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Design of cold-formed stainless steel structural members proposed allowable stress design specification with commentary

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Third Progress Report

DESIGN OF COLD-FORMED STAINLESS STEEL STRUCTURAL MEMBERS

PROPOSED ALLOWABLE STRESS DESIGN SPECIFICATION WITH COMMENTARY

by

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INTRODUCTION

This progress report on the design of cold-formed stainless steel structural members contains the following two parts:

Part I: Proposed Specification for the Design of Cold-Formed Stainless Steel Structural Members (Third Draft).

Part II: Commentary on the Proposed Specification for the Design of Cold-Formed Stainless Steel Structural Members (Second Draft).

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PART I

PROPOSED SPECIFICATION FOR THE DESIGN OF COLD-FORMED STAINLESS STEEL STRUCTURAL MEMBERS

(Third Draft)

January 1988

FOREWORD

(To be prepared by American Society of Civil Engineers)

ASCE STANDARD SPECIFICATION FOR THE DESIGN OF COLD-FORMED STAINLESS STEEL STRUCTURAL MEMBERS

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NOTATION

Symbol	Definition	Section
A	Full, unreduced cross-sectional area of the member	3.4,3.6.2
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. D.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. D.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_s	Cross-sectional area of transverse stiffeners	2.4,2.4.1, 2.4.2,App. D.1
A'_s	Effective area of stiffener	2.4,2.4.1, 2.4.2
A_{st}	Gross area of shear stiffener	App. D.2
A_w	Cross section area of the thinner welded part	5.2.1
A_{wn}	Net web area	5.4
a	For a reinforced web element, the distance between transverse stiffeners	App. D.2
a	Length of bracing interval	4.3.2.2
b	Effective design width of compression element	2.2.1,2.2.1, 2.3.1,2.3.2, 2.4.1,2.4.2,2.5

Symbol	Definition	Section
b_d	Effective width for deflection calculation	2.2.1,2.2.2
b_e	Effective design width of sub-element or element	1.1.2,2.5
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 the effective proportional limit-to-yield strength, F_{pr}/F_y	3.6.1,3.6.2
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_v	Shear stiffener coefficient	App. D.2
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	App. B
D	Outside diameter of cylindrical tube	3.6.1,3.6.2
D	Dead load, includes weight of the 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. D.2
d	Depth of section	2.4,4.1.1, 4.3.2.2,App. B
d	Diameter of bolt	5.3,5.3.1, 5.3.2,5.3.3

Symbol	Definition	Section
d_h	Diameter of standard hold	5.3.1,5.4
d_s	Reduced effective width of stiffener	2.4,2.4.2
d'_s	Actual effective width of stiffener	2.4,2.4.2
d_{wc}	Coped web depth	5.4
E_o	Initial modulus of elasticity	2.2.1,2.3.1, 2.4,3.3.1.1, 3.3.1.2,3.3.2, 3.3.5,3.6.1, 3.6.2,4.3.3
E_r	Reduced modulus of elasticity	2.2.1
E_s	Secant modulus	3.3.1.1,3.4
E_{sc}	Secant modulus in the compression flange	2.2.1
E_{st}	Secant modulus in the tension flange	2.2.1
E_s/E_o	Plasticity reduction factor for unstiffened compression elements	3.3.1.1,3.4
E_t	Tangent modulus in compression	3.3.1.1,3.4, 3.4.1,3.6.2
E_t/E_o	Plasticity reduction factor for lateral buckling	3.3.1.2,3.6.2
$\sqrt{E_t/E_o}$	Plasticity reduction factor for stiffened compression elements	3.3.1.1,3.4
e_{min}	The distance e measured in the line of force from the centerline of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part toward which the force is directed	5.3.1

Symbol	Definition	Section
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_p	Allowable bearing stress	5.3.3
F_{pr}	Effective proportional limit	3.6.1
F_t	Nominal tension stress limit on net section	5.3.2,5.3.4
F'_t	Nominal tensile strength for bolts subject to combination of shear and tension	5.3.4
F_u	Tensile strength in the longitudinal direction	5.3.1,5.3.2, 5.3.3,5.3.4
F_{ua}	Tensile strength of the annealed base metal	5.2.1,5.2.2
F_v	Allowable shear stress on the gross area of a bolt	5.3.4
F_{xx}	Strength level designation in AWS electrode classification	5.2.1,5.2.2
F_y	Yield strength used for design, not to exceed the specified yield strength or established in accordance with Section 6.4, or as increased for cold work of forming in Section 1.5.2.2	1.1.2,1.3.3, 1.5.2.1,1.5.2.2, 2.2.1,2.5,3.2, 3.2,3.3.1,3.3.2, 3.3.4,3.3.5, 3.6.1,3.6.2, 5.2.1,App. D.1
F_{ys}	Yield strength of stiffener steel	App. D.1
F_{yv}	Shear yield strength	3.3.2
F_{yw}	Lower value of the yield strength in beam web F_y or stiffener section F_{ys}	App. D.1

Symbol	Definition	Section
f	Stress in the 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 the full, unreduced flange width	App. B
f_b	Perceptible stress for local distrotion	3.3.1.1,3.4
f_d	Computed compressive stress in the element being considered. Calculations are based on the effective section at the load for which deflections are determined	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
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 a bolt	5.4
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	3.3.2
g	Vertical distance between two rows of connections nearest to the top and bottom flanges	4.1.1

Symbol	Definition	Section
h	Depth of flat portion of web measured along the plane of web	2.1.2,3.3.2, 3.3.4,App. D.2
I_a	Adequate moment of inertia of stiffener so that each component element will behave as a stiffened element	2.1.1,2.4, 2.4.1,2.4.2
I_b	Moment of inertia of the full, unreduced section about the axis of bending	3.5
I_s	Actual moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened	2.1.2,2.4, 2.4.1,2.4.2, 2.5
I_{sf}	Moment of inertia of the full area of the multiple stiffened element, including the intermediate stiffeners, about its own centroidal axis parallel to the element to be stiffened	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 the compression portion of a section about the 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

Symbol	Definition	Section
K_b	Effective length factor in the 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. D.2
L	Full span for simple beams, distance between inflection points for continuous beams, twice the length of cantilever beams	4.1.1,App. C
L	Length of fillet weld	5.2.2
L	Unbraced length of member	3.3.1.2,3.4.1
L	Live load	6.2
L_b	Actual unbraced length in the plane of bending	3.5
L_{st}	Length of transverse stiffener	App. D.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	Applied bending moment	3.3.3,3.3.5
M_a	Allowable bending moment	3.3.1,3.3.3, 3.3.5,3.6.1

Symbol	Definition	Section
M_{ax},M_{ay}	Allowable moments about the centroidal axes determined in accordance with Section 3.3	3.5
M_{axo}, M_{ayo}	Nominal moments about the centroidal axes determined in accordance with Section 3.3.1 excluding the provisions of Section 3.3.1.2	3.5
M_c	Critical moment	3.3.1.2
M_{ld}	Permissible moment for local distortions	3.3.1.1
M_n	Nominal moment strength	3.3.1,3.3.1.1, 3.3.1.2,3.6.1
M_x,M_y	Applied moments about the centroidal axes determined in accordance with Section 3.3	3.5
M_y	Moment causing initial yielding	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 the shear center of one channel to the mid-plane of its web	4.1.1,4.3.2.2
N	Actual length of bearing	3.3.4
n	Number of holes	5.4
P	Applied axial load	3.5
P	Concentrated load or reaction	3.3.5
P	Force transmitted by bolt	5.3,5.3.1
P	Force transmitted by weld	5.2,5.2.2
P_a	Allowable concentrated load or reaction for one transverse stiffener	5.2.1,5.2.2

Symbol	Definition	Section
P_{ao}	Allowable axial load determined in accordance with Section 3.4 for L = 0.	3.5
P_{cr}	$\pi^2 EI_b/(K_b L_b)^2$	3.5
P_L	Force to be resisted by intermediate beam brace	4.3.2.2
P_{ld}	Permissible load for local distortions	3.4
P_n	Nominal axial strength of member	3.4,3.6.2
P_n	Nominal strength of connection component	5.2.2,5.2.3
q	Uniformly distributed factored load in the plane of the web	4.1.1
R	Required load carrying capacity	6.2
R	Coefficient	3.4
R	Inside bend radius	3.3.4
r	Raqius 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 the given conditions of end support and intermediate bracing	4.1.1
r_o	Polar radius of gyration of cross section about the shear center	3.3.1.2,3.4.3

Symbol	Definition	Section
r_x, r_y	Radius of gyration of cross section about centroidal principal axes	3.3.1.2
S	$1.28\sqrt{E_o/f}$	2.4,2.4.1
S_c	Elastic section modulus of the effective section calculated at a stress M_c/S_f in the extreme compression fiber	3.3.1.1,3.3.1.2, 3.4
S_e	Elastic section modulus of the 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	Fastener spacing	4.1.2
s	Spacing in line of stress of welds, rivets, or bolts connecting a compression coverplate or sheet to a non-integral stiffener or other element	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 an I-section	4.1.1
T_a	Allowable tensile strength	3.2,5.3.2
T_n	Nominal tensile strength	3.2
T_s	Strength of connection in tension	4.1.1

Symbol	Definition	Section
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. D
t	Thickness of thinnest connected part	5.3.1,5.4
t_s	Equivalent thickness of a multiple-stiffened element	2.5,App. D.1
t_w	Effective throat of weld	5.2.2
V	Actual shear force	3.3.3
V_a	Allowable shear force	3.3.2,3.3.3,5.4
V_n	Nominal shear force	5.4
w	Flat width of element exclusive of radii	1.1.2,1.5.2.1, 2.1.1,2.1.2, 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

Symbol	Definition	Section
w	Flat width of the bearing plate	3.3.5
w_f	Width of flange projection beyond the web or half the distance between webs for box- or U-type sections	App. C
w_f	Projection of flanges from inside face of web	App. B
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. D.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
$1/\alpha_x$, $1/\alpha_y$	Magnification factors	3.5
η	Plasticity reduction factor	3.3.1.1,3.4
θ	Angle between web and bearing surface $\geq 45^o$ but no more than 90^o	3.3.4
μ	Poisson's ratio in the elastic range = 0.3	3.3.1.1,3.4
σ_{ex}	Torsional buckling stress about x axis	3.3.1.2,3.4.3
σ_{ey}	Torsional buckling stress about y axis	3.3.1.2
σ_t	Torsional buckling stress	3.3.1.2,3.4.3

Symbol	Definition	Section
ρ	Reduction factor	2.2.1
λ	Slenderness factor	2.2.1
λ_c	3.048C	3.6.1
Ψ	f_2/f_1	2.2.2
Ω_b	Factor of safety for bearing	5.3.3
Ω_c	Factor of safety for axial compression	3.4,3.5, 3.6.2,App D.1
Ω_e	Factor of safety for sheet tearing	5.3.1
Ω_f	Factor of safety for flexural	3.3.1,3.6.1
Ω_{st}	Factor of safety for end crushing of transverse stiffener	App. D.1
Ω_t	Factor of safety for tension on net section	3.2,5.3.2
Ω_v	Factor of safety for shear rupture	5.4
Ω_{w1}	Factor of safety for yielding	5.2.1
Ω_{w2}	Factor of safety for fracture of annealed base metal	5.2.1,5.2.2
Ω_{w3}	Factor of safety for fracture of weld metal	5.2.1,5.2.2

CONVERSION TABLE

This table contains some conversion factors between US Customary and SI Metric Units.

