stiffener; and (3) element is not fully effective even with an adequate stiffener.

2.4.2 Uniformly Compressed Elements with Edge Stiffener

This LRFD Specification provided an equation for the minimum moment of inertia for an edge stiffener and an expression for the minimum overall depth of a simple stiffening lip. Section 2.4.2 of the Specification contains three different cases for the determination of the effectiveness of compression elements and stiffeners. These design provisions are the same as those used in the proposed ASCE ASD Specification.[6]

The interaction of the plate elements, as well as the degree of edge support, full or partial, is compensated for in the expressions for the buckling coefficient, *k,* and the depth and area of the stiffener. For more information, see Reference 29.

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

This Specification retains Eq. 2.5-1 from Reference 6 for evaluating the minimum required rigidity, I_{min}, of an intermediate stiffener for a multiple-stiffened element. It is based on the assumption that an intermediate stiffener must stiffen two compression elements, while an edge stiffener is required to stiffen only one such element. For this reason, the minimum required rigidity of an intermediate stiffener is specified to be twice that of an edge stiffener.

In addition, Specification Section 2.5(a) stipulates that only intermediate stiffeners adjacent to web elements (see Figure C3) shall be counted as effective. Additional stiffeners would have two or more sub-elements between themselves and the nearest shear-transmitting element (i.e., web) and hence, could be ineffective. Specification Section 2.5(b) applies the same reasoning to intermediate stiffeners between a web and an edge stiffener.

If intermediate stiffeners are spaced so closely that the sub-elements are fully effective, i.e., $b = w$, no plate-buckling of the sub-element will occur. Therefore, the entire assembly of sub-element and intermediate stiffeners between webs behaves like a single compression element whose rigidity is given by the moment of inertia, I_s, of the full, multiple-stiffened element, including stiffeners. Although the effective width calculations are based on an equivalent element having width, w_s, and thickness, t_s, the actual thickness must be used when calculating section properties.

2.6 Stiffeners

For detailed discussions, see Appendix C.

3. MEMBERS

In this Specification, the design provisions for cold-formed stainless steel structural members are given by the design strength, i.e., ϕR_n. To clarify the phenomenon being evaluated, the nominal capacity, R_n, and the resistance factor, ϕ, are explicitly stated in this Section.

3.1 Properties of Sections

The geometric properties of a member shall be evaluated using conventional methods of structural design. These properties are based on either full cross-section dimensions, effective widths, or net section, as applicable.

For flexural members and axially loaded members, both the full and effective dimensions are used. The full dimensions are used when calculating the critical load or moment, while the effective dimensions, evaluated at the stress corresponding to the critical load or moment, are used to calculate the nominal capacity.

3.2 Tension Members

The design of tension members is based on the limit state that yielding failure occurs on the net area of the member. When mechanical facteners are used, the design should also be limited by Section 5.3.2 of the Specification.

The resistance factor ϕ used for the design of cold-formed stainless steel tension members is taken as 0.85, which is smaller than that used for cold-formed carbon steel members.[5] This ϕ factor was derived from the procedure described in Section 1.5.1 of this Commentary using a selected β_o value of approximately 3.0. In the determination of the resistance factor, the following formulas were used for R_m and R_n:

$$R_m = A_n(F_y)_m \qquad \text{(C-23)}$$

$$R_n = A_nF_y \qquad \text{(C-24)}$$

$$\text{i.e.,} \quad \frac{R_m}{R_n} = \frac{(F_y)_m}{F_y} \qquad \text{(C-25)}$$

in which: A_n = the net area of the cross section; and $(F_y)_m = 1.10F_y$, as discussed in Section 1.5.2 of the Commentary. The statistical analysis of material properties for stainless steels is reported in Reference 21. By using $V_M = 0.10$, $V_F = 0.05$, and $V_P = 0$, the coefficient of variation V_R is:

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} = 0.11 \qquad \text{(C-26)}$$

Based on $V_Q = 0.21$ and the resistance factor of 0.85, the value of β is 2.9, which is close to the stated target value of $\beta = 3.0$.

3.3 Flexural Members

Section 3.3 of the Specification provides various design aspects related to flexural strength, lateral buckling strength, shear strength, combined bending and shear strength, web crippling strength, and combined bending and web crippling strength. For brace design, see Specification Section 4.3.

3.3.1 Strength for Bending Only

Bending strengths of flexural members are differentiated according to whether or not the member is laterally braced. If such members are laterally supported, then they are proportioned according to the nominal section strength (Sec. 3.3.1.1). If they are laterally unbraced, then lateral-torsional buckling (Sec. 3.3.1.2) is the additional limit state.

3.3.1.1 Nominal section strength. The bending strength of beams with a stiffened or unstiffened compression flange is based on either the initiation of yielding (Procedure I) or the inelastic reserve capacity (Procedure II) of the member. The use of the post-buckling strength for compression elements in LRFD is the same as that included in Reference 6. Section 2.2 of this Commentary provides an extensive discussion on this subject.

The experimental bases for the cold-formed stainless steel beams were examined in Reference 21, where a total of 17 tests were studied according to the effectiveness of compression flanges and webs. On the basis of the initiation of yielding, the nominal strength R_n is determined by the product of the effective section modulus and the specified minimum yield strength, i.e., $R_n = S_eF_y$. The nominal moment strength for certain sections can be calculated on the basis of the inelastic reserve capacity if the member satisfies the conditions given in Specification Section 3.3.1.1(b). The nominal section strength determined by this approach shall not exceed $1.25S_eF_y$ or that cause a maximum compression strain of C_ye_y.

The computed safety index β is equal to 3.26 with the corresponding resistance factor $\phi = 0.90$ for beams with stiffened compression flanges. This ϕ factor was determined on the basis of a load combination of $1.2D_n + 1.6L_n$ and a ratio of $D_n/L_n = 0.2$. For determining the resistance factors ϕ, the following values of P_m, V_P, M_m, V_M, F_m and V_F are presented in Reference 21 for professional, material, and fabrication factors:

$$P_m = 1.09; \qquad V_P = 0.06$$

$$M_m = 1.10; \qquad V_M = 0.10$$

$$F_m = 1.00; \qquad V_F = 0.05$$

Due to the lack of test data, the calibration of flexural members with partially stiffened compression flanges and unstiffened compression flanges was not conducted. In order to maintain the consistency of structural safety for using stainless steels, the resistance factor used for beams with partially stiffened compression flanges and unstiffened compression flanges is taken as 0.90 and 0.85, respectively, which is relatively smaller than that used for cold-formed carbon steels.

As discussed in Section 2.2 of the Commentary, stiffened and unstiffened compression elements can withstand stresses considerably in excess of their critical buckling stress without impairment of their ability to carry load. However, stresses above the buckling stress may cause minor local distortions under service load. When such local distortions at service load must be limited, the perceptible stresses, f_b, specified in Eqs. 3.3.1.1-5 to 3.3.1.1-8 of the Specification and the elastic section modulus of the full, unreduced section should be used to determine the nominal moment. The resistance factor ϕ used for determining the design flexural strength due to local distortion is taken as 1.0.

These permissible compressive stresses obtained from Reference 6 are based on the considerations that if some barely perceptible distortions at the design load are allowed, the maximum compressive stresses for stiffened and unstiffened compression elements are limited to $1.2F_{cr}$ and F_{cr}, respectively. If no local distortions at the design load are permitted, the maximum compressive stresses for stiffened and unstiffened compression elements should not exceed 0.9 F_{cr} and 0.75 F_{cr}, respectively.[12,14] In this expression, F_{cr} is the critical buckling stress.

This provision is considered to be necessary for a stainless steel structural member because of its low proportional limits and due to the fact that more attention is often given to the appearance of exposed surfaces of stainless steel used for architectural purposes.

For the design of continuous beams, Reference 62 presents numerical models for flexural behavior and deflection predictions. Simplified models for deflection calculations for the design purposes are also presented in this publication. A research project on the behavior of compact square and circular tubular beams made of austenitic stainless steel of Type 304L, reported in Reference 66, gives design criteria based on experimental and analytical studies for such members.

3.3.1.2 Lateral buckling strength. Due to the lack of test data, the calibration of beams subjected to lateral buckling was not conducted. The design expressions for laterally unbraced segments of flexural members included in this section were adopted from Reference 5 for cold-formed carbon steel members. This provision enables direct consideration of the interaction between local and overall lateral buckling by a reduction of the critical moment. This reduction is equal to the ratio of the effective section modulus to the full section modulus, S_c/S_f. In order to maintain the consistency of structural safety used for cold-formed stainless steels, the resistance factor used for the design of beams subjected to lateral buckling is taken as 0.85, which is relatively smaller than that used for cold-formed carbon steels.

For doubly symmetric I-sections and point-symmetric Z-sections, the critical moment can be calculated by using either the simplified formulas (Eqs. 3.3.1.2-2 and 3.3.1.2-3) or the theoretical value (Eq. 3.3.1.2-4). Because the Z-section has less resistance to lateral buckling, the critical moment is taken as one-half of that for I-sections.

For singly symmetric sections (x-axis is assumed to be the axis of symmetry) subjected to lateral buckling, two theoretical formulas are given in this provision. Eq. 3.3.1.2-4 of the Specification is an explicit expression for determining the critical moment for doubly symmetric I-sections and channels bending about the x-axis.[33] Eq. 3.3.1.2-5 is used to determine the theoretical torsional-flexural buckling moment for a singly symmetric section bending about the y-axis.[34]

It should be noted that the critical moments calculated from Specification Section 3.3.1.2 shall not exceed M_y, which is the maximum moment causing initial yielding at the extreme compression fiber of the full section. In order to account for the inelastic response of stainless steels, the plasticity reduction factor, E_t/E_o, is introduced in various equations for lateral buckling in the inelastic range.[35]

The predictions of this section are checked by tests performed at the Rand Afrikaans University on stainless steel Type 304 and 430 doubly symmetric beams made from cold-formed channels (Reference 67) and lipped channels. A study of the strength of lipped channel beams is presented in Reference 68.

3.3.2 Strength for Shear Only

This provision is essentially the same as that used in Reference 6. In view of the fact that the appropriate test data on shear are not available, the ϕ_v factors used in Section 3.3.2 were derived from the condition that the nominal resistance for the LRFD method is the same as the nominal resistance for the allowable stress design method.[5] Thus, the resistance factors can be computed from the following formula:

$$\phi = \frac{1.2D_n + 1.6L_n}{(\text{F.S.})(D_n + L_n)} = \frac{1.2(D_n/L_n) + 1.6}{(\text{F.S.})(D_n/L_n + 1)} \quad \text{(C-27)}$$

By using a dead-to-live-load ratio of $D_n/L_n = 1/5$ and appropriate safety factors, the ϕ_v factors can be computed from the equation. As a result, the resistance factor ϕ_v used for elastic and inelastic shear buckling is taken as 0.85, and ϕ_v is taken as 0.95 for shear yielding. It should be noted that the definition of h is the flat portion of the web instead of the clear distance between flanges.