	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

ASCE STANDARD SPECIFICATION FOR THE DESIGN OF COLD-FORMED STAINLESS STEEL STRUCTURAL MEMBERS

1. GENERAL

1.1 Limits of Applicability and Terms

1.1.1 Scope and Limits of Applicability

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

Nothing herein is intended to conflict with the provisions of the "Specification for the Design of Cold-Formed Steel Structural Members" issued by American Iron and Steel Institute and governing the design of structural members cold-formed to shape from carbon and high strength, low alloy steel sheet or strip.

1.1.2 Terms

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

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

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

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

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

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

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

(g) Confirmatory Test. A confirmatory test is a test made, when desired, on members, connections, and assemblies designed according to the provisions of Sections 1 through 5 of this Specification or its specific references, in order to compare actual versus calculated performance.

(h) 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.

(i) Specified Minimum Yield Strength. The specified minimum yield strength is the lower limit of yield strength which varies with the rolling direction and the type of stress, 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.

(j) 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 width 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.

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 Non-Conforming Shapes and Constructions

The provisions of the Specification are not intended to prevent the use of alternate shapes or constructions not specifically prescribed herein. Such alternates shall meet the provisions of Section 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

The maximum thickness for Type 409 used in this Specification is limited to 0.15 inch (3.8 mm). The maximum thickness for Types 430 and 439 is limited to 0.125 inch (3.2 mm).

1.3.2 Other Stainless Steels

The provision 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 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 percent for a two-inch 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 stainless steel thickness of the cold-formed product as delivered to the job site shall not at any location be less than 95 percent of the thickness, t, used in its design; however, thickness may 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 Load

The live 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 or Earthquake Loads

Where load combinations specified by the applicable building code include wind or earthquake loads, the resulting forces may be multiplied by 0.75.

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 stablity under ponding conditions.

1.5 Structural Analysis and Design

1.5.1 Design Basis

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

1.5.2 Yield Strength and Strength Increase from Cold Work of Forming

1.5.2.1 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 1.5.2.2.

1.5.2.2 Strength Increase from Cold Work of Forming

Except as permitted by this Section, allowable stresses shall be based upon 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 1.5.2.2.1 and 1.5.2.2.2.

1.5.2.2.1 Type of Sections

The provisions of Section 1.5.2.2 shall apply only to the following, regardless of whether the stress to be imposed on the member in service is in tension or compression:

(a) 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.

(b) Cylindrical tubular members in which the ratio D/t of outside diameter to wall thickness does not exceed $0.112E_o/F_y$.

1.5.2.2.2 Limitations

Application of the provisions of Section 1.5.2.2 shall be on the following basis:

(a) Mechanical properties shall be determined on the basis of full section tests, in accordance with the provisions of Section 6.4.

(b) Provisions shall apply only to the following Sections of the Specification:

1.4.4	Wind or Earthquake Loads
3.2	Tension Members
3.3.1.1	Nominal Section Strength
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 upon the properties of the unformed material.

(c) 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.

1.5.3 Serviceability

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

1.5.4 Design Tables and Figures

The design tables (Tables A1 through A19) and figures (Figures A1 through A12) used in this Specification are given in Appendix A.

1.6 Reference Documents

This Specification recognizes other published and latest approved specifications and manuals for design contemplated herein, as follows:

1. American National Standards Institute, ANSI A58.1-1982, "Minimum Design Loads in Buildings and Other Structures," American National Standards Institute, Inc., (ANSI), 1430 Broadway, New York, New York 10018

2. Applicable standards of the American Society for Testing and Materials, (ASTM), 1916 Race Street, Philadelphia, Pennsylvania 19013

3. American Welding Society, AWS D1.3-81, "Structural Welding Code-Sheet steel," American Welding Society, (AWS), 550 N.W. LeJeune Road, Miami, Florida 33126

4. American Iron and Steel Institute, "Cold-Formed Steel Design Manual," 1986 Edition, American Iron and Steel Institute, (AISI), 1133 15th Street, N.W., Washington, D.C. 20005-2701

2. ELEMENTS

2.1 Dimensional Limits and Considerations

2.1.1 Flange Flat-Width-to-Thickness Considerations

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

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

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

Simple lip	50
Any other kind of stiffener having $I_s > I_a$ and $D/w < 0.8$ according to Section 2.4.2	90

(2) Stiffened compression element with both longitudinal edges connected to other stiffened elements — 400

(3) Unstiffened compression element and elements with an 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 in excess of approximately 75 are likely to develop noticeable out-of-plane distortions at the full allowable 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 large w/t ratios, Sections 3.3.1.1 and 3.4 stipulates 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.

(b) Flange Curling

Provisions for limiting the amount of curling or the deflection of a tension or compression flange are given in Appendix B.

(c) Shear Lag Effects-- Unusually Short Spans Supporting Concentrated Loads

The effective design width of tension and compression flanges of beams supporting concentrated loads and having short spans shall be limited in accordance with Appendix C.

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:

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

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

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

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

In the above,

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

t = Web thickness

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

2.2 Effective Widths of Stiffened Elements

2.2.1 Uniformly Compressed Stiffened Elements

(a) Load Capacity Determination

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

$b = w$ when $\lambda \leq 0.673$ (Eq. 2.2.1-1)

$b = \rho w$ when $\lambda > 0.673$ (Eq. 2.2.1-2)

where

w = Flat width as shown in Figure 1

$\rho = (1 - 0.22/\lambda)/\lambda$ (Eq. 2.2.1-3)

λ is a slenderness factor determined as follows:

$\lambda = (1.052/\sqrt{k})(w/t)(\sqrt{f/E_o})$ (Eq. 2.2.1-4)

where

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 logitudinal edge. Values for stiffened elements with an edge stiffener or one intermediate stiffener are gven in Section 2.4.

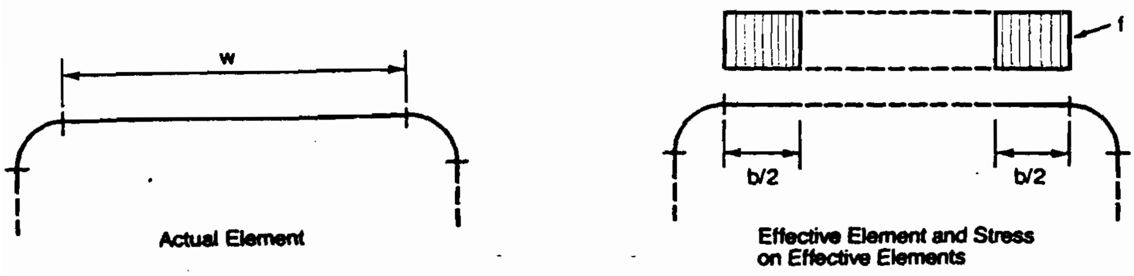

Figure 1 Stiffened Elements with Uniform Compression

f for load capacity determination is as follows:

For flexural members:

(1) If Section 3.3.1.1 is used, $f = F_y$ if the initial yielding is in compression in the element considered.

If the initial yielding is not in the compression element considered, then the stress shall be determined for the element considered on the basis of the effective section at M_y (moment causing initial yielding).

(2) 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 to be F_n as determined in Section 3.4.

(b) Deflection Determination

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

$b_d = w$ when $\lambda \leq 0.673$ (Eq. 2.2.1-5)

$b_d = \rho w$ when $\lambda > 0.673$ (Eq. 2.2.1-6)

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 is the computed compression stress in the element being considered, and that the reduced modulus of elasticity, E_r, shall be substituted for E_o in Eq. 2.2.1-4.

where

$E_r = (E_{st}+E_{sc})/2.$ (Eq. 2.2.1-7)

E_{st} = Secant modulus corresponding to the stress in the tension flange

E_{sc} = Secant modulus corresponding to the stress in the compression flange

Values of the secant moduli may be obtained from Tables A2 and A3 or Figures A1 and A2 in Appendix A.

2.2.2 Effective Widths of Webs and Stiffened Elements with Stress Gradient

(a) Load Capacity Determination

The effective widths, b_1 and b_2 as shown in Figure 2 shall be determined from the following formulas:

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

For $\psi \leq -0.236$

$$b_2 = b_e/2 \qquad \text{(Eq. 2.2.2-2)}$$

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

For $\psi > -0.236$

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

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:

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

$\psi = f_2/f_1$

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, $f_1 \geq f_2$.

(b) Deflection Determination

The effective widths in computing deflections at a given load shall be determined in accordance with Section 2.2.2a 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 deflection are determined.