The general equation for the shear buckling stress of flat sheet or plate elements is given in Eq. C-18 of this Commentary, simply replace σ_{cr} by τ_{cr}. The shear buckling coefficient k_v is 5.35 for simply supported conditions. Various plasticity reduction factors used for inelastic shear buckling are given by different authors. One of the simplest is that suggested by Gerard[36] who takes $\eta = G_s/G_o$; that is, the ratio of the secant shear modulus to the initial shear modulus. Other values for the plasticity reduction factor are either too involved for design use or are excessively conservative.

In Eq. 3.3.2-1 of the Specification, the design shear strength shall not exceed the product of the shear yield strength F_{yv} and the web area ht. Experimental studies of the shear buckling of Types 304, 316, and 430 stainless steel web panels are reported in References 69 and 70.

3.3.3 Strength for Combined Bending and Shear

This section is based on the interaction formulas included in the Reference 5 for the design of cold-formed carbon steel members. For cantilever beams and continuous beams, high bending stresses often combine with high shear stresses at interior supports. In the design of such members, it has been the practice to use Specification Equation 3.3.3-1 to safeguard against buckling of flat webs due to the combination of bending and shear stresses (Reference 26). The interaction formula expressed in the ratios of moments and forces is suitable for the design of beams with unreinforced webs.

In addition, an interaction equation (Eq. 3.3.3-2) is included in this Specification for beam webs with adequate transverse stiffeners. The resistance factor ϕ_b for bending is given in Section 3.3.1 and the ϕ_v value for shear is given in Section 3.3.2.

3.3.4 Web Crippling Strength

Since no research work has been done in the Cornell project to study the problem of web crippling of beams made of cold-formed stainless steels, the design provisions for single unreinforced webs and I-sections are adopted from Reference 2 for the design of cold-formed carbon steel. Due to the lack of test data, the resistance factors used in these design provisions are derived on the same basis as that used for Section 3.3.2. The yield strength for longitudinal compression should be used for the design of web crippling of beams and is given in Table A1 of this Specification.

Section 3.3.4 of the Specification provides design equations to prevent web crippling of flexural members having flat single webs (channels, Z-sections, hat sections, tubular members, roof deck, floor deck, etc.) and I-beams (made of two channels connected back-to-back, by welding two angles to a channel, or by connecting three channels). Different design equations are used for various loading conditions. As shown in Figure C9, Eqs. 3.3.4-1, 3.3.4-2, and 3.3.4-3 are used for end one-flange loading; Eqs. 3.3.4-4 and 3.3.4-5 are used for interior one-flange loading; Eqs. 3.3.4-6 and 3.3.4-7 are used for end two-flange loading; and Eqs. 3.3.4-8 and 3.3.4-9 are used for interior two-flange loading. These design equations are determined on the basis of the experimental evidence for cold-formed carbon steel sections (Reference 37). The assumed distributions of loads or reactions in beam webs are shown in Figure C10. Experimental studies of the web crippling of Types 304 and 430 stainless steel beams are reported in References 71 and 72. An explicit approach to the design of columns of any type of stainless steel is provided in Reference 78.

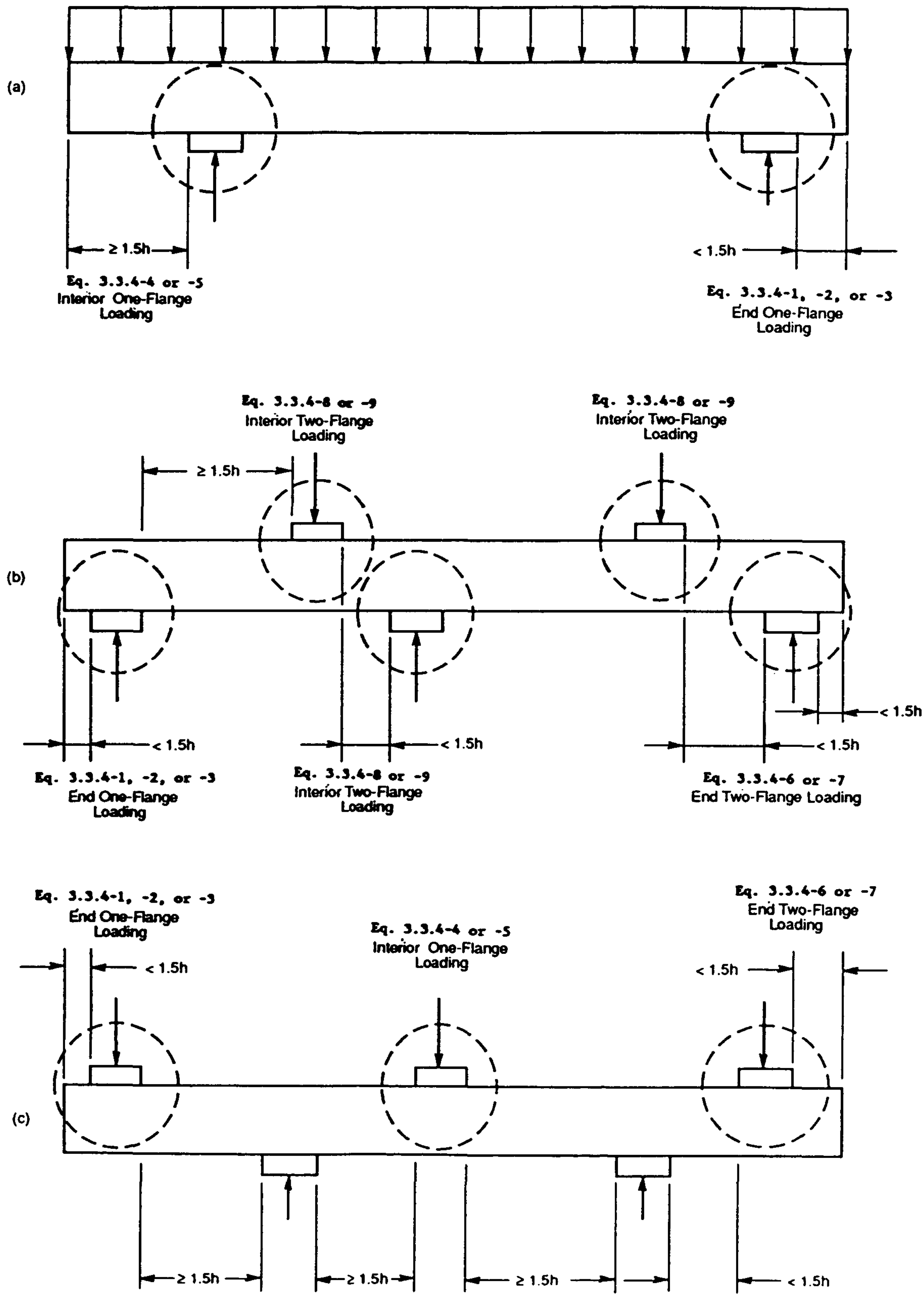


FIGURE C9. Application of Design Equations Listed in Table 2

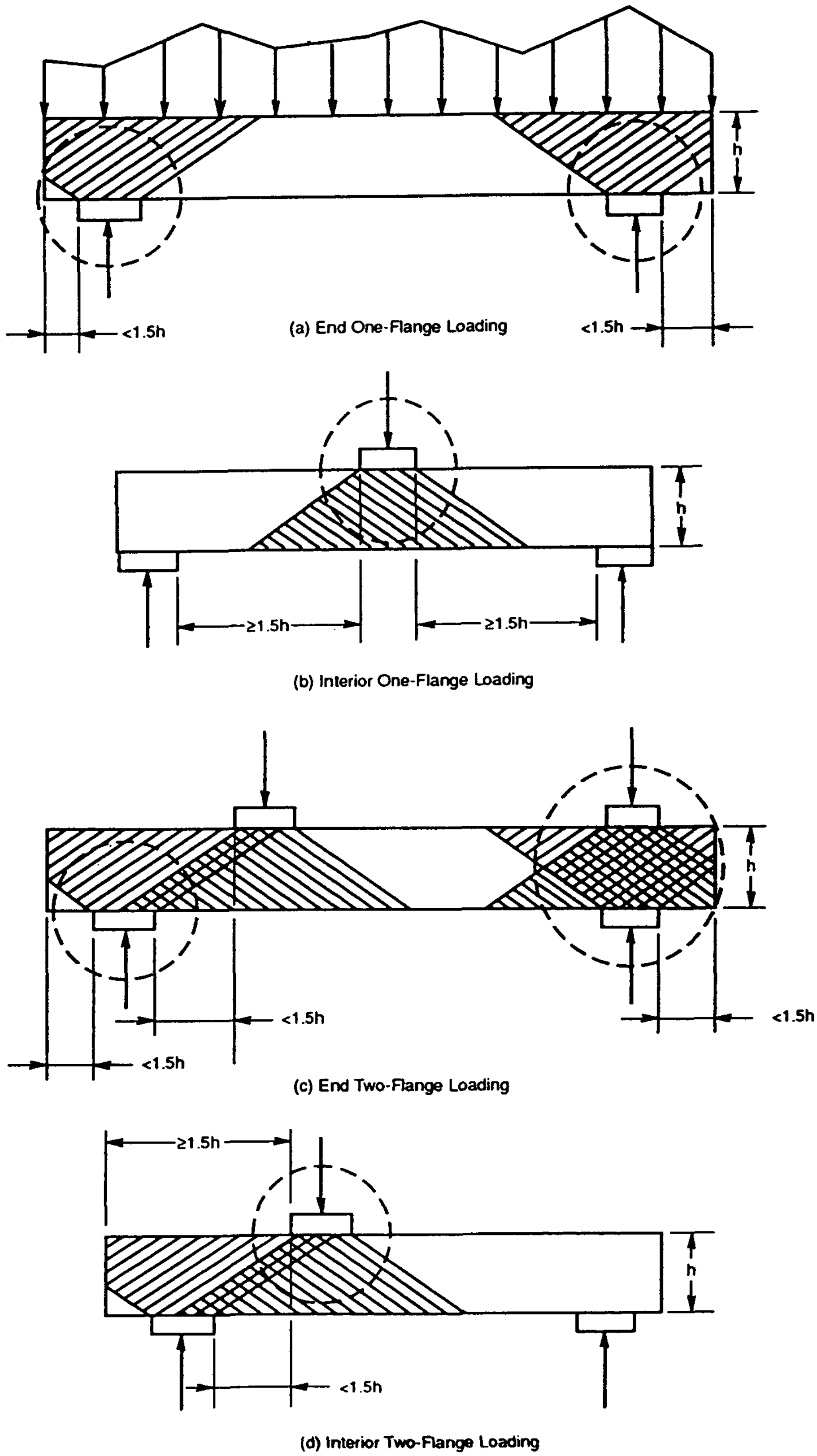


FIGURE C10. Assumed Distribution of Reaction or Load

3.3.5 Combined Bending and Web Crippling Strength

This Section contains two interaction equations for the combination of bending and web crippling. These two formulas are based on Reference 5 in conjunction with proper resistance factors used for the design of cold-formed stainless steel members.