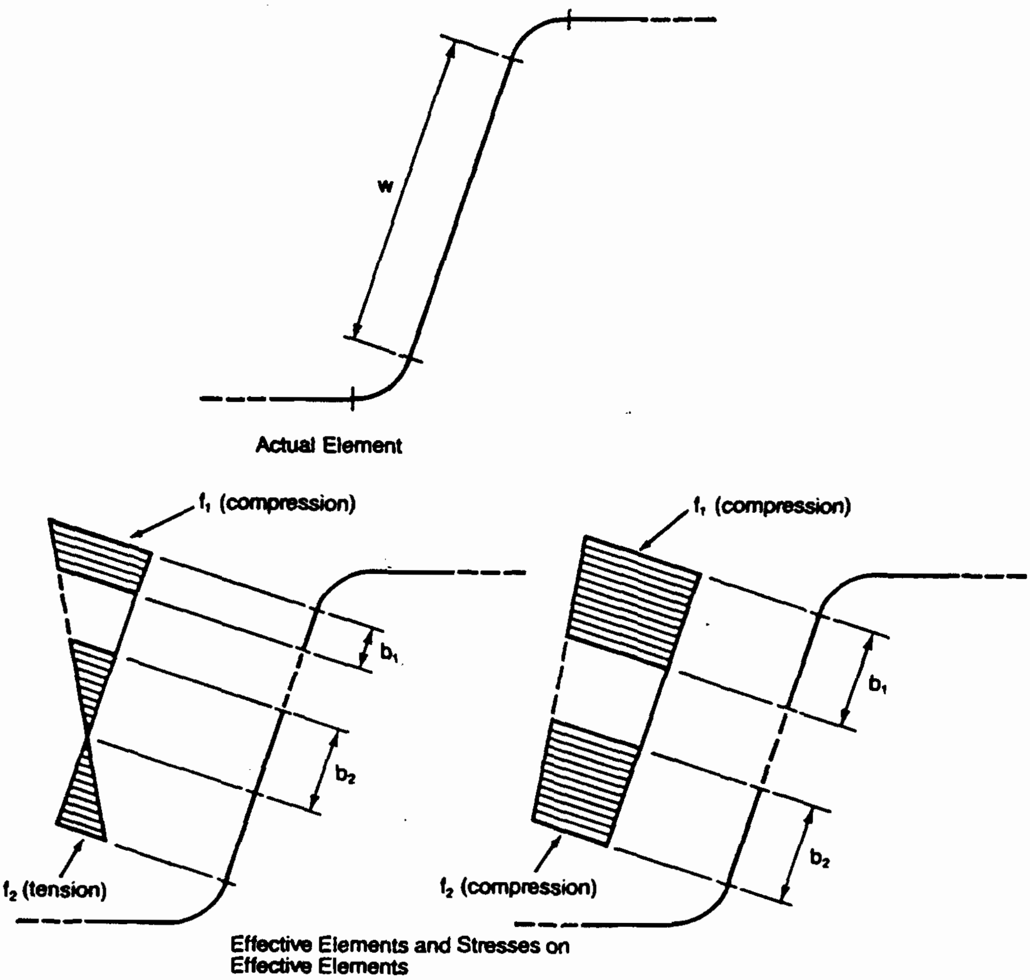

Figure 2 Stiffened Elements with Stress Gradient and Webs

2.3 Effective Widths of Unstiffened Elements

2.3.1 Uniformly Compressed Unstiffened Elements

(a) Load Capacity Determination

The effective widths, b, of unstiffened compression elements with uniform compression shall be determined in accordance with Section 2.2.1a with the exception that k should be taken as 0.5, and w is defined in Figure 3.

(b) Deflection Determination

The effective widths used in computing deflections shall be determined in accordance with Section 2.2.1.b except that k should be taken as 0.5.

2.3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

(a) Load Capacity Determination

Effective widths, b, of unstiffened compression elements and edge stiffeners with stress gradient shall be determined in accordance with Section 2.2.1a with $f = f_3$ as in Figure 5 in the element and $k = 0.5$.

(b) Deflection Determination

Effective widths, b, of unstiffened compression elements and edge stiffeners with stress gradient shall be determined in accordance with Section 2.2.1b except that f_{d3} is substituted for f and $k = 0.5$.

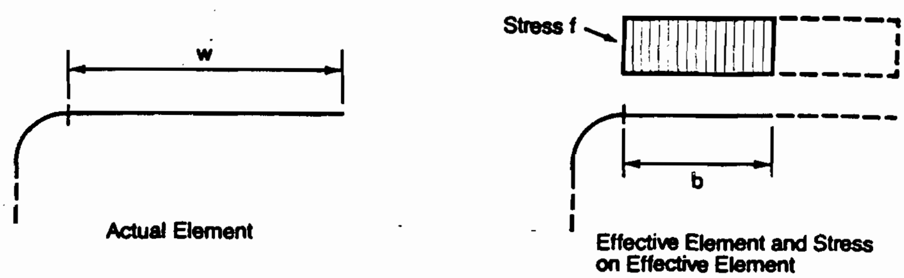

Figure 3 Unstiffened Elements with Uniform Compression

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

The following notation is used in this section.

S = $1.28\sqrt{E_o/f}$

k = Buckling coefficient

b_o = Dimension defined in Figure 4

d, w, D = Dimensions defined in Figure 5

d_s = Reduced effective width of the stiffener as specified in this section. d_s, calculated according to Section 2.4.2, is to be used in computing the overall effective section properties (see Figure 5)

d'_s = Effective width of the stiffener calculated according to Section 2.3.1 (see Figure 5)

C_1,C_2 = Coefficients defined in Figures 4 and 5

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

I_a = Adequate moment of inertia of stiffener, so that each component element will behave as a stiffened element.

I_s, A'_s = Moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened and the effective area of the stiffener, respectively. For edge stiffeners the round corner between the stiffener and the element to be

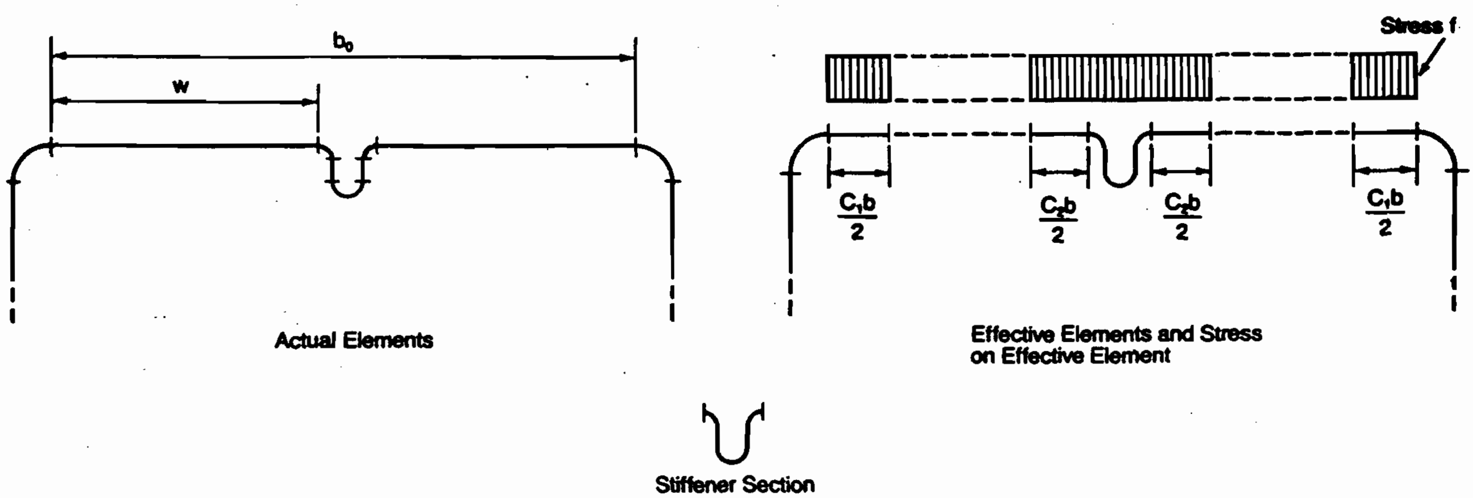

Figure 4 Elements with Intermediate Stiffeners

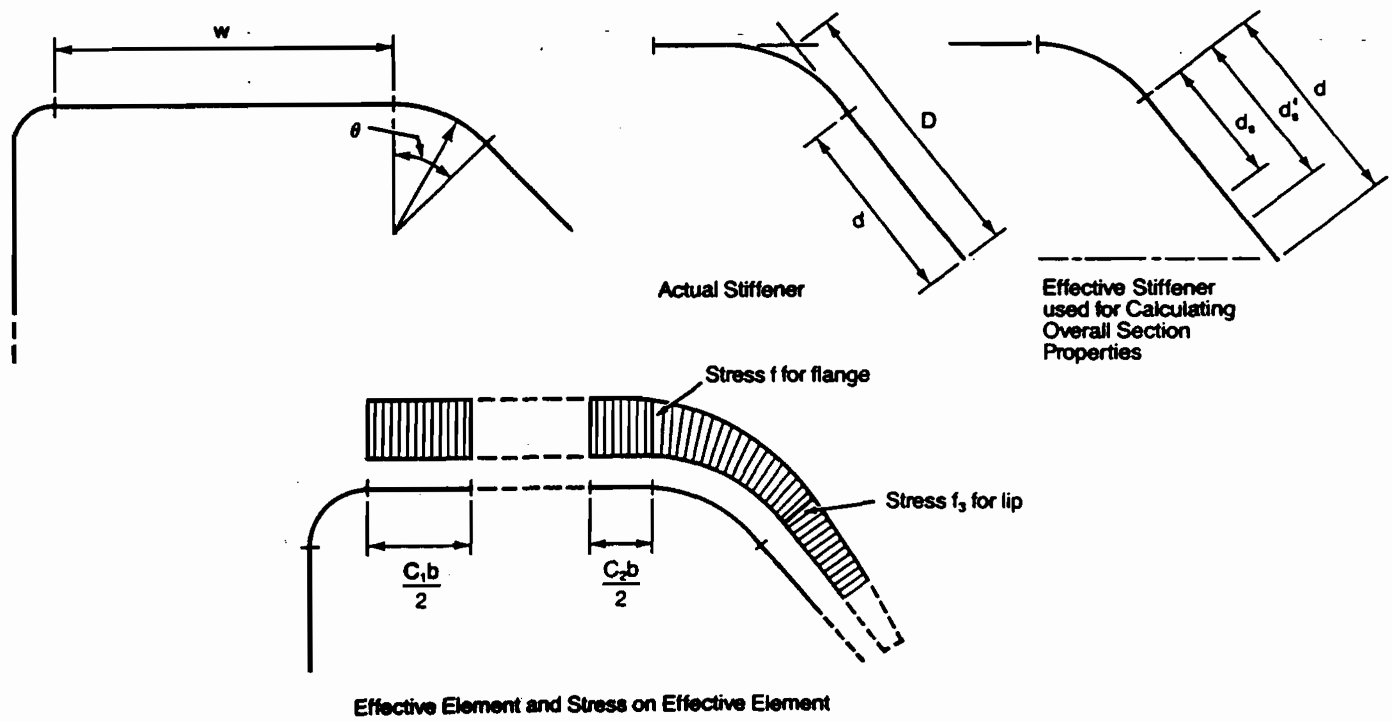

Figure 5 Elements with Edge Stiffeners

stiffened shall not be considered as a part of the stiffener.