The exception clause for Eq. 3.3.5-1 applies to the interior supports of continuous spans using decks and beams, as shown in Figure C11. Test results on cold-formed carbon steel continuous beams and decks[39] indicate that, for these types of members, the post-buckling behavior of webs at interior supports differs from the type of failure mode occurring under concentrated loads on single span beams. This post-buckling strength enables the member to redistribute the moments in continuous spans. For this reason, Eq. 3.3.5-1 is not applicable to the interaction between bending and the reaction at interior supports of continuous spans.

With regard to Eq. 3.3.5-2, it was shown in Reference 37 that when the h/t ratio of an I-beam web does not exceed $2.33/\sqrt{F_y/E_o}$ and when $\lambda < 0.673$ (i.e., web is fully effective), the bending moment has little or no effect on the web crippling load. For this reason, the allowable reaction or concentrated load can be determined by the formulas given in Specification Section 3.3.4 without reduction for the presence of bending.

3.4 Concentrically Loaded Compression Members

The available experimental data on cold-formed stainless steel concentrically loaded compression members were evaluated in Reference 21, in which column tests failed by yielding, flexural buckling, and torsional-flexural buckling. These test results were compared to the predictions based on the same mathematical models on which Reference 6 was based. For other types of column failure, the design provisions in these LRFD criteria are the same as those used in Reference 6.

A simple approach to account for the interaction of local and overall buckling is included in Specification Section 3.4. The axial strength of compression members is determined by the product of the critical stress and the effective area (Eq. 3.4-2). The critical stress, evaluated for the full section, is the stress level for which the effective area is calculated.

For channels, Z-shapes, and single angle sections with unstiffened flanges, the axial strength shall also be limited by Eq. 3.4-3 to prevent local buckling failure. When local distortions in stiffened elements of compression members under service loads must be limited, the perceptible stresses, f_b, specified in Eqs. 3.4-6 and 3.4-7 and the full area of the unreduced cross section should be used to determine the design axial strength. Commentary Section 3.3.1.1 provides discussions for this subject.

Specification Section 3.4(d) is a new provision for the possibility of a reduction in capacity due to initial sweep of the member. This additional requirement is also based on the AISI Specification for carbon steel members.[2]

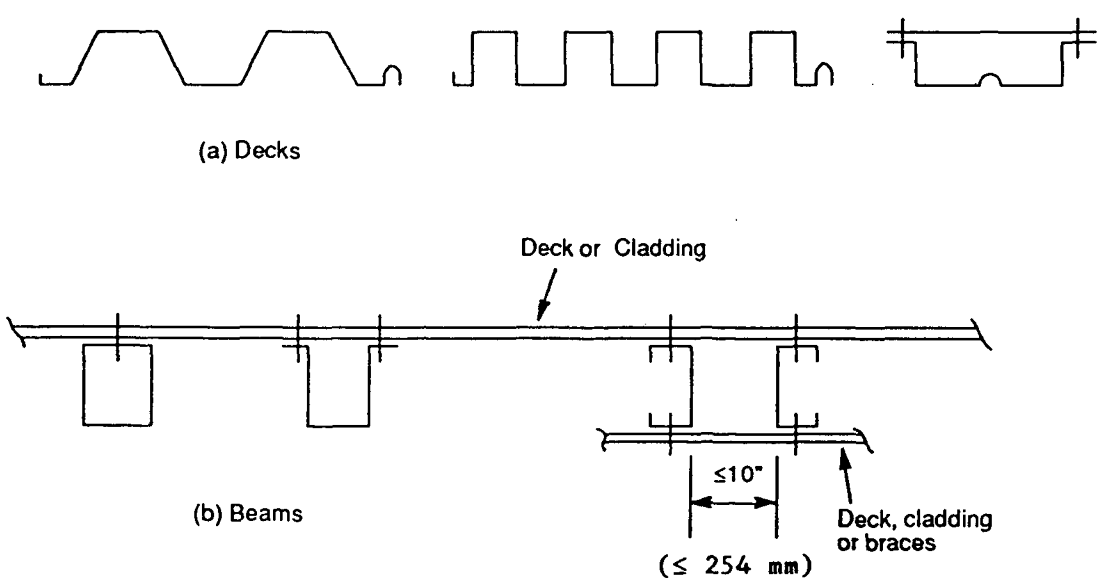


FIGURE C11. Sections Used for One Exception Clause of Section 3.3.5

3.4.1 Sections Not Subject to Torsional or Torsional-Flexural Buckling

This section deals with the flexural buckling strength of axially loaded compression members. For doubly symmetric shapes, closed cross-section shapes, or cylindrical sections axially loaded by a concentric force, these members may fail by flexural buckling and are governed by the design provisions of this section. For some open shapes, such as C-channels, hat sections, point-symmetric sections, and angles, flexural, torsional, or torsional-flexural buckling may govern, depending on the bracing conditions, cross-sectional dimensions, and the unbraced length.

In these LRFD criteria, the flexural column buckling capacity is based on the same prediction model as that employed in the formulation of Reference 6. A total of 29 flexural column buckling tests were examined. The resistance factor $\phi_c = 0.85$ was selected on the basis of a load combination of $1.2D_n + 1.6L_n$ and for the ratio of $D_n/L_n = 0.2$. The corresponding safety index is equal to 3.26. The statistical data used in this study are summarized as follows:

$$P_m = 1.19; \qquad V_P = 0.11$$

$$M_m = 1.10; \qquad V_M = 0.10$$

$$F_m = 1.00; \qquad V_F = 0.05$$

Eq. 3.4.1-1 of the Specification is simply the tangent modulus formula. This formula is generally recognized as the experimentally verified method for predicting the column buckling strength. The test results of I- and box section columns of annealed and cold-rolled stainless steels are given in References 12 and 14. The value of effective length factor, K, may be obtained from Reference 1. Modifications to the tangent modulus method were recommended in Reference 73 for square and circular tubular columns of austenitic stainless steel of Type 304L, based on experimental and analytical studies.

3.4.2 Doubly or Point-Symmetric Sections Subject to Torsional Buckling

Doubly or point-symmetric shapes may buckle torsionally depending on their cross-sectional dimensions and the unbraced length. This Section contains a design formula for the determination of the torsional buckling stress. The member should be checked for flexural and torsional buckling as specified in Sections 3.4.1 and 3.4.2 of the Specification.

3.4.3 Singly Symmetric Sections Subject to Torsional-Flexural Buckling

As indicated in Commentary Section 3.4.1, concentrically loaded columns can buckle by: (1) Bending about one of the principal planes; (2) twisting about the shear center; or (3) bending and twisting simultaneously. Case 3 is the type of torsional-flexural buckling which can occur at a load lower than the flexural buckling load.

The torsional-flexural column buckling capacity used in these criteria is based on the same prediction model as that employed in the formulation of Reference 6. A total of 45 column tests were examined for this type of failure mode. The resistance factor $\phi_c = 0.85$ was also selected on the basis of a load combination of $1.2D_n + 1.6L_n$ and for the ratio of $D_n/L_n = 0.2$. The corresponding safety index is equal to 3.17. The statistical data used in this study are summarized as follows:

$$P_m = 1.11; \qquad V_P = 0.07$$

$$M_m = 1.10; \qquad V_M = 0.10$$

$$F_m = 1.00; \qquad V_F = 0.05$$

The design formulas (Eqs. 3.4.3-1 and 3.4.3-2) included in this Section of the Specification are to be used for determining the critical stress for torsional-flexural buckling. They are adopted from the AISI Specification[2] for carbon steel members with some necessary modifications. Further experimental corroboration of the method of column design is provided in References 74 and 75 for Types 304, 409, and 430 stainless lipped-channel and hat-shaped columns.

3.4.4 Nonsymmetric Sections

For nonsymmetric open shapes, the analysis for torsional-flexural buckling becomes extremely tedious, unless its need is sufficiently frequent to warrant computerization. Section 3.4.4 of the Specification specifies that rational analysis shall be used, or tests shall be made according to Section 6, when dealing with nonsymmetric open shapes.

3.5 Combined Axial Load and Bending

This provision is adopted from Reference 16 for the design of cold-formed carbon steel members, except that tangent modulus, E_t, is used to calculate the critical buckling load. A recent study reported in Reference 29 indicates that the interaction equations (Eqs. 3.5-1, 3.5-2, and 3.5-3) are applicable to all cold-formed steel shapes.

3.6 Cylindrical Tubular Members

The design provision for cylindrical tubular members is based on Reference 6 to include the design guidelines for members subject to either bending or axial compression.

3.6.1 Bending

The resistance factor $\phi = 0.90$ used in Section 3.6.1 for bending is the same as that used in Section 3.3.1.1 for nominal section strength of beams. It should be noted that the definition of *D*, outside diameter of the cylindrical tubular members, differs from the previous Specification. The limit of D/t is the same as that used in Reference 6. The variable K_c is determined on the basis of the ratio of the proportional limit-to-yield strength. Figure C12 shows the relationships between the ratio of the nominal moment to the yield moment, M_n/M_y, and the ratio of $(E_o/F_y)(t/D)$. In Figure C12, Line 1 represents the ultimate moment governed by yielding and Line 2 corresponds to the nominal moment in the inelastic buckling range.

3.6.2 Compression

In order to maintain the consistency of structural safety for using cold-formed stainless steels, the resistance factor ϕ_c used in Section 3.6.2 for compression is taken as 0.80, which is relatively smaller than that used for concentrically loaded compression members. This provision uses the product of the effective area and the flexural buckling stress to determine the axial compression strength of cylindrical tubular members. The effective area defined in Eq. 3.6.2-2 is similar to that used in Reference 2, except that the plasticity reduction factor, E_t/E_o, is used in this Specification. The ratio A_o/A used in Eq. 3.6.2-2 reflects the influence of local buckling on the axial compression strength for concentrically loaded compression members.