For the stiffener shown in Figure 5,

$I_s = (d^3 t \sin^2\theta)/12$ (Eq. 2.4-2)

$A'_s = d'_s t$

2.4.1 Uniformly Compressed Elements with an Intermediate Stiffener

(a) Load Capacity Determination

Case I: $b_o/t \leq S$ (Eq. 2.4.1-1)

$I_a = 0$ (no intermediate stiffener needed)

$b = w$ (Eq. 2.4.1-3)

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

Case II: $S < b_o/t < 3S$ (Eq. 2.4.1-5)

$I_a/t^4 = [50(b_o/t)/S] - 50$ (Eq. 2.4.1-6)

b and A_s shall be calculated according to Section 2.2.1a

where $k = 3(I_s/I_a)^{1/2} + 1 \leq 4$ (Eq. 2.4.1-7)

$A_s = A'_s(I_s/I_a) \leq A'_s$ (Eq. 2.4.1-8)

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

$I_a/t^4 = [128(b_o/t)/S] - 285$ (Eq. 2.4.1-9)

b and A_s shall be calculated according to Section 2.2.1a

where $k = 3(I_s/I_a)^{1/3} + 1 \leq 4$ (Eq. 2.4.1-10)

$A_s = A'_s(I_s/I_a) \leq A'_s$ (Eq. 2.4.1-11)

(b) Deflection Determination

Effective widths shall be determined as in Section 2.4.1a except that f_d is substituted for f.

2.4.2 Uniformly Compressed Elements with an Edge Stiffener

(a) Load Capacity Determination

Case I: $w/t \leq S/3$ (Eq. 2.4.2-1)

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

$b = w$ (Eq. 2.4.2-3)

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

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

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

$I_a/t^4 = 399[(w/t)/S - 0.33]^3$ (Eq. 2.4.2-6)

$n = 1/2$

$C_2 = I_s/I_a \leq 1$ (Eq. 2.4.2-7)

$C_1 = 2 - C_2$ (Eq. 2.4.2-8)

b shall be calculated according to Section 2.2.1a where

$k = [4.82-5(D/w)](I_s/I_a)^n + 0.43$ (Eq. 2.4.2-9)

$\leq 5.25 - 5(D/w)$

for $0.8 \geq D/w > 0.25$

$k = 3.57(I_s/I_a)^n + 0.43 \leq 4.0$ (Eq. 2.4.2-10)

for $D/w \leq 0.25$

$d_s = d'_s(I_s/I_a) \leq d'_s$ (Eq. 2.4.2-11)

for simple lip stiffener

$A_s = A'_s(I_s/I_a) \leq A'_s$ (Eq. 2.4.2-12)

for other stiffener shape

Case III: $w/t \geq S$

$I_a/t^4 = [115(w/t)/S] + 5$ (Eq. 2.4.2-13)

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

(b) Deflection Determination

Effective widths shall be determined as in Section 2.4.2a 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:

$$I_{min} = [3.66 \sqrt{(w/t)^2 - (0.119E_o/F_y)}]t^4 \qquad \text{(Eq. 2.5-1)}$$

but not less than $18.4t^4$

where

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

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

(a) If the spacing of intermediate stiffeners between two webs is such that for the sub-element between stiffeners $b < w$ as determined in Section 2.2.1, only two intermediate stiffeners (those nearest each web) shall be considered effective.

(b) If the spacing of intermediate stiffeners between a web and an edge stiffener is such that for the sub-element between stiffeners $b < w$ as determined in Section 2.2.1, only one intermediate stiffener, that nearest the web, shall be considered effective.

(c) If intermediate stiffeners are spaced so closely that for the elements between stiffeners $b = w$ as determined in Section 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:

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

where

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

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

$$b_e/t = b/t - 0.10(w/t - 60) \qquad \text{(Eq. 2.5-3)}$$

where

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

b = Effective design width determined in accordance with the provisions of Section 2.2.1

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

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

For $60 < w/t < 90$:

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

where

$$\alpha = (3-2b_e/w)-(1/30)(1-b_e/w)(w/t) \qquad \text{(Eq. 2.5-5)}$$

For $w/t \geq 90$:

$$A_{ef} = (b_e/w)A_{st} \quad \text{(Eq. 2.5-6)}$$

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

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

2.6 Stiffeners

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

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 applied tensile force shall not exceed T_a determined as follows:

$$T_a = T_n/\Omega_t \qquad \text{(Eq. 3.2-1)}$$

where

T_n = Strength of member when loaded in tension

$= A_nF_y \qquad$ (Eq. 3.2-2)

A_n = Net area of the cross section

F_y = Specified yield strength as given in Table A1 in Appendix A

Ω_t = Factor of safety for tension

$= 1.85$

When mechanical fasteners are used in connections for tension members, the allowable tensile force shall also not exceed P_a determined from Section 5.3.2.

3.3 Flexural Members

3.3.1 Strength for Bending Only

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

$$M_a = M_n/\Omega_f \qquad \text{(Eq. 3.3.1-1)}$$

where

M_n = Smaller of the moment strengths calculated according to Sections 3.3.1.1 and 3.3.1.2

Ω_f = Factor of safety for bending

= 1.85

3.3.1.1 Nominal Section Strength

Section strength, M_n, shall be calculated on the basis of initiation of yielding in the effective section as follows:

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

where

F_y = Specified yield strength as given in Table A1.

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

When local distortions in flexural members under service loads must be limited, the allowable moment calculated in accordance with Eq. (3.3.1-1) shall not exceed the permissible moment determined by the compressive stresses, f_b, and the elastic section modulus of the full, unreduced section, S_f, as follows:

$$M_{ld} = S_f f_b \qquad \text{(Eq. 3.3.1.1-2)}$$

(i) If small, barely perceptible amounts of local distortions are allowed:

For stiffened compression elements

$$f_b = 1.2\ F_{cr} \qquad \text{(Eq. 3.3.1.1-3)}$$

For unstiffened compression elements

$$f_b = F_{cr} \qquad \text{(Eq. 3.3.1.1-4)}$$

(ii) If no local distortions are permissible:

For stiffened compression elements

$$f_b = 0.9\ F_{cr} \qquad \text{(Eq. 3.3.1.1-5)}$$

For unstiffened compression elements

$$f_b = 0.75 F_{cr} \qquad \text{(Eq. 3.3.1.1-6)}$$

where

F_{cr} = Critical buckling stress

$$= \frac{\Pi^2 k \eta E_o}{12(1-\mu^2)(w/t)^2} \qquad \text{(Eq. 3.3.1.1-7)}$$

η = Plasticity reduction factor corresponding to the compression stress

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

as given in Tables A6 and A7 or in Figures A3 and A4

$= E_s/E_o$, for unstiffened compression elements

as given in Tables A8 and A9 or in Figures A5 and A6

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

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

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

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 = Critical moment calculated according to (a), (b) or (c) below:

(a) For doubly-symmetric I-sections bent about the centroid axis perpendicular to the web (x-axis):

$$M_c = \Pi^2 E_o C_b (E_t/E_o) \frac{dI_{yc}}{L^2} \leq M_y \qquad \text{(Eq. 3.3.1.2-2)}$$

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

(b) For point-symmetric Z-sections bend about the centroid axis perpendicular to the web:

$$M_c = 0.5\Pi^2 E_o C_b (E_t/E_o) \frac{dI_{yc}}{L^2} \leq M_y \qquad \text{(Eq. 3.3.1.2-3)}$$

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

* The provisions of this Section apply to I-, Z-, C- and other singly-symmetric section flexural members (not including multiple-web deck, 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.

(c) 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):

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

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

(ii) For bending about the centroid axis perpendicular to the symmetry axis:

$$M_c = C_s C_b A\sigma_{ex}[j+C_s\sqrt{j^2+r_o^2(\sigma_t/\sigma_{ex})}] \leq M_y \qquad \text{(Eq. 3.3.1.2-5)}$$

In the above,

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

= $S_f F_y$

L = Unbraced length of the member

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

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

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

$\sigma_{ex} = [\Pi^2 E_o/(K_x L_x/r_x)^2](E_t/E_o)$ (Eq. 3.3.1.2-6)

$\sigma_{ey} = [\Pi^2 E_o/(K_y L_y/r_y)^2](E_t/E_o)$ (Eq. 3.3.1.2-7)

$\sigma_t = [1/(Ar_o^2)][G_oJ+\Pi^2 E_o C_w/(K_t L_t)^2](E_t/E_o)$ (Eq. 3.3.1.2-8)

A = Full cross-sectional area

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

d = Depth of section

E_t/E_o = Plasticity reduction factor corresponding to the stress, as given in Tables A10 and A11 or in Figures A7 and A8

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

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

where

M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length, taken about the strong axis of the member, and where M_1/M_2, the ratio of end moments, is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite sign (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, and for members subject to combined axial load and bending moment (Section 3.5), C_b shall be taken as unity.

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

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

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

G_o = Initial shear modulus (Tables A4 and A5)

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

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

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

J = St. Venant torsion constant of the cross section

C_w = Torsional warping constant of the cross section

$$j = [1/(2I_y)](\int_A x^3 dA + \int_A xy^2 dA) - x_o \qquad \text{(Eq. 3.3.1.2-10)}$$

3.3.2 Strength for Shear Only

The shear force at any section shall not exceed the allowable shear, V_a, calculated as follows:

$$V_a = 2.61E_o t^3 (G_s/G_o)/h \leq 0.61F_{yv}ht \qquad \text{(Eq. 3.3.2-1)}$$

where

t = Web thickness

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

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

G_s/G_o = Plasticity reduction factor corresponding to shear stress, as given in Table A12 or in Figures A9 and A10

F_{yv} = Shear yield strength of steel as given in Table A1

Where the web consists of two or more sheets, each sheet shall be considered as a seperate element carrying its share of the shear force.

3.3.3 Strength for Combined Bending and Shear

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

$$(M/M_a)^2 + (V/V_a)^2 \leq 1.0 \qquad \text{(Eq. 3.3.3-1)}$$

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

$$0.6(M/M_a) + (V/V_a) \leq 1.3 \qquad \text{(Eq. 3.3.3-2)}$$

In the above equations:

M_a = Allowable moment when bending alone exists based on Section 3.3.1.1 excluding lateral buckling

V_a = Allowable shear force 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 components thereof, acting perpendicular to the longitudinal axis of the member, acting 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, concentrated loads and reactions shall not exceed the values of P_a given in Table 1.

The formulas in Table 1 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_a represents the concentrated load or reaction for one solid web connecting top and bottom flanges. For two or more webs, P_a shall be computed for each individual web and the results added to obtain the allowable load or reaction for the multiple web.

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

TABLE 1

Allowable Web Crippling Strength , P_a

		Shapes Having Single Webs		I-sections [1]
		Stiffened Flanges	Unstiffened Flanges	Stiffened and Unstiffened Flanges
Opposing Loads Spaced > 1.5h [2]	End Reaction [3]	Eq. 3.3.4-1	Eq. 3.3.4-2	Eq. 3.3.4-3
	Interior Reaction [4]	Eq. 3.3.4-4	Eq. 3.3.4-4	Eq. 3.3.4-5
Opposing Loads Spaced ≤ 1.5h [5]	End Reaction [3]	Eq. 3.3.4-6	Eq. 3.3.4-6	Eq. 3.3.4-7
	Interior Reaction [4]	Eq. 3.3.4-8	Eq. 3.3.4-8	Eq. 3.3.4-9

Footnotes and Equation References to Table 1:

(1) I-sections made of two channels connected back to back or similar sections which provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a channel).

(2) At locations of one concentrated load or reaction acting either on the top or bottom flange, when the clear distance between the bearing edges of this and adjacent opposite concentrated loads or reactions is greater than 1.5h.

(3) For end reactions of beams or concentrated loads on the end of cantilevers when the distance from the edge of the bearing to the end of the beam is less than 1.5h.

(4) For reactions and concentrated loads when the distance from the edge of bearing to the end of the beam is equal to or greater than 1.5h.

(5) At locations of two opposite concentrated loads or of a concentrated load and an opposite reaction acting simultaneously on the top and bottom flanges, when the clear distance between their adjacent bearing edges is equal to or less than 1.5h.

Equations for Table 1:

$$t^2C_3C_4C_\theta(165-0.30(h/t))(1+0.01(N/t)) \quad \text{(Eq. 3.3.4-1)}$$

$$t^2C_3C_4C_\theta(108-0.14(h/t))(1+0.01(N/t)) \quad \text{(Eq. 3.3.4-2)}$$

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

$$t^2F_yC_6(4.55+0.57\sqrt{N/t}) \quad \text{(Eq. 3.3.4-3)}$$

$$t^2C_1C_2C_\theta(269-0.37(h/t))(1+0.007(N/t)) \quad \text{(Eq. 3.3.4-4)}$$

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

$t^2 F_y C_5 (0.88+0.12m)(6.82+1.48\sqrt{N/t})$ (Eq. 3.3.4-5)

$t^2 C_3 C_4 C_\theta (122-0.29(h/t))(1+0.01(N/t))$ (Eq. 3.3.4-6)

$t^2 F_y C_8 (0.64+0.31m)(4.55+0.57\sqrt{N/t})$ (Eq. 3.3.4-7)

$t^2 C_1 C_2 C_\theta (385-1.13(h/t))(1+0.0013(N/t))$ (Eq. 3.3.4-8)

$t^2 F_y C_7 (0.82+0.15m)(6.82+1.48\sqrt{N/t})$ (Eq. 3.3.4-9)

In the above-referenced formulas,

P_a = Allowable concentrated load or reaction per web, kips

$C_1 = (1.22-0.22k)k$, when $F_y \leq 91.5$ ksi (Eq. 3.3.4-10)

$= 1.69$, when $F_y > 91.5$ ksi (Eq. 3.3.4-11)

$C_2 = (1.06-0.66R/t) \leq 1.0$ (Eq. 3.3.4-12)

$C_3 = (1.33-0.33k)k$, when $F_y \leq 66.5$ ksi (Eq. 3.3.4-13)

$= 1.34$, when $F_y > 66.5$ ksi (Eq. 3.3.4-14)

$C_4 = (1.15-0.15R/t) \leq 1.0$ but not less than 0.5 (Eq. 3.3.4-15)

$C_5 = (1.49-0.53k) \geq 0.6$ (Eq. 3.3.4-16)

$C_6 = 1 + (h/t)/750$, when $h/t \leq 150$ (Eq. 3.3.4-17)

$= 1.20$, when $h/t > 150$ (Eq. 3.3.4-18)

$C_7 = 1/k$, when $h/t \leq 66.5$ (Eq. 3.3.4-19)

$= (1.10 - (h/t)/665)/k$, when $h/t > 66.5$ (Eq. 3.3.4-20)

$C_8 = (0.98 - (h/t)/865)/k$ (Eq. 3.3.4-21)

$C_\theta = 0.7 + 0.3(\theta/90)^2$ (Eq. 3.3.4-22)

F_y = Longitudinal compressive yield strength of the web, ksi

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

$k = F_y/33$ (Eq. 3.3.4-23)

$m = t/0.075$ (Eq. 3.3.4-24)

t = Web thickness, inches

N = Actual length of bearing, inches. For the case of two equal and opposite concentrated loads distributed over unequal bearing lengths, the smaller value of N shall be taken

R = Inside bend radius

θ = Angle between the plane of the web and the plane of the bearing surface $\geq 45^{\circ}$, but not more than 90°.

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:

For shapes having single unreinforced webs:

$$1.2(P/P_a) + (M/M_a) \leq 1.5 \qquad \text{(Eq. 3.3.5-1)}$$

Exception: At the interior supports of continuous spans, the above formula is not applicable to deck or beams with two or more single webs, provided the compression edges of adjacent webs are laterally supported in the negative moment region by continuous or intermittently connected flange elements, rigid cladding, or lateral bracing, and the spacing between adjacent webs does not exceed 10 inches.

For shapes having multiple unreinforced webs such as I-sections made of two channels connected back-to-back, or similar sections which provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a channel);

$$1.1(P/P_a) + (M/M_a) \leq 1.5 \qquad \text{(Eq. 3.3.5-2)}$$

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

In the above formulas,

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

P_a = Allowable concentrated load or reaction in the absence of bending moment determined in accordance with Section 3.3.4

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

M_a = Allowable bending moment if bending alone exists based on Section 3.3.1.1 excluding lateral buckling

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

t = Thickness of the web or flange

λ = Slenderness factor 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.

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

$$P_a = P_n/\Omega_c \qquad \text{(Eq. 3.4-1)}$$

where

$$P_n = A_e F_n \qquad \text{(Eq. 3.4-2)}$$

A_e = Effective area calculated at the stress F_n

Ω_c = Factor of safety for axial compression

= 2.15

F_n is the least of the flexural, torsional and torsional-flexural buckling stress determined according to Sections 3.4.1 through 3.4.4.

(b) For C- and Z-shapes, and single-angle sections with unstiffened flanges, P_n shall be taken as the smaller of P_n calculated above and P_n calculated as follows:

$$P_n = AF_{cr} \qquad \text{(Eq. 3.4-3)}$$

where

A = Area of the full, unreduced cross section

F_{cr} = Critical buckling stress

$$= \frac{\Pi^2 k\eta E_o}{12(1-\mu^2)(w/t)^2} \qquad \text{(Eq. 3.4-4)}$$

η = Plasticity reduction factor corresponding to the compression stress

= E_s/E_o, for unstiffened compression elements as given in Tables A8 and A9 or Figures A5 and A6

k = Plate buckling coefficient

= 0.5 for unstiffened compression elements

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

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

w = Flat width of the unstiffened element

t = Thickness of the unstiffened element

(c) When local distortions in stiffened elements of compression members under service loads must be limited, the allowable load calculated in accordance with Eq. (3.4-1) shall not exceed the permissible load determined by the compressive stresses, f_b, and the area of the full, unreduced cross section, A, as follows:

$$P_{ld} = Af_b \qquad \text{(Eq. 3.4-5)}$$

(i) If small, barely perceptible amounts of local distortions are allowed:

$$f_b = 1.2\ F_{cr} \qquad \text{(Eq. 3.4-6)}$$

(ii) If no local distortions are permissible:

$$f_b = 0.9\ F_{cr} \qquad \text{(Eq. 3.4-7)}$$

where

F_{cr} = Critical buckling stress determined by Eq. (3.4-4)

η = Plasticity reduction factor corresponding to the compression stress

= $\sqrt{E_t/E_o}$, for stiffened compression elements as given in Tables A6 and A7 or Figures A3 and A4

k = Plate buckling coefficient for stiffened elements as defined in Section 2

E_o, μ, w, and t are as defined above.

(d) Angle sections shall be designed for the applied axial load, P, acting simutaneously with a moment equal to PL/1000 applied about the minor principal axis causing compression in the tips of the angle legs.

(e) 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:

$$F_n = \frac{\Pi^2 E_t}{(KL/r)^2} \leq F_y \qquad \text{(Eq. 3.4.1-1)}$$

where

E_t = Tangent modulus in compression corresponding to the buckling stress, Tables A13 and A14 or Figures A11 and A12

K = Effective length factor*

L = Unbraced length of member

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

* In frames where lateral stability is provided by diagonal bracing, shear walls, attachment to an adjacent structure having adequate 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, shal 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.