3.7 Arc-and-Tangent Corrugated Sheets

This provision is the same as that used in Reference 6, except that the requirements are expressed in terms of nominal moment instead of allowable moment.

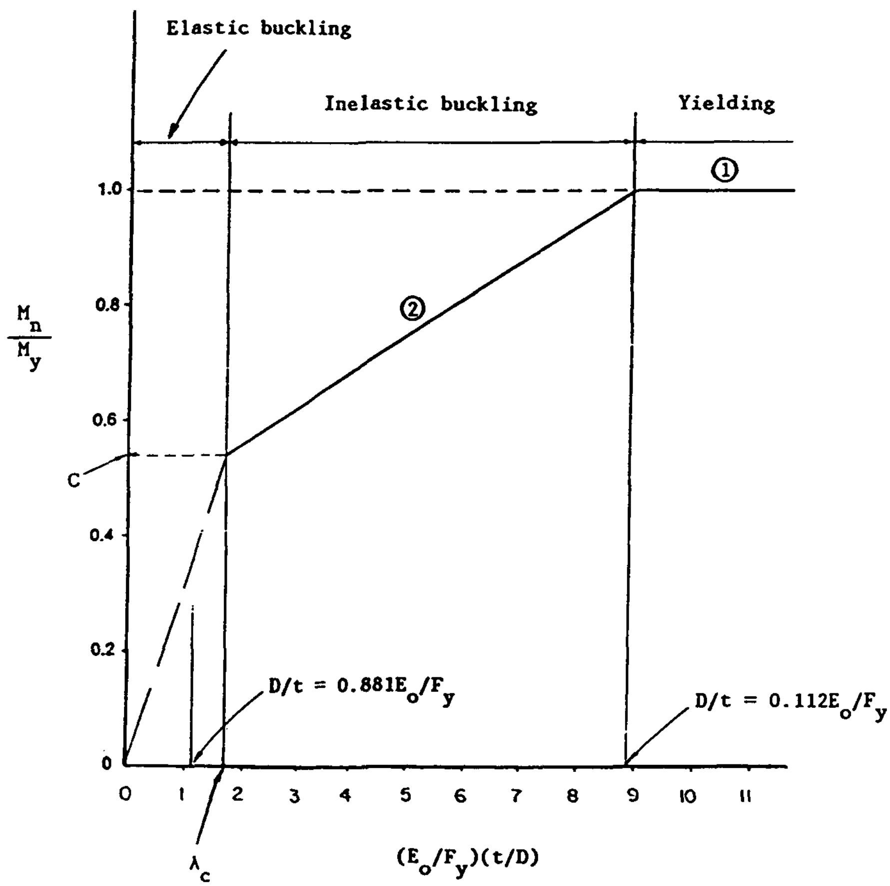


FIGURE C12. Ultimate Moment Capacity of Stainless Steel Cylindrical Tubes

Arc-and-tangent type corrugated sheets and panels are often used for roofing, siding, and curtain walls. Previous tests[39] on carbon steel sections indicated that the moment-resisting capacities of such sections were about 20% higher than those computed by elastic analysis. This high load-carrying capacity of corrugated sheets is mainly due to the plastic yielding of material and the increase of yield strength resulting from cold-forming operation. In order to recognize the substantial reserve of strength due to plastic behavior, a high nominal moment of $F_y S_f$ is permitted in Section 3.7 for arc-and-tangent corrugated sheets. The resistance factor $\phi_b = 0.90$ used in Section 3.7 for bending is the same as that used in Section 3.3.1.1 for nominal bending strength of beams.

Section 3.7 also permits the determination of load-carrying capacity of arc-and-tangent corrugated sheets by tests. For this case, the provision of Section 6.2 of the Specification should be used. In the determination of the design load, consideration must also be given to the practical limitation of deflection and permanent set of the sheets after releasing the applied load. Excessive deflection may cause leakage at end laps or loosening of end connections.

4. STRUCTURAL ASSEMBLIES

4.1 Built-Up Sections

4.1.1 I-Sections Composed of Two Channels

I-beams made by connecting two channels back to back are often used as either compression or flexural members. The provisions of Section 4.3 of this Specification are the same as those included in the 1974 Edition of the AISI Specification.[3,38]

For the I-sections to be used as compression members, the longitudinal spacing of connectors must not exceed the value of s_{max}, computed by using Eq. 4.1.1-1 of the Specification. This requirement is to prevent flexural buckling of individual channels about the axis parallel to the web at a load smaller than that at which the entire I-section would buckle. This provision is based on the requirement that the slenderness ratio of an individual channel between connectors, s_{max}/r_{cy}, not be greater than one-half of the pertinent slenderness ratio, L/r_I, of the entire I-section to account for one of the connectors becoming loose or ineffective.[26,40]

Even though Section 4.1 of the Specification refers only to I-sections, Eq. 4.1.1-1 can also be used for determining the maximum spacing of welds for box-shaped compression members made by connecting two channels tip to tip. In this case, r_I is the smaller of the two radii of gyration of the box-shaped section.

For the I-sections to be used as flexural members, the longitudinal spacing of connectors is limited by Eqs. 4.1.1-2 and 4.1.1-3 of the Specification. The first requirement (Eq. 4.1.1-2) is an arbitrarily selected limit to prevent any possible excessive distortion of the top flange between connectors. The second requirement (Eq. 4.1.1-3) is based on the strength and arrangement of connectors and the intensity of the load acting on the beam. Figure C13 shows that when the transverse load is applied to the I-beam, the load Q to be carried by each channel (i.e., half of the total beam load over the length, s) actually acts in the plane of the web. Because the load Q does not pass through the shear center of the channel section, each channel tends to rotate about its own shear center and to separate along the top. The twisting moment Qm is resisted by the torque, $T_s g$, provided by the connectors. The location of shear center is given by Eq. 4.1.1-4 of the Specification for channels having unstiffened and stiffened flanges, which is determined on the basis of Reference 41.

4.1.2 Spacing of Connections in Compression Elements

The requirements for the spacing of connections in compression elements are the same as those used in the 1974 Edition of the AISI Specification,[3] except that some slight changes have been made in case (c) of Specification Section 4.1.2.

When compression elements are joined to other parts of built-up members by intermittent connections, these connections must be closely spaced to develop the required strength of the connected element. Figure C14 shows a box-shaped beam made by connecting a flat sheet to an inverted hat section. If the connectors are appropriately placed, this flat sheet will act as a stiffened compression element with a width, w, equal to the distance between rows of connectors, and the sectional properties can be calculated accordingly. This is the intent of the provisions in Section 4.1.2.

Section 4.1.2(a) of the Specification requires that the necessary shear strength be provided by the same standard structural design procedure that is used in calculating flange connections in bolted or welded plate girders or similar structures.

Section 4.1.2(b) of the Specification ensures that the part of the flat sheet between two adjacent connectors will not buckle as a column (see Figure C14) at a stress less than $1.85f$, where f is the design stress of the connected compression element. The formula of Section 4.1.2(b) is directly obtained from the Euler formula, $\sigma_e = \pi^2 E_o/(KL/r)^2$, by substituting $\sigma_e = 1.85f$, $E_t = E_o$, $K = 0.6$, $L = s$, and $r = t/\sqrt{12}$.

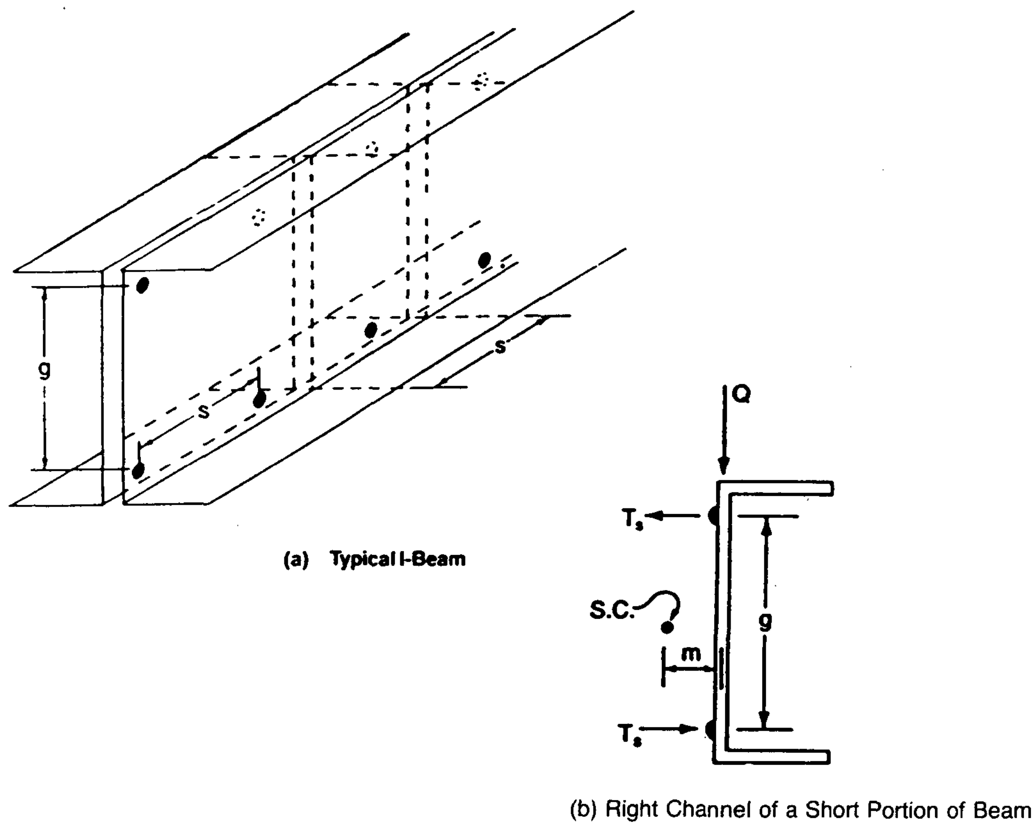


FIGURE C13. I-Beam Composed of Two Channels

Section 4.1.2(c) ensures satisfactory spacing to make a row of connections act as a continuous line of stiffening for the flat sheet under most conditions. This requirement is similar to that in Reference 2, except that a slight modification has been made to account for the inelastic behavior of stainless steel. In this case, the initial modulus of elasticity, E_o, should be selected from Tables A4 and A5 of the Specification instead of using E_o = 29,500 ksi (203 $\times$ 10^3 MPa) given in Reference 2. References 40 and 42 provide further information for this topic.

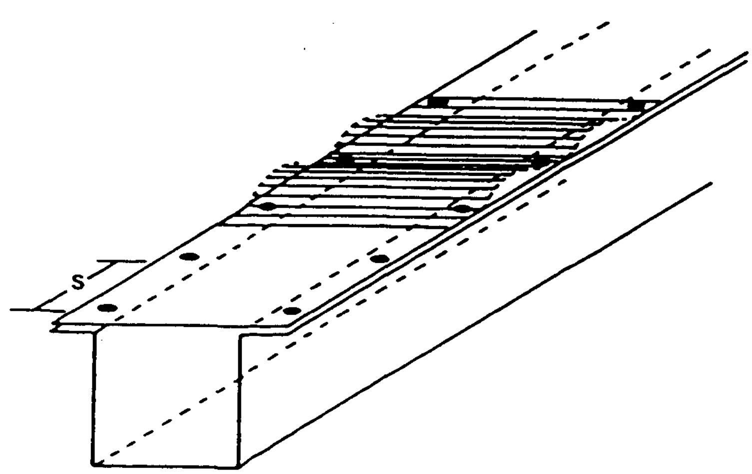


FIGURE C14. Spacing of Connections in Compression Element

4.2 Mixed Systems

When cold-formed stainless steel members are used in conjunction with other construction materials, the design requirements of other material specifications must also be satisfied.

4.3 Lateral Bracing

Bracing design requirements have been expanded in the current Specification. A new provision regarding bracing for symmetrical beams and columns is included in Section 4.3.1.

4.3.1 Symmetrical Beams and Columns

The Specification does not provide any generally accepted techniques for determining the required strength and stiffness for discrete braces in cold-formed stainless steel construction. Design engineers are encouraged to seek out the related references[43–47] and to obtain guidance for design of a brace or brace system for cold-formed stainless steel structures.

4.3.2 Channel-Section and Z-Section Beams

Channels and Z-sections used as beams to support transverse loads applied in the plane of the web may twist and deflect laterally unless adequate lateral supports are provided. Section 4.3.2 of the Specification includes two subsections. The first subsection (Section 4.3.2.1) is a new provision that deals with the bracing requirement when one flange of the beam is connected to deck or sheathing material. The second subsection (Section 4.3.2.2) covers the requirements for the spacing and design of braces, when neither flange of the beam is braced by deck or sheathing material. These design provisions are based on the AISI Specification for the design of cold-formed carbon steel structural members.[2]

4.3.2.1 Bracing when one flange is connected. When channel and Z-sections are used in roofs and walls to directly support attached covering material, the latter provides some measure of lateral support to the connected flange of the beam. As a result, forces are generated in the plane of the covering material by the tendency to lateral movement and/or twist of the beam. These accumulated forces must be transferred into a sufficiently stiff part of the framing system. Further information on this topic can be found in Reference 42. The provision of Section 4.3.2.1 for the design of braces calls for special tests in accordance with Section 6 of the Specification to insure the adequacy of the type and spacing of braces to be used for such beams when one flange is connected to covering material.

4.3.2.2 Neither flange connected to sheathing. These provisions are similar to those included in the 1974 Edition of the AISI Specification,[3] except that some editorial changes have been made in the current Specification. When neither flange is braced by deck or sheathing material, discrete bracing must be provided. Section 4.3.2.2 specifies the spacing of such braces, applicable to both channel and Z-beams, and the forces for which these braces must be designed. A detailed discussion of the background for these provisions is contained in Reference 40.

An exception added in Specification Section 4.3.2.2 permits omission of discrete braces when all loads and reactions on a beam are transmitted through members that frame into the section in such a manner as to effectively restrain the member rotation and lateral displacement. Frequently this occurs in the end walls of metal buildings.

4.3.3 Laterally Unbraced Box Beams

This provision is the same as that used in the 1974 Edition of the AISI Specification,[3] except that the requirement is expressed in a dimensionless unit. A brief discussion on lateral buckling of box beams can be found in Reference 42.

5. CONNECTIONS AND JOINTS

5.1 General Provisions

This Specification contains provisions for welded and bolted connections only. Other means of connection, e.g., rivets, screws, and special devices are proprietary devices for which the information on connection strength must be obtained for manufacturers or from tests carried out by or for the user. Guidelines provided in Specification Section 6 are to be used in these tests.

The design provisions contained in this Section are primarily based on the reevaluation of experimental evidences obtained from the test program at Cornell University for welded and bolted connections using stainless steels.[48,49] In addition to Cornell data, Reference 50 provides additional test results for the study of welded connections.

The resistance factors used for welded and bolted connections were derived for a target reliability index $\beta_o = 4.0$, and the statistical data are summarized in the subsequent sections.

Ease of fabrication and erection are factors to be considered in the design of all joints and splices. Attention should be given to: the clearances necessary for safe erection, the clearances needed for tightening fasteners, the need for access for welding, the requirements of welding procedures, and the effects of angular and length tolerances on fit-up. Attention should also be given to the requirements for subsequent inspection and maintenance.

5.2 Welded Connections

The design provisions for welded connections are developed on the basis of the research findings reported in References 48, 49, 50 and the AISI Specification for cold-formed carbon steel.[2] This Specification provides design requirements for butt welds, fillet welds, and resistance welds. Because Type 430 ferritic stainless steel is susceptible to brittle martensite formation after welding, weld connections should not be used for this type of material.

5.2.1 Groove Welds in Butt Joints

For groove welds in butt joints, a total of 43 test specimens have been evaluated to determine resistance factors. The design equation for the nominal strength in tension and compression of a groove weld in a butt joint is based on the tensile strength of the annealed base metal, F_{ua}. For cold-formed carbon steel, the nominal strength is determined on the basis of the yield strength of the base metal, F_y (Reference 5). Due to the difference in design provisions, the ϕ factor used for cold-formed carbon steels is larger than that used in this Specification for stainless steels. The design provisions and the ϕ factors for shear on the effective area are the same as that used in the AISI LRFD criteria for cold-formed carbon steel members (Reference 5).

The resistance factor, ϕ, of 0.6 used for determining the factored nominal ultimate strength in tension of butt welds is based on the results reported in Reference 21. Based on the calibration of this design provision, the computed safety index, β, for $D_n/L_n = 0.2$ is equal to 4.13. The statistical data used for deriving the ϕ factor are given as follows:

$$P_m = 1.11; \quad V_P = 0.08$$
$$M_m = 1.10; \quad V_M = 0.05$$
$$F_m = 1.00; \quad V_F = 0.15$$

For detailed information on determining the ϕ factor, see Reference 21.

5.2.2 Fillet Welds

For fillet welded connections, Reference 49 indicates that the design should be based on: (1) fracture of the annealed base metal; and (2) fracture of the weld metal. Because transverse fillet welds are stressed more uniformly than longitudinal fillet welds, the capacity of fillet welds subjected to transverse loading was found to be higher than that for longitudinal loading.

In Section 5.2.2 of the Specification, separate provisions are included for fillet welds subjected to longitudinal direction and transverse direction loadings. For longitudinal fillet welds, the ϕ factor of 0.55 is used for preventing the failures of connected plate and weld metal. For transverse fillet welds, the ϕ factors of 0.55 and 0.65 are used separately for preventing the failure of connected plate and weld metal, respectively. These ϕ factors are obtained from the calibrations of design provisions reported in Reference 21. The statistical data used for deriving these ϕ factors are given in Table C8.

The tensile strengths of the weld metal listed in Table A15 of the Specifications are based on Refer-

TABLE C8. Computed Safety Indices, β, for Cold-Formed Stainless Steels Using Fillet Welds ($D_n/L_n = 0.2$)

Case	No. of tests	ϕ	M_m	V_M	F_m	V_F	P_m	V_P	β
Longitudinal loading									
1	10	0.55	1.10	0.05	1.0	0.15	1.083	0.131	4.09
2	10	0.55	1.10	0.05	1.0	0.15	1.058	0.126	4.04
Transverse loading									
3	10	0.55	1.10	0.05	1.0	0.15	1.027	0.088	4.14
4	10	0.65	1.10	0.05	1.0	0.15	1.207	0.089	4.11

Note: Case 1. = plate failure of longitudinal fillet welds.
Case 2. = weld failure of longitudinal fillet welds.
Case 3. = plate failure of transverse fillet welds.
Case 4. = weld failure of transverse fillet welds.

ences 51 and 52. Table C9 in the Commentary includes supplementary information on elongation and heat treatment for weld metal. The tensile strengths of annealed base metals listed in Table A16 for Types 201, 301, 304, and 316 stainless steels are based on Reference 11. For Types 409, 430, and 439, the tensile strengths of annealed base metals listed in Table A16 are adapted from References 8 and 9.

5.2.3 *Resistance Welds*

Due to the lack of test data for resistance welds, the calibration for this subject has not been conducted. The design provisions of this Specification for resistance welds are the same as those used in Reference 6, except that the nominal shear strengths are used in this Specification instead of the allowable shear strengths. In order to maintain the consistency of structural safety for cold-formed stainless steels, a ϕ factor of 0.60 used for the design of resistance welds was slightly smaller than that used for cold-formed carbon steels.[5]

Table 3 of Section 5.2.3 includes nominal shear strengths for annealed, 1/4 and 1/2 hard temper materials. These data were obtained from the AWS "Recommended Practices of Resistance Welding."[53] The nominal shear strengths are determined by multiplying a safety factor of 2.50 to the allowable shear strengths of arc spot welding which are listed in Reference 3. Although the AWS results were based on the testing of specimens using Types 301, 304, and 316 stainless steels, these values can be used conservatively for Type 201.

In Table 4 of Specifications Section 5.2.3 on pulsation welding, the nominal shear strengths are included only for 1/4 and 1/2 hard temper stainless steels. The exclusion of the design data for annealed stainless steel was due to the fact that heavy sheets of annealed materials thicker than about 0.125 in. (3.18 mm) would not be available from most steel producers. These nominal shear strengths for pulsation welding are also determined on the basis of a safety factor of 2.50 and the allowable shear strengths given in Reference 53.

5.3 Bolted Connections

This provision contains some general requirements for bolt installation and maximum sizes of holes. Even though no specific requirements are given in this section for installation, bolts should be properly tightened according to good practice used in building construction.