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:

$$F_n = \sigma_t \le F_y \qquad \text{(Eq. 3.4.2-1)}$$

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:

$$F_n = [1/(2\beta)][(\sigma_{ex}+\sigma_t)-\sqrt{(\sigma_{ex}+\sigma_t)^2-4\beta\sigma_{ex}\sigma_t}\,] \le F_y \qquad \text{(Eq. 3.4.3-1)}$$

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

$$F_n = \sigma_t\sigma_{ex}/(\sigma_t+\sigma_{ex}) \le F_y \qquad \text{(Eq. 3.4.3-2)}$$

where σ_t and σ_{ex} are defined in Section 3.3.1.2, and

$$\beta = 1-(x_o/r_o)^2 \qquad \text{(Eq. 3.4.3-3)}$$

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 axial force and bending moment shall satisfy the following interaction equations:

$$P/P_a + C_{mx}M_x/(M_{ax}\alpha_x) + C_{my}M_y/(M_{ay}\alpha_y) \le 1.0 \qquad \text{(Eq. 3.5-1)}$$

$$P/P_{ao} + M_x/M_{axo} + M_y/M_{ayo} \le 1.0 \qquad \text{(Eq. 3.5-2)}$$

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

$$P/P_a + M_x/M_{ax} + M_y/M_{ay} \le 1.0 \qquad \text{(Eq. 3.5-3)}$$

where

P = Applied axial load

M_x and M_y = Applied moments with respect to the centroidal axes of the effective section determined for the axial load alone. For angle sections, M_y shall be taken either as the applied moment or the applied moment plus PL/1000 whichever results in a lower value of P_a.

P_a = Allowable axial load determined in accordance with Section 3.4

P_{ao} = Allowable axial load determined in accordance with Section 3.4, with $F_n = F_y$

M_{ax} and M_{ay} = Allowable moments about the centroidal axes determined in accordance with Section 3.3

M_{axo} and M_{ayo} = Allowable moments about the centroidal axes determined in accordance with Section 3.3.1, excluding the provisions of Section 3.3.1.2(lateral buckling)

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

$$= 1/[1-(\Omega_c P/P_{cr})] \qquad \text{(Eq. 3.5-4)}$$

Ω_c = Factor of safety used in determining P_a

P_{cr} = $\Pi^2 E_t I_b/(K_b L_b)^2$ (Eq. 3.5-5)

I_b = Moment of inertia of the full, unreduced cross section about the axis of bending

L_b = Actual unbraced length in the plane of bending

K_b = Effective length factor in the plane of bending

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

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

   $C_m = 0.85$

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

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

where

   M_1/M_2 is the ratio of the smaller to the larger moment at the ends of that portion of the member under consideration which is unbraced in the plane of bending. M_1/M_2 is positive when the member is bent in reverse curvature and negative when it is bent in single curvature.

3. For compression members in frames braced against joint translation in the plane of loading and subject to transverse loading between their supports, the value of C_m may be determined by rational analysis. However, in lieu of such analysis, the following values may be used:

(a) for members whose ends are restrained, $C_m = 0.85$

(b) for members whose ends are unrestrained, $C_m = 1.0$

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 actual moment uncoupled from axial load, shear, and local concentrated force or reaction shall not exceed M_a calculated as follows:

$$M_a = M_n/\Omega_f \qquad \text{(Eq. 3.6.1-1)}$$

where

M_n = Nominal moment

Ω_f = Factor of safety for bending

= 1.85

For $D/t \leq 0.112E_o/F_y$

$$M_n = F_y S_f \qquad \text{(Eq. 3.6.1-2)}$$

For $0.112E_o/F_y < D/t \leq 0.881E_o/F_y$

$$M_n = K_c F_y S_f \qquad \text{(Eq. 3.6.1-3)}$$

where

$$K_c = \frac{(1-C)}{8.93-\lambda_c}\frac{(E_o/F_y)}{(D/t)} + \frac{5.882C}{8.93-\lambda_c} \qquad \text{(Eq. 3.6.1-4)}$$

C = Ratio of the effective proportional limit-to-yield strength as given in Table A19

λ_c = 3.048C, the limiting value of $(E_o/F_y)/(D/t)$ based on the specified ratio C

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

3.6.2 Compression

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

The axial load shall not exceed P_a calculated as follows:

$$P_a = P_n/\Omega_c \qquad \text{(Eq. 3.6.2-1)}$$

where

P_n = A_eF_n

Ω_c = Factor of safety for axial compression

= 2.15

F_n = The flexural buckling stress determined according to Section 3.4.1

$$A_e = [1 - (1 - (E_t/E_o)^2)(1 - A_o/A)]A \qquad \text{(Eq. 3.6.2-2)}$$

$$A_o = K_c A \le A, \quad \text{for } D/t \le 0.881E_o/F_y \qquad \text{(Eq. 3.6.2-3)}$$

A = Area of the unreduced cross section

E_t/E_o = Plasticity reduction factor corresponding to the buckling stress, as given in Tables A10 and A11 or in Figures A7 and A8

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 allowable bending moment may be taken as $0.6F_yS_f$, where S_f is as defined in Section 3.6.1, or as M_a based on Section 3.6.1, whichever is smaller.

The allowable load carrying capacity 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

(a) For compression members:

$$s_{max} = L\, r_{cy} / 2r_I \qquad \text{(Eq. 4.1.1-1)}$$

where

s_{max} = Longitudinal spacing of connections

L = Unbraced length of compression member

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

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

(b) For flexural members:

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

In no case shall the spacing exceed the value

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

where

L = Span of beam

T_s = Strength of connection in tension

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

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

m = Distance from the shear center of one channel to the mid-plane of its web

$m = (\bar{b}t/12I_x) \times [6\bar{D}(\bar{d})^2+3(\bar{b})(\bar{d})^2-8(\bar{D})^3]$ (Eq. 4.1.1-4)

$\bar{b} = B - (t/2 + \alpha t/2)$

$\bar{d} = d - t$

$\bar{D} = \alpha(D - t/2)$

B = Flange width

d = Depth of channel or beam

D = Depth of stiffening lip

t = Thickness of channel section

α = Coefficient, for sections with stiffening lips, $\alpha = 1.0$; for sections without stiffening lips, $\alpha = 0$.

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

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

$T_s = Pm/2g$ (Eq. 4.1.1-5)

The maximum spacing of connections s_{max} depends upon the intensity of the load directly at the connection. Therefore, if uniform spacing of connections is used over the whole length of the beam, it shall be determined at the point of maximum local load intensity. In cases where this procedure would result in uneconomically close spacing, either one of the following methods may be adopted: (a) the connection spacing may be varied along the beam according to the variation of the load intensity; or (b) reinforcing cover plates may be welded to the flanges at points where concentrated loads occur. The 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 non-integral stiffener or other element shall not exceed

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

(b) $s = 1.11t\sqrt{E_t/f}$, where t is the thickness of the cover plate or sheet, f is the stress at design loads in the cover plate or sheet, and E_t is the tangent modulus in compression; nor

(c) three times the flat width, w, of the narrowest unstiffened compression element tributary to the connections, but need not be less than $1.03t\sqrt{E_o/F_y}$ if $w/t < 0.5\sqrt{E_o/F_y}$, or $1.24t\sqrt{E_o/F_y}$ if $w/t \geq 0.5\sqrt{E_o/F_y}$, unless closer spacing is required by (a) or (b) above.

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

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

4.2 Mixed Systems

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

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 (a) the top flange is connected to deck or sheathing material in such a manner as to effectively restrain deflection of the connected flange,* or (b) neither flange is so connected. When both flanges are so connected, no further bracing is required.

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

4.3.2.1 Bracing When One Flange is Connected

Channel 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.

4.3.2.1.1 Type and Spacing of Braces

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 the beam design strength times 1.85, instead of that required by Section 6.

4.3.2.2 Neither Flange Connected to Sheathing

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

(a) For a uniformly loaded beam, $P_L = 1.5K'$ times the load within a distance 0.5a each side of the brace.

(b) For concentrated loads $P_L = 1.0K'$ times the concentrated load P within a distance 0.3a each side of the brace, plus a force F determined from the following formula, for each such concentrated load P located farther than 0.3a, but not farther than 1.0a from the brace:

$$F = (1.0/0.7) \times (1 - x/a)PK' \qquad \text{(Eq. 4.3.2.2-1)}$$

In the above formulas:

For channels:

$$K' = m/d \qquad \text{(Eq. 4.3.2.2-2)}$$

where

m = Distance from shear center to mid-plane of the web, as specified in Section 4.1.1

d = Depth of channel

For Z-sections:

$$K' = I_{xy}/I_x \qquad \text{(Eq. 4.3.2.2-3)}$$

where

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

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

For channels and Z-sections:

x = Distance from concentrated load P to brace

a = Length of bracing interval

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

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

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

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 $0.089E_o/F_y$.

5. CONNECTIONS AND JOINTS

5.1 General Provisions

Connections shall be designed to transmit the maximum load in the connected member. Proper regard shall be given to eccentricity.

5.2 Welded Connections

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.

Allowable loads, P_a, for groove and fillet welds shall be determined from Sections 5.2.1 and 5.2.2, respectively. Allowable loads 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 allowable load in tension or compression for a groove weld in a butt joint, welded from one or both sides, shall be the least of the following three values, provided that an effective throat equal to or greater than the thickness of the material is consistently obtained.

$$P_a = F_y A_w/\Omega_{w1} \quad \text{(Eq. 5.2.1-1)}$$

$$P_a = F_{ua} A_w/\Omega_{w2} \quad \text{(Eq. 5.2.1-2)}$$

$$P_a = F_{xx} A_w/\Omega_{w3} \quad \text{(Eq. 5.2.1-3)}$$

where

F_y = Yield strength of connected stainless steel as given in Table A1

F_{ua} = Tensile strength of the annealed base metal as given in Table A16

F_{xx} = Tensile strength of the weld metal as given in Table A15

A_w = Cross section area of the thinner welded part

Ω_{w1} = 1.85, factor of safety for yielding

Ω_{w2} = 2.50, factor of safety for fracture of the annealed base metal

Ω_{w3} = 2.50, factor of safety for fracture of the weld metal

5.2.2 Fillet Welds

The allowable shear load, P_a, on a fillet weld in lap or T-joints shall not exceed the following:

(a) For longitudinal loading:

$$P_a = 0.60F_{ua}tL/\Omega_{w2} \qquad \text{(Eq. 5.2.2-1)}$$

$$P_a = 0.60F_{xx}t_wL/\Omega_{w3} \qquad \text{(Eq. 5.2.2-2)}$$

(b) For transverse loading:

$$P_a = 0.75F_{ua}tL/\Omega_{w2} \qquad \text{(Eq. 5.2.2-3)}$$

$$P_a = 0.75F_{xx}t_wL/\Omega_{w3} \qquad \text{(Eq. 5.2.2-4)}$$

where

L = Length of fillet weld

t = Thickness of thinnest connected sheet

t_w = Effective throat = 0.707t

F_{ua}, F_{xx}, Ω_{w2} and Ω_{w3} are defined in Section 5.2.1.