The design requirements included in this section deal with minimum spacing and edge distance, tension in connected parts, bearing in bolted connections, and

TABLE C9. AWS Requirements for Mechanical Property of All-Weld Metal[51,52]

AWS classification	Elongation in 2 in. (50.8 mm) gage length min., %	Heat Treatment
E209	15	None
E219	15	None
E240	15	None
E307	30	None
E308	35	None
E308H	35	None
E308L	35	None
E308Mo	35	None
E308MoL	35	None
E309	30	None
E309L	30	None
E309Cb	30	None
E309Mo	30	None
E310	30	None
E310H	10	None
E310Cb	25	None
E310Mo	30	None
E312	22	None
E316	30	None
E316H	30	None
E316L	30	None
E317	30	None
E317L	30	None
E318	25	None
E320	30	None
E320LR	30	None
E330	25	None
E330H	10	None
E347	30	None
E349	25	None
E410	20	a
E410NiMo	15	b
E430	20	c
E502	20	a
E505	20	a
E630	7	d
E16-8-2	35	None
E7Cr	20	a

a. Specimen shall be heated to between 1550° and 1600°F (840° and 870°C), held for 2 hr, furnace-cooled at rate not exceeding 100°F (38°C) per hr. to 1100°F (595°C), and air-cooled to ambient.
b. Specimen shall be heated to between 1100° and 1150°F (595° and 620°C), held for 1 hr., and air-cooled to ambient.
c. Specimen shall be heated to between 1400° and 1450°F (760° and 790°C), held for 2 hr., furnace-cooled at rate not exceeding 100°F (38°C) per hr. to 1100°F (595°C), and air-cooled to ambient.
d. Specimen shall be heated to between 1870° and 1925°F (1025° and 1050°C), held for 1 hr., air-cooled to at least 60°F (15°C), and then precipitation-hardened at 1135° to 1165°F (610° to 630°C), held for 4 hr., and air-cooled to ambient.

shear and tension in bolts. All ϕ factors were computed from the statistical data given in Reference 21 and a target safety index of $\beta_o = 4.0$.

5.3.1 Spacing and Edge Distance

The minimum edge distance of each individual connected part, e_{min}, is determined by using the shear strength of the connected part in the longitudinal direction. The design equation is based on the following basic formula established from the test results:[49]

$$e_{reqd} = P/F_u t \qquad \text{(C-28)}$$

in which: e_{reqd} = the required minimum edge distance to prevent shear failure of the connected part for the force, P, transmitted by one bolt; and t = the thickness of the thinnest connected part. The resistance factor $\phi = 0.7$ used in this provision is based on the results of calibration of design provisions included in Reference 21. The statistical data used to determine the ϕ factor are summarized as follows:

$$P_m = 1.06; \qquad V_P = 0.05$$
$$M_m = 1.10; \qquad V_M = 0.05$$
$$F_m = 1.00; \qquad V_F = 0.05$$

For the selected value of $\phi = 0.7$, the computed safety index is equal to 4.10.

The design requirements for bolted connections with standard, over-sized and slotted holes are also included in this provision.

5.3.2 Tension in Connected Part

In these criteria, the formulas used for computing the tension stress on the net section of connected parts are based on the reevaluation of test results presented in Reference 49. It was found that by using Eq. 5.3.2-3 for double shear connections and Eq. 5.3.2-4 for single shear connections, good correlations can be achieved between tested and predicted values against tension failure on the net section of connected parts.

In Eqs. 5.3.2-2 and 5.3.2-3, F_u is taken as the tensile strength in the longitudinal direction as given in Table A16. The resistance factor used to determine the design tensile strength on net section is 0.70 for double shear connections and single shear connections. This ϕ factor was obtained from the results of calibration reported in Reference 21. The statistical data used for this case are given as follows:

$$P_m = 1.10; \qquad V_P = 0.10$$
$$M_m = 1.10; \qquad V_M = 0.05$$
$$F_m = 1.00; \qquad V_F = 0.05$$

For the selected value of $\phi = 0.70$, the value of $\beta = 4.04$.

No design equations are given in this section for connections without washers due to lack of test data.

5.3.3 Bearing

In this Specification, the nominal bearing stress was determined on the basis of the tensile strength of the connected part in the longitudinal direction. Based on the test results presented in Reference 49, different nominal bearing stresses are specified for single and double shear connections. The resistance factor used in this provision is taken as 0.65, which is based on the study reported in Reference 21. The statistical data obtained from Reference 21 are given as follows:

$$P_m = 1.02; \qquad V_P = 0.08$$
$$M_m = 1.10; \qquad V_M = 0.05$$
$$F_m = 1.00; \qquad V_F = 0.05$$

For the resistance factor ϕ of 0.65, the corresponding safety index is equal to 4.14.

5.3.4 Shear and Tension in Stainless Steel Bolts

In this Specification, the nominal shear stresses for stainless steel bolts are given in Table 6 of the Specification. This table was prepared on the basis of References 10, 48, 54, and 63. The nominal shear stress for bolts with no threads in the shear plane was taken as 60% of the minimum tensile strength. When threading is included in the shear plane, 75% of the computed nominal shear stress (i.e., 45% of the minimum tensile strength) is used for bolts with diameters larger than and equal to $\frac{1}{2}$ in. (12.7 mm). This is due to the fact that the actual shear stress in bolts is to be calculated on the basis of the gross cross-sectional area of nominal area, and that the ratios of stress area-to-nominal-area range from 0.65 to 0.76 for diameters of bolts varying from $\frac{1}{4}$ to $\frac{3}{4}$ in. (6.35 mm to 19.05 mm). Because the ratio of stress area-to-nominal-area is small for bolts, a 10% reduction of the nominal shear stress is used for bolts with diameters less than $\frac{1}{2}$ in. (12.7 mm). This practice is comparable to that for high-strength carbon steel structural bolts. For the bolts not listed in Table 6, the nominal shear stresses can be determined in the same manner. This ϕ factor of 0.65 used for determining the design shear strength and the ϕ factor of 0.75 used for the design tensile strength are adopted from the design of cold-formed carbon steel bolted connections.[5]

In Table 6, the nominal tensile stresses for stainless steel bolts with diameters larger than and equal to $\frac{1}{2}$ in. (12.7 mm) are based on 75% of the minimum tensile strength. A 10% reduction of the nominal tensile stress is used for bolts with diameters less than $\frac{1}{2}$ in. (12.7 mm).

When bolts are subjected to a combination of shear and tension, the reduced nominal tension stresses, F'_{nt}, are given in Eqs. 5.3.4-2 and 5.3.4-3. The design equations for determining F'_{nt} are derived from the following equation:

$$F'_{nt} = 1.25\,F_{nt} - Af_v \qquad \text{(C-29)}$$

where:

F'_{nt} = reduced nominal tensile stress for bolts subjected to a combination of shear and tension;
F_{nt} = nominal tensile stress for bolts subject only to tension;
A = 2.4 for threads not excluded from shear planes;
= 1.9 for threads excluded from shear planes; and
f_v = shear stress in bolt.

The ϕ factor of 0.75 used for bolts subjected to a combination of shear and tension was adopted from Reference 5 for the design of cold-formed carbon steel bolted connections.

6. TESTS

This section covers the requirements of tests to determine material properties and structural performance for cold-formed stainless steel structural members or assemblies. These provisions do not apply to the panels used as shear diaphragms. A general discussion of structural diaphragms is given in References 40 and 55.

6.1 Determination of Stress-Strain Relationships

This provision is the same as that included in the 1974 Edition of the AISI Specification,[3] except that the updated ASTM specifications are used in the current Specification. This section is included in case the material does not correspond to the mechanical properties given in Section 1.3, or for obtaining detailed mechanical properties for special cases when necessary. Discussions of the statistical approaches are contained in References 12 and 14.

6.2 Tests for Determining Structural Performance

This Section contains provisions for proof of structural adequacy by load tests. It is restricted to cases where calculation of safe load-carrying capacity or deflection cannot be made in accordance with the provisions of this Specification.

Many cold-formed stainless steel structural applications have different composition or configuration which is not covered by the provisions of this Specification; their performance and adequacy therefore cannot be demonstrated by the Specification. Tests according to Section 6 are the only means of determining the structural adequacy.

The determination of load-carrying capacities of the tested elements, assemblies, connections, or members is based on the same basis for the LRFD criteria. The correction factor C_p is used in the determination of the ϕ factor to account for the influence due to the small number of tests (Reference 60). It should be noted that when the number of tests is large enough, the effect of correction factor is negligible.

Provision 3 applies when the strength of the test specimen is greater than the specified strength. This requires that the test results must be reduced in the ratio of the actual strength to the specified minimum strength to obtain the load-carrying capacity. The provision is self-explanatory regarding similar corrections for tensile strength or sheet thickness.

6.3 Tests for Determining Mechanical Properties of Full Sections

Methods for utilizing the effects of cold work are incorporated in Section 1.5.4 of the Specification. In that section, it is specified that as-formed mechanical properties, in particular the yield strength, can be determined by full-section tests under some limitations. This Specification section spells out in considerable detail the types and methods of these tests, and their number as required for use in connection with Specification Section 1.5.4. For details of some testing procedures which have been used for such purpose, see References 56 and 57.

COMMENTARY APPENDICES

APPENDIX A: DESIGN TABLES AND FIGURES

In this Specification, all the design tables and figures are included in Appendix A. These technical data provide the basic design information on material properties for Types 201, 301, 304, and 316 (annealed, 1/16, 1/4, and 1/2 Hard) austenitic stainless steels and Types 409, 430, and 439 (annealed) ferritic stainless steels. In addition, design tables for tensile strengths of weld metals and base metals for welded connections are also included.

Table A1 of the Specification lists the yield strengths for seven types of stainless steels according to the direction and type of stress. The yield strengths of Types 201, 301, 304, and 316 austenitic stainless steels for transverse tension are based on the values specified in ASTM A666-84.[11] Other yield strengths for longitudinal tension, transverse compression, and longitudinal compression are derived from the tested yield strengths given in Tables C3, C4, and C5 of the Commentary.

For Types 409, 430, and 439 stainless steel, the yield strengths have been adjusted for different types of stresses. These revised values are needed in order to reflect the tested data. The ASTM specified yield strength for Types 409, 430, and 439 ferritic stainless steels is 30 ksi (205 MPa). The tested yield strengths of Types 409, 430, and 439 stainless steels are given in Table C6 of the Commentary.

The shear yield strengths listed in Table A1 are calculated as 57.7 percent of the average value of longitudinal tension, transverse tension, transverse compression and longitudinal compression. The relationship between tension, compression, and shear stress-strain curves is discussed in Reference 12.

Tables A2 through A14 and Figures A1 through A12 provides the design data on secant modulus, initial modulus of elasticity, tangent modulus, and plasticity reduction factors for Types 201, 301, 304, 316, 409, 430, and 439 stainless steels used in this Specification. The aforementioned mechanical properties for Types 201, 301, 304, and 316 are the same as those used in Reference 6. For Types 409, 430, and 439, the mechanical properties are based on References 13 and 15.

For the design of welded connections, the tensile strengths of weld metals listed in Table A15 are based on References 51 and 52. The tensile strengths of base metals listed in Table A16 for Types 201, 301, 304, and 316 (annealed, 1/16, 1/4, and 1/2 Hard) are adapted from ASTM A666-84.[11] In the same table, the tensile strengths of base metals for Types 409, 430, and 439 (annealed) are based on ASTM 176-85a and ASTM A240-86.[8,9]

Table A17 lists the ratios of the effective proportional limit-to-yield strength. The values of F_{pr}/F_y for Types 201, 301, 304, and 316 stainless steels are the same as those used in the proposed ASCE ASD Specification. For Types 409, 430, and 439, the ratios of F_{pr}/F_y are determined on the basis of References 13 and 15.

APPENDIX B: MODIFIED RAMBERG-OSGOOD EQUATION

The stress-strain relationships for annealed and cold-rolled stainless steels are nonlinear and anisotropic. This leads to a relatively difficult design because the stress-strain curves can not be represented by a linear function. Thus, it is desirable to have an analytical expression for the study and design of stainless steel structural elements and members.

For the purpose of simplicity, the Ramberg-Osgood formula may be used to represent the stress-strain relationship. The original expression suggested by Ramberg and Osgood[58] is given as Eq. CB-1.

$$\varepsilon = \frac{\sigma}{E} + K\left(\frac{\sigma}{E}\right)^n \qquad \text{(CB-1)}$$

where:

ε = normal strain;
σ = normal stress; and
E = modulus of elasticity.

K and n are constants which are evaluated through two secant yield strengths at slopes of $0.7E$ and $0.85E$, respectively.

Based on the Ramberg-Osgood equation as given in Eq. CB-1, it can be shown that

$$K = \frac{\varepsilon_1}{(\sigma_1/E)^n} \qquad \text{(CB-2)}$$

or:

$$K = \frac{\varepsilon_2}{(\sigma_2/E)^n} \qquad \text{(CB-3)}$$

and

$$n = \frac{\log(\varepsilon_2/\varepsilon_1)}{\log(\sigma_2/\sigma_1)} \qquad \text{(CB-4)}$$

where: σ_1 and σ_2 = the specified yield strengths; and ε_1 and ε_2 = the specified strains. The modulus of elasticity, E, is regarded constant and equal to the initial value, E_o.

The Ramberg-Osgood formula was modified by Hill[59] in 1944 by using two offset yield strengths rather than the secant yield strengths because the former are commonly used. Hill indicated that the yield strength determined at the 0.2% offset strain, i.e., F_y, may be used for determining the constant K. Thus, the constant K can be expressed as

$$K = \frac{0.002}{(F_y/E_o)^n} \qquad \text{(CB-5)}$$

Consequently, Eq. CB-1 can be written as

$$\varepsilon = \frac{\sigma}{E_o} + 0.002\left(\frac{\sigma}{F_y}\right)^n \qquad \text{(CB-6)}$$

in which:

$$n = \frac{\log(0.002/\varepsilon_1)}{\log(F_y/\sigma_1)} \qquad \text{(CB-7)}$$

Although the 0.2% offset yield strength in Eq. CB-7 seemed to be a common, reasonable choice, the remaining set of the offset stress and strain (σ_1 and ε_1) has not been uniquely decided. For cold-formed stainless steels, the constants of n have been determined on the basis of the 0.01% offset strength because this value is well defined as the proportional limit of the material property. The n values given in Table B of the Specification are calculated by Eq. CB-7 on the basis of 0.2% and 0.01% offset stress and strain and may be used in the modified Ramberg-Osgood equation for annealed and cold-rolled stainless steels.

For the design of cold-formed stainless steel structural members, the secant moduli, tangent moduli, and plasticity reduction factors are given in the Design Tables and Figures of the Specification. These tables and figures were prepared on the basis of the actual stress-strain curves obtained from the test data for stainless steels. In this Specification, these design values can be alternatively determined by using analytical expressions for the purpose of simplicity. Based on the Modified Ramberg-Osgood equation with a proper constant of n, it appears that Eqs. B-1 through B-5 of the Specification may be used for determining the secant modulus (E_s), tangent modulus (E_t), and the plasticity reduction factors (η).

A comparison between the analytical and tabulated values of the secant and tangent moduli in longitudinal compression for Types 304 and 301 stainless steels has been performed and reported in Reference 21. It indicated that the differences between the computed and tabulated values are small for most cases. The largest difference occurs at the "knee" of the stress-strain curve. Because iterative processes are often used for the design of cold-formed stainless steel members, it would be convenient to use Eqs. B-1 through B-5 of the Specification in the computer programs for the design of such members.

APPENDIX C: STIFFENERS

Design requirements for attached transverse stiffeners and for intermediate stiffeners are given in Appendix C of this Specification. Due to the lack of test results of cold-formed stainless steel members with stiffeners, the provisions are adopted from Reference 5. Eq. C-1 of Appendix C serves to prevent end crushing of the transverse stiffeners, while Eq. C-5 is to prevent column-type buckling for the web-stiffeners. The equations for computing the effective areas (A_b and A_c) and the effective width (b_1 and b_2) were determined on the basis of Reference 32. The resistance factor ϕ_c = 0.85 was adopted from Reference 5 for the design of cold-formed carbon steel structural members.

The equations for determining the minimum required moment of inertia Eq. C-8 and the minimum required gross area Eq. C-9 of attached intermediate stiffeners are based on the studies summarized in Reference 32. In this reference, test data show that even though the shear stress is based on the buckling strength of web elements, rather than on tension field action, it is still necessary to provide the required moment of inertia and gross area of intermediate stiffeners. This is because the flanges of cold-formed steel beams often are quite flexible, as compared with the flanges of hot-rolled shapes and plate girders. In Eq. C-8, the minimum value of $(h/50)^4$ was selected from the AISC Specification.[1] The resistance factors for the design of shear stiffeners were the same as that used for the design of shear strength of beams (Section 3.3.2).

For rolled-in transverse stiffeners, the required dimensions and the allowable loads should be determined by special tests.

APPENDIX D: ALLOWABLE STRESS DESIGN (ASD)

Since 1968, the design of cold-formed stainless steel structural members has been based on the allowable stress design provisions specified in the AISI Specification.[3] The ASD method described in Appendix D of this Standard may be used as an alternate to the LRFD method.

In Table D, the safety factors to be used for the design of cold-formed stainless steel structural members and connections are based on Table C9 of Reference 6.

REFERENCES

[1] *Specification for the design, fabrication and erection of structural steel for buildings.* (1978). American Institute of Steel Construction, Nov. 1.

[2] *Specification for the design of cold-formed steel structural members* with commentary. (1986). American Iron and Steel Institute.

[3] *Specification for the design of cold-formed stainless steel structural members.* (1974). American Iron and Steel Institute.

[4] *Load and resistance factor design specification for structural steel buildings.* (1986). American Institute of Steel Construction.

[5] Hsiao, L. E., Yu, W. W., and Galambos, T. V. (1988). "Load and resistance factor design of cold-formed steel: Load and resistance factor design specification for cold-formed steel structural members with commentary." *Tenth Progress Report,* University of Missouri-Rolla, Feb.

[6] Lin, S. H., Yu, W. W., and Galambos, T. V. (1988). "Design of cold-formed stainless steel structural members—Proposed allowable stress design specification with commentary." *Third Progress Report,* University of Missouri-Rolla, Jan.

[7] "Standard methods and definitions for mechanical testing of steel products." *ASTM Designation: A 370-77,* American Society for Testing and Materials.

[8] "Standard specification for stainless and heat-resisting chromium steel plate, sheet, and strip." *ASTM Designation: A 176-85a,* American Society for Testing and Materials.

[9] "Standard specification for heat-resisting chromium and chromium-nickel steel plate, sheet, and strip for pressure vessels." *ASTM Designation: A 240-86,* American Society for Testing and Materials.

[10] "Standard specification for stainless and heat-resisting steel bars and shapes." *ASTM Designation: A 276-85a,* American Society for Testing and Materials.

[11] "Standard specification for austenitic stainless steel, sheet, strip, plate, and flat bar for structural applications." *ASTM Designation: A 666-84,* American Society for Testing and Materials.

[12] Johnson, A. L. (1967). "The structural performance of austenitic stainless steel members," thesis presented to the Cornell University at Ithaca, NY, in partial fulfillment of the requirements for the degree of Doctor of Philosophy. Also Dept. of Struct. Engrg, *Report No. 327.* Cornell University, Nov., 1966.

[13] Van den Berg, G. J., and Van der Merwe, P. (1987). "Experimental stress-strain curves for cold-rolled types 201 and 304 steel sheets." Rand Afrikaans University, Johannesburg, South Africa, Nov.

[14] Wang, S. T. (1969). Cold-rolled austenitic stainless steel: Material properties and structural performance, *Report No. 334,* Dept. of Struct. Engrg, Cornell University, Ithaca, N.Y., July.

[15] "Statistical summary—Mechanical properties of 1/4 hard and 1/2 hard types 201 and 301 stainless steels." (1988). Allegheny Ludlum Steel, Pittsburgh, Pa., June.

[16] Ravindra, M. K., and Galambos, T. V. (1978). Load and resistance factor design for steel. *J. Struct. Div., Proc.,* ASCE, *104*(ST9).

[17] Hsiao, L. E., Yu, W. W., and Galambos, T. V. (1988). "Load and resistance factor design of cold-formed steel: Calibration of the AISI specification." *Ninth Progress Report,* University of Missouri-Rolla, Feb.

[18] Ellingwood, B., Galambos, T. V., MacGregor, J. G., and Cornell, C. A. (1980). "Development of a probability based load criterion for American national standard A58: Building code requirements for minimum design loads in buildings and other structures." *NBS Special Publication 577,* June.

[19] Galambos, T. V., Ellingwood, B. R., MacGregor, J. G., and Cornell, C. A. (1982). "Probability based load criteria: Assessment of current design practice." *J. Struct. Div., Proc.,* ASCE *108*(ST5), 959–977.

[20] Ellingwood, B. R., MacGregor, J. G., Galambos, T. V., and Cornell, C. A. (1982). Probability Based Load Criteria: Load Factors and Load

Combinations. *J. Struct. Div., Proc.,* ASCE *108*(ST5), 978–997.

[21] Lin, S. H., Yu, W. W., and Galambos, T. V. (1988). "Load and resistance factor design of cold-formed stainless steel—Statistical analyses of material properties and development of the LRFD provisions." *Fourth Progress Report,* University of Missouri-Rolla, Oct.

[22] ANSI/ASCE 7-88 *Minimum Design Loads for Buildings and Other Structures,* American Society of Civil Engineers.

[23] Johnson, A. L., and Winter, G. (1966). Behavior of stainless steel columns and beams. *Proc.,* ASCE, *92*(ST5), 97–118.