5.2.3 Resistance Welds

When Type 301, 304 or 316 sheets are joined by spot welding the allowable shear per spot shall be as follows:

TABLE 2

Allowable Shear Strength for Spot Welding

Thickness of Thinnest Outside Sheet, in.	Allowable Shear Strength* per Spot, kips		
	Annealed & 1/16 Hard	1/4 Hard	1/2 Hard
0.006	0.024	0.028	0.034
0.008	0.040	0.052	0.058
0.010	0.060	0.068	0.084
0.012	0.074	0.084	0.100
0.014	0.096	0.100	0.128
0.016	0.112	0.120	0.152
0.018	0.128	0.144	0.188
0.021	0.148	0.188	0.200
0.025	0.200	0.240	0.272
0.031	0.272	0.320	0.372
0.034	0.320	0.368	0.440
0.040	0.400	0.508	0.560
0.044	0.480	0.580	0.680
0.050	0.580	0.680	0.800
0.056	0.680	0.800	0.980
0.062	0.780	0.960	1.160
0.070	0.960	1.120	1.420
0.078	1.080	1.360	1.600
0.094	1.420	1.680	2.120
0.109	1.680	2.000	2.560
0.125	2.000	2.400	3.040

1 in. = 25.40 mm ; 1 kip = 4.448 kN

* The allowable tensile strength per spot may conservatively be taken as 25% of the allowable shear strength.

When Type 301, 304 or 316 sheets are joined by pulsation welding, the allowable shear per spot shall be as follows:

TABLE 3

Allowable Shear Strength for Pulsation Welding

Thickness of Thinnest Outside Sheet, in.	Allowable Shear Strength per Spot, kips	
	1/4 Hard	1/2 Hard
0.156	3.040	4.000
0.187	3.900	4.920
0.203	4.240	5.200
0.250	5.400	6.800

1 in. = 25.40 mm ; 1 kip = 4.448 kN

(The above values are based on "Recommended Practices for Resistance Welding," C1.1-66, American Welding Society, 1966, and are for a safety factor of 2.5. Values for intermediate thicknesses may be obtained by straight line interpolation. The values above may also be applied conservatively for Type 201. In all cases, welding shall be performed in accordance with the American Welding Society's "Recommended Practices for Resistance Welding.")

5.3 Bolted Connections

The following requirements govern bolted connections of cold-formed stainless steel structural members.

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

The holes for bolts shall not exceed the sizes specified in Table 4, except that larger holes may be used in column base or structural systems connected to concrete walls.

TABLE 4

Maximum size of Bolt Holes

Nominal Bolt Diameter (d)(in.)	Standard Hole Diameter (in.)	Oversized Hole Diameter (in.)	Short-Slotted Hole Diameter (in.)	Long-Slotted Hole Diameter (in.)
$< 1/2$	d+1/32	d+1/16	(d+1/32)by(d+1/4)	(d+1/32)by($2^1/_2$d)
$\geq 1/2$	d+1/16	d+1/8	(d+1/16)by(d+1/4)	(d+1/16)by($2^1/_2$d)

1 in. = 25.40 mm

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 distance, e, measured in the line of force from the center of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part toward which the force is directed shall not be less than the value of e_{min} determined below:

$$e_{min} = e\ \Omega_e \qquad \text{(Eq. 5.3.1-1)}$$

where

$$e = P/(F_u t) \qquad \text{(Eq. 5.3.1-2)}$$

P = Force transmitted by bolt

Ω_e = Factor of safety for sheet tearing

= 2.4

t = Thickness of thinnest connected part

F_u = Tensile strength of the connected sheet in the longitudinal direction

In addition, the minimum distance between centers of bolt holes shall provide sufficient clearance for bolt heads, nuts, washers and the wrench but shall not be less than 3 times the nominal bolt diameter, d. Also, the distance from the center of any standard hole to the end or other boundary of the connecting member shall not be less than $1^1/_2$d.

For oversized and slotted holes, the distance between edges of two adjacent holes and the distance measured from the edge of the hole to the end or other boundary of the connecting member in the line of stress shall not be less than the value of $(e_{min} - (d_h/2))$, in which e_{min} is the required distance computed from the above equation, and d_h is the diameter of a standard hole defined in Table 4. 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 tension force on the net section of a bolted connection shall not exceed T_a from Section 3.2 or P_a calculated as follows:

$$P_a = P_n/\Omega_t \quad \text{(Eq. 5.3.2-1)}$$

where

$$P_n = A_n F_t \quad \text{(Eq. 5.3.2-2)}$$

A_n = Net section area

Ω_t = Factor of safety for tension on the net section

= 2.40

$$F_t = (1.0 - 0.9r + 3r\ d/s)F_u \le F_u \quad \text{for double shear} \quad \text{(Eq. 5.3.2-3)}$$

$$F_t = (1.0 - r + 2.5\ r\ d/s)F_u \le F_u \quad \text{for single shear} \quad \text{(Eq. 5.3.2-4)}$$

where

r = The force transmitted by the bolt or bolts at the section considered, divided by the tension force in the member at that section. If r is less than 0.2, it may be taken equal to 0.

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

F_u = Tensile strength of the connected sheet in the longitudinal direction

F_t = Nominal tension stress for connections with washers under both bolt head and nut

d and t are defined in Section 5.3.1.

5.3.3 Bearing

The bearing force shall not exceed P_a calculated as follows:

$$P_a = P_n/\Omega_b \qquad \text{(Eq. 5.3.3-1)}$$

where

$$P_n = F_p dt \qquad \text{(Eq. 5.3.3-2)}$$

Ω_b = Factor of safety for bearing

= 2.40

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

(a) Single shear connections

$$F_p = 2.00F_u \qquad \text{(Eq. 5.3.3-3)}$$

(b) Double shear connections

$$F_p = 2.75F_u \qquad \text{(Eq. 5.3.3-4)}$$

d,t and F_u are defined in Section 5.3.1.

5.3.4 Shear and Tension in Bolts

The bolt force resulting from shear, tension or combination of shear and tension shall not exceed the allowable bolt force, P_a, calculated as follows (The factor of safety is included in Tables A17 and A18):

$$P_a = A_b F \qquad \text{(Eq. 5.3.4-1)}$$

where

A_b = Gross cross-sectional area of bolt

F is given by F_v, F_t, and F'_t in Tables A17 and A18.

When bolts are subject to a combination of shear and tension, the tension force shall not exceed the allowable force, P_a, based on F'_t, as given in Table A18, where f_v, the shear stress produced by the same force, shall not exceed the allowable value F_v given in Table A17.

5.4 Shear Rupture

At beam-end connectins, where one or more flanges are coped and failure might occur along a plane through the fasteners, the shear force shall not exceed the allowable shear force V_a, calculated as follows:

$$V_a = V_n/\Omega_v \qquad \text{(Eq. 5.4-1)}$$

where

$$V_n = 0.6F_u A_{wn} \qquad \text{(Eq. 5.4-2)}$$

$$A_{wn} = (d_{wc} - nd_h)t \qquad \text{(Eq. 5.4-3)}$$

d_{wc} = Coped web depth

n = Number of holes in the critical plane

d_h = Hole diameter

Ω_v = Factor of safety for shear rupture

= 2.22

F_u and t are defined in Section 5.3.1.

6. TESTS

Tests shall be made by an independent testing laboratory or by a manufacturer's testing laboratory witnessed by a qualified 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 and A240 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

"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 percent 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 procedures.

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 steel are such that calculation of their safe load-carrying capacity or deflection cannot be made in accordance with the provisions of this Specification, their structural performance shall be established from tests and evaluated in accordance with the following procedure.

(a) Where practicable, evaluation of test results shall be made on the basis of the mean values resulting from tests of not fewer than three identical specimens, provided the deviation of any individual test result from the mean value obtained from all test does not exceed $\pm$ 10 percent. If such deviation from the mean exceeds 10 percent, at least three more tests of the same kind shall be made. The average of the three lowest values of all tests made shall then be regarded as the result of the series of tests.

(b) The required load-carrying capacity shall be:

$$R = DF_D + LF_L \qquad \text{(Eq. 6.2-1)}$$

where D and L are the dead and live loads, respectively, D shall include the weight of the test specimen. F_D and F_L are the dead and live load factors specified below. R shall be taken as the largest applicable value determined as follows:

(1) The minimum load-carrying capacity, R, shall be calculated from the formula

$$R \geq 2D + 2L \qquad \text{(Eq. 6.2-2)}$$

R shall be multiplied by 1.25 for stainless steels not listed in Section 1.3.1.

R may be divided by $1^1/_3$ when the loading consists of wind or earthquake loads alone, or in combination with dead, live, or snow loads, but shall not be less than R calculated for the combination of dead and live loads only, without wind or earthquake loads.

(2) The load at which distortions interfere with the proper functioning of the specimen in actual use shall not be less than:

$$R \geq D + 1.5L \quad \text{(Eq. 6.2-3)}$$

(3) The load carrying capacity when limited by connection failure shall not be less than:

$$R = 2.5D + 2.5L \quad \text{(Eq. 6.2-4)}$$

(c) 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 where tensile strength is the critical factor.

Consideration shall 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 Confirming Structural Performance

The procedure and formulas specified in Section 6.2 are not applicable to confirmation tests on specimens whose capacities can be computed according to Sections 2 through 5, where the specification provides generally a safety factor of 1.85 except in the case of arc-and-tangent corrugated sheets used for roofing, siding, and curtain walls, for which Section 3.7 permits either calculation with a safety factor of 1/0.6 or tests. A successful confirmatory test shall demonstrate a safety factor not less than that implied in the Specification for the type of behavior involved.

6.4 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 sections as follows:

(a) Tensile yield strength determinations shall be made by any of the methods described in the current edition of "Methods and Definitions for Mechanical Testing of Steel Products," ASTM Designation A370.

(b) Compressive yield strength determinations shall be made by means of compression tests of short stub column specimens of the section, and shall be taken as either the maximum compressive strength of the section or the stress determined by the 0.2 percent offset method, whichever is reached first in the test.