[24] Karren, K. W., and Winter, G. (1964). "The effects of cold-forming on the mechanical properties of structural sheet steel." *Report No. 317,* Dept of Struct. Engrg., Cornell University, Ithaca, N.Y., Sept.

[25] Callender, J. H. (1955). "The design of stainless steel curtain walls." *Arch. Rec.,* Oct.

[26] Winter, G. (1970). *Commentary on the 1968 edition of the specification for the design of cold-formed steel structural members.* American Iron and Steel Institute.

[27] Wang, S. T., Errera, S. J. and Winter, G. (1975). "Behavior of Cold-Rolled Stainless Steel Members." *J. Struct. Div., Proc.,* ASCE *101*(ST11), 2337–2357. 1975.

[28] Bleich, F. (1952). *Buckling strength of metal structures.* McGraw-Hill Book Company, New York, N.Y.

[29] Pekoz, T. B. (1986). "Development of a unified approach to the design of cold-formed steel members." *Report SG-86-4,* American Iron and Steel Institute, Nov.

[30] Bijlaard, P. P. (1941). *Theory of the plastic stability of thin plates.* Int. Assoc. Bridge and Struct. Engrg, Vol. VI, 45.

[31] Stowell, E. Z. (1948). "A unified theory of plastic buckling of columns and plates." *NACA Tech. Note 1556,* Apr.

[32] Phung, N., and Yu, W. W. (1978). "Structural behavior of transversely reinforced beam webs." *Final Report, Civ. Engrg. Study 78-5.* University of Missouri-Rolla, June.

[33] Galambos, T. V. (1968). *Structural members and frames.* Prentice-Hall Inc., Englewood Cliffs, N.J.

[34] Pekoz, T. B., and Winter, G. (1969). Torsional-Flexural Buckling of Thin-Walled Sections Under Eccentric Load. *Proc.,* ASCE, *J. Struct. Div., 95*(ST5), 941–963.

[35] Timoshenko, S. P., and Gere, J. M. (1961). *Theory of elastic stability.* (2nd Ed). McGraw-Hill Book Company, New York, N.Y.

[36] Gerard, G. (1962). *Introduction to structural stability theory.* McGraw-Hill Book Company, New York, N.Y.

[37] Hetrakul, N., and Yu, W. W. (1978). "Structural behavior of beam webs subjected to web crippling and a combination of web crippling and bending." *Final Report, Civ. Engrg. Study 78-4,* University of Missouri-Rolla, June.

[38] Yu, W. W. (1980). *Web crippling and combined web crippling and bending of steel decks.* University of Missouri-Rolla.

[39] *Sectional properties of corrugated steel sheets.* (1964). American Iron and Steel Institute.

[40] Yu, W. W. (2000). *Cold-formed steel design.* Wiley-Interscience, New York, N.Y.

[41] "Supplementary information on the 1986 edition of the specification for the design of cold-formed steel structural members." (1986). *Cold-formed steel design manual—Part III.* American Iron and Steel Institute.

[42] "Commentary on the 1986 edition of the specification for the design of cold-formed steel structural members." (1986). *Cold-formed steel design manual—Part II.* American Iron and Steel Institute.

[43] Winter, G. (1960). "Lateral bracing of columns and beams." *Trans.,* ASCE.

[44] Salmon, C. G., and Johnson, J. E. (1980). *Steel structure, design and behavior.* (2nd Ed), Harper & Row, New York, N.Y.

[45] Lutz, L. A., and Fisher, J. M. (1985). A unified approach for stability bracing requirements. *Engrg. J.,* American Institute of Steel Construction. 4th Quarter, *22* (4).

[46] Haussler, R. W. (1964). "Strength of elastically stabilized beams." *Proc.,* ASCE, *90*(ST3). Also ASCE *Trans. 130,* 1965.

[47] Haussler, R. W., and Pabers, R. F. (1973). "Connection strength in thin metal roof structures." *Proc.,* 2nd Int. Specialty Conference on Cold-Formed Steel Structures. University of Missouri-Rolla, Oct.

[48] Errera, S. J., Tang, B. M., and Popowich, D. W. (1970). Strength of bolted and welded connections in stainless steel. *Report No. 335,* Dept. Struct. Engrg., Cornell University, Ithaca, N.Y., Aug.

[49] Errera, S. J., Popowich, D. W., and Winter, G. (1974). "Bolted and welded stainless steel con-

nections." *Proc.,* ASCE, *J. Struct. Div., 100*(ST6), 1279–1296.

[50] Flannery, J. W. (1968). "Tests justify higher design stress for welded quarter-hard 301 stainless." *Welding Engrg,* Apr.

[51] "Specification for corrosion-resisting chromium and chromium-nickel steel bare and composite metal cored and stranded welding electrodes and welding rods." (1981). *AWS A5.9-81,* American Welding Society.

[52] "Specification for covered corrosion-resisting chromium and chromium-nickel steel welding electrodes." (1981). *AWS A5.4-81,* American Welding Society.

[53] "Recommended practices for resistance welding." (1966). *AWS C-1.1-66.* American Welding Society.

[54] "Standard specification for alloy-steel and stainless steel bolting materials for high temperature service." (1986). *ASTM Designation: A 193/A 193M-86,* American Society for Testing and Materials.

[55] *Design of light gage steel diaphragms* (1st Ed). (1967). American Iron and Steel Institute.

[56] Chajes, A., Britvec, S. J., and Winter, G. (1963). "Effects of cold-straining on structural steels." *Proc.,* ASCE, *89*(ST2), 1–32.

[57] Winter, G. (1947). "Strength of thin steel compression flanges." *Trans.,* ASCE, *112,* 527–554.

[58] Ramberg, W., and Osgood, W. R. (1943). "Descriptions of stress-strain curves by three parameters." *NACA Tech. Note, No. 902,* July.

[59] Hill, H. N. (1944). "Determination of stress-strain relations from offset yield strength values." *NACA Tech. Note, No. 927,* Feb.

[60] Pekoz, T., and Hall, W. B. (1988). "Probabilistic evaluation of test results." *Proc.,* 9th Int. Specialty Conference on Cold-Formed Steel Structures, University of Missouri-Rolla.

[61] "Building officials and code administrators." (1990). *National Building Code.*

[62] Wang, S. T., and Yeh, S. S. (1973). "Post-local-buckling behavior of continuous beams." *J. Struct. Div., Proc.,* ASCE, *100*(ST6), 1169–1187.

[63] "Standard specification for stainless steel bolts, hex cap screws, and studs." *ASTM Designation, F593-86a,* American Society for Testing and Materials.

[64] Van den Berg, G. J., and van der Merwe, P. (1992). "Prediction of Corner Mechanical Properties for Stainless Steels due to Cold-Forming." *Proc., 11th Intl. Spec. Conf. on Cold-Formed Steel Struct.,* Oct., St. Louis, Mo.

[65] Coetzee, J. S., van den Berg, G. J., and van der Merwe, P. (1990). "The Effect of Workhardening and Residual Stresses due to Cold-Work of Forming on the Strength of Cold-Formed Stainless Steel Lipped Channel Sections." *Proc., 10th Intl. Spec. Conf. on Cold-Formed Steel Struct.,* Oct., St. Louis, Mo.

[66] Rasmussen, K. J. R., and Hancock, G. J. (1993). "Design of Stainless Steel Tubular Members. II: Beams." *J. Struct. Div., Proc.,* ASCE, *119*(No. 8), 2368–2386.

[67] Van Wyk, M. L., van den Berg, G. J., and van der Merwe, P. (1990). "The Lateral Torsional Buckling Strength of Doubly Symmetric Stainless Steel Beams." *Proc., 10th Intl. Spec. Conf. on Cold-Formed Steel Struct.,* Oct., St. Louis, Mo.

[68] Bredenkamp, P. J., van den Berg, G. J., and van der Merwe, P. (1992). "The Lateral Torsional Buckling Strength of Cold-Formed Stainless Steel Lipped Channel Beams." *Proc., 11th Intl. Spec. Conf. on Cold-Formed Steel Struct.,* Oct., St. Louis, Mo.

[69] Carvalho, E. C. G., van den Berg, G. J., and van der Merwe, P. (1990). "Local Shear Buckling in Cold-Formed Stainless Steel Beam Webs." *Proc., Annual Tech. Session Struct. Stability Res. Council,* Apr., St. Louis, Mo.

[70] Carvalho, E. C. G., van den Berg, G. J., and van der Merwe, P. (1992). "Behaviour of Cold-Formed Stainless Steel Beams Subjected to Shear." *Proc., Annual Tech. Session Struct. Stability Res. Council,* Apr., Pittsburgh, Pa.

[71] Korvink, S. A., van den Berg, G. J., and van der Merwe, P. (1993). "Web Crippling of Stainless Steel Cold-Formed Beams." *Proc., Annual Tech. Session, Struct. Stability Res. Council,* Apr. 5–6, Milwaukee, Wis. 79–101.

[72] Korvink, S. A., van den Berg, G. J., and van der Merwe, P. (1992). "Web Crippling of Stainless Steel Cold-Formed Beams." *Proc., 11th Intl. Spec. Conf. on Cold-Formed Steel Struct.,* Oct., St. Louis, Mo.

[73] Rasmussen, K. J. R., and Hancock, G. J. (1993). "Design of Stainless Steel Tubular Members. I: Columns." *J. Struct. Div., Proc.,* ASCE, *119* (No. 8), 2349–2367.

[74] Coetzee, J. S., van den Berg, G. J., and van der Merwe, P. (1990). "The Behaviour of Stainless Steel Lipped Channel Axially Loaded Compression Members." *Proc., Annual Tech. Session, Struct. Stability Res. Council,* Apr., St. Louis, Mo.

[75] Van den Berg, G. J., and van der Merwe, P. (1988). "Torsional Flexural Buckling of Cold-Formed Stainless Steel Columns." *Proc., 9th Intl. Spec. Conf. on Cold-Formed Steel Struct.,* Nov., Rolla, Mo.

[76] Lin, S. H., Yu, W. W., and Galambos, T. V. (1992). "ASCE LRFD Method for Stainless Steel Structures." *ASCE J. Struct. Engrg.* 118 (no. 4), 1056–1070.

[77] Reyneke, W., and van den Berg, G. J. (1996). "The Strength of Partially Stiffened Stainless Steel Compression Members." *Proc., 13th Intl. Spec. Conf. on Cold-Formed Steel Struct.,* Oct., St. Louis, Mo.

[78] Rasmussen, K. J. R., and Rondal, J. (1996). "An Explicit Approach to Design of Stainless Steel Columns." *Proc., 13th Intl. Spec. Conf. on Cold-Formed Steel Struct.,* Oct., St. Louis, Mo.

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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 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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