(c) Where the principal effect of the loading to which the member will be subjected in service will be to produce bending stresses, the yield strength to be used shall be the lower of the yield strengths 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.

(d) For acceptance and control purposes two full section tests shall be made from each lot of not more than 50 tons nor less than 30 tons of each section, or one test from each lot of less than 30 tons of each section. For this purpose a lot may be defined as that tonnage of one section that is formed in a single production run of material from one heat or below.

(e) At the option of the manufacturer, either tension or compression tests may be used for routine acceptance and control purposes, provided the manufacturer demonstrates that values measured by the test used will reliably indicate the yield strength of the section for the type of steel used, 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 A19) and figures (Figures A1 through A12) shall be used in this Specification.

TABLE A1

Yield Strength

Type of Stress	F_y, ksi				
	Types 201,301,304 and 316				Types 409, 430 and 439
	Annealed	1/16 Hard	1/4 Hard	1/2 Hard *	
Longitudinal Tension	30	40 + 45	75	110	30
Transverse Tension	30	40 + 45	75	110	30
Transverse Compression	30	40 + 45	90	120	30
Longitudinal Compression	28	36 + 41	50	65	30
Shear Yield Strength, F_{yv}	17	23 + 25	42	56	17

1 ksi = 6.895 MPa

* Types 304 and 316 stainless steels (1/2 hard temper) shall not be used for major structural members due to low ductility.

\+ Flat bars

TABLE A2

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

Stress ksi	Secant Modulus, E_s, ksix10^3					
	Longitudinal Compression			Transverse Compression		
	Annealed & 1/16 Hard	1/4 Hard	1/2 Hard	Annealed & 1/16 Hard	1/4 Hard	1/2 Hard
0	28.0	27.0	27.0	28.0	28.0	28.0
4	28.0	27.0	27.0	28.0	28.0	28.0
8	28.0	27.0	27.0	28.0	28.0	28.0
12	28.0	27.0	27.0	28.0	28.0	28.0
16	24.8	27.0	27.0	28.0	28.0	28.0
20	21.3	27.0	27.0	28.0	28.0	28.0
24	18.5	26.2	26.7	27.7	28.0	28.0
28		24.0	25.4		28.0	28.0
32		21.3	24.2		28.0	28.0
36		18.8	23.0		27.9	28.0
40		16.9	21.8		27.8	28.0
44		15.3	20.6		27.4	28.0
48		13.9	19.4		27.0	28.0
52		12.5	18.2		26.4	27.9
56			17.1			27.6
60			16.0			27.2
64			15.0			26.8
68			14.0			26.4

1 ksi = 6.895 MPa

TABLE A2 (cont'd)

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

Stress ksi	Secant Modulus, E_s, ksix10^3					
	Longitudinal Tension			Transverse Tension		
	Annealed & 1/16 Hard	1/4 Hard	1/2 Hard	Annealed & 1/16 Hard	1/4 Hard	1/2 Hard
0	28.0	27.0	27.0	28.0	28.0	28.0
4	28.0	27.0	27.0	28.0	28.0	28.0
8	28.0	27.0	27.0	28.0	28.0	28.0
12	28.0	27.0	27.0	28.0	28.0	28.0
16	28.0	27.0	27.0	28.0	28.0	28.0
20	28.0	27.0	27.0	28.0	28.0	28.0
24	27.1	27.0	27.0	25.6	28.0	28.0
28		27.0	27.0		28.0	28.0
32		26.8	27.0		28.0	28.0
36		26.1	27.0		27.9	28.0
40		25.4	27.0		27.4	28.0
44		24.6	26.8		26.8	28.0
48		23.8	26.6		25.9	28.0
52		22.9	26.4		25.0	27.9
56			26.1			27.6
60			25.7			27.2
64			25.3			26.7
68			24.9			26.2

1 ksi = 6.895 MPa

TABLE A3

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

Stress	Secant Modulus, E_s, ksix10^3							
	Type 409				Types 430 and 439			
ksi	Long. Comp.	Tran. Comp.	Long. Ten.	Tran. Ten.	Long. Comp.	Tran. Comp.	Long. Ten.	Tran. Ten.
0	27.0	29.0	27.0	29.0	27.0	29.0	27.0	29.0
4	27.0	29.0	27.0	29.0	27.0	29.0	27.0	29.0
8	27.0	29.0	27.0	29.0	27.0	29.0	27.0	29.0
12	27.0	29.0	27.0	29.0	26.6	29.0	26.9	29.0
16	26.8	29.0	26.9	29.0	25.3	29.0	26.6	29.0
20	25.6	28.9	26.1	28.9	22.2	28.7	24.8	28.7
24	21.5	27.1	22.4	27.1	17.3	26.4	20.1	26.3
28	13.6	17.1	14.1	17.1	12.0	16.4	13.0	16.3
32	6.5	4.8	6.2	4.8	7.7	5.2	6.9	5.3

1 ksi = 6.895 MPa

Note: Long. Comp. = Longitudinal Compression
Tran. Comp. = Transverse Compression
Long. Ten. = Longitudinal Tension
Tran. Ten. = Transverse Tension

TABLE A4

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

Type / Modulus	Annealed & 1/16 Hard	1/4 Hard and 1/2 Hard	
	Longitudinal & Transverse Tension and Compression	Longitudinal Tension and Compression	Transverse Tension and Compression
Initial Modulus of Elasticity, E_o, ksix10^3	28.0	27.0	28.0
Initial Shear Modulus, G_o, ksix10^3	10.8	10.5	10.8

1 ksi = 6.895 MPa

TABLE A5

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

Type / Modulus	Longitudinal Tension and Compression	Transverse Tension and Compression
Initial Modulus of Elasticity, E_o, ksix10^3	27.0	29.0
Initial Shear Modulus, G_o, ksix10^3	10.5	11.2

1 ksi = 6.895 MPa

TABLE A6

Plasticity Reduction Factors for Stiffened Elements
(Types 201, 301, 304 and 316)

Stress ksi	$\sqrt{E_t/E_o}$ Longitudinal Compression Annealed & 1/16 Hard	1/4 Hard	1/2 Hard	Transverse Compression Annealed & 1/16 Hard	1/4 Hard	1/2 Hard
0	1.00	1.00	1.00	1.00	1.00	1.00
4	1.00	1.00	1.00	1.00	1.00	1.00
8	1.00	1.00	1.00	1.00	1.00	1.00
12	1.00	1.00	1.00	1.00	1.00	1.00
16	0.77	1.00	1.00	1.00	1.00	1.00
20	0.67	1.00	1.00	1.00	1.00	1.00
24	0.58	0.79	0.98	0.81	1.00	1.00
28	0.50	0.71	0.86	0.62	1.00	1.00
32		0.65	0.80		1.00	1.00
36		0.60	0.75		1.00	1.00
40		0.56	0.71		0.99	1.00
44		0.52	0.68		0.93	1.00
48		0.48	0.64		0.88	1.00
52		0.45	0.60		0.84	1.00
56			0.57		0.80	0.97
60			0.53		0.77	0.94
64			0.50		0.73	0.91
68			0.47		0.70	0.87

1 ksi = 6.895 MPa

TABLE A7

Plasticity Reduction Factors for Stiffened Elements
(Types 409, 430 and 439)

Stress	$\sqrt{E_t/E_o}$			
	Type 409		Types 430 and 439	
ksi	Longitudinal Compression	Transverse Compression	Longitudinal Compression	Transverse Compression
0	1.00	1.00	1.00	1.00
4	1.00	1.00	1.00	1.00
8	1.00	1.00	0.99	1.00
12	0.99	1.00	0.96	1.00
16	0.96	1.00	0.84	0.99
20	0.81	0.96	0.66	0.94
24	0.54	0.69	0.47	0.64
28	0.30	0.28	0.33	0.28
32	0.17	0.10	0.24	0.14

1 ksi = 6.895 MPa

TABLE A8

Plasticity Reduction Factors for Unstiffened Elements
(Types 201, 301, 304 and 316)

Stress ksi	E_s/E_o Longitudinal Compression Annealed & 1/16 Hard	1/4 Hard	1/2 Hard	Transverse Compression Annealed & 1/16 Hard	1/4 Hard	1/2 Hard
0	1.00	1.00	1.00	1.00	1.00	1.00
4	1.00	1.00	1.00	1.00	1.00	1.00
8	1.00	1.00	1.00	1.00	1.00	1.00
12	1.00	1.00	1.00	1.00	1.00	1.00
16	0.89	1.00	1.00	1.00	1.00	1.00
20	0.76	1.00	1.00	1.00	1.00	1.00
24	0.66	0.97	0.99	0.99	1.00	1.00
28	0.57	0.89	0.94	0.86	1.00	1.00
32	0.46	0.79	0.90	0.70	1.00	1.00
36	0.35	0.70	0.85	0.49	1.00	1.00
40	0.23	0.63	0.81	0.28	0.99	1.00
44	0.12	0.57	0.76	0.08	0.98	1.00
48		0.51	0.72		0.96	1.00
52		0.46	0.67		0.94	0.99
56			0.63		0.92	0.98
60			0.59		0.89	0.97
64			0.56		0.86	0.96
68			0.52		0.83	0.94
72					0.80	0.92
76					0.76	0.91
80					0.72	0.89
84					0.68	0.87
88					0.64	0.86
92					0.59	0.84
96						0.82
100						0.80
104						0.78
108						0.76
112						0.74
116						0.71
120						0.68

1 ksi = 6.895 MPa

TABLE A9

Plasticity Reduction Factors for Unstiffened Elements
(Types 409, 430 and 439)

Stress	E_s/E_o			
	Type 409		Types 430 and 439	
ksi	Longitudinal Compression	Transverse Compression	Longitudinal Compression	Transverse Compression
0	1.00	1.00	1.00	1.00
4	1.00	1.00	1.00	1.00
8	1.00	1.00	1.00	1.00
12	1.00	1.00	0.99	1.00
16	0.99	1.00	0.94	1.00
20	0.95	1.00	0.82	0.99
24	0.79	0.93	0.64	0.91
28	0.50	0.59	0.44	0.56
32	0.24	0.17	0.28	0.18

1 ksi = 6.895 MPa

TABLE A10

Plasticity Reduction Factors for Lateral Buckling Strength
(Types 201, 301, 304 and 316)

Stress ksi	E_t/E_o Longitudinal Compression Annealed & 1/16 Hard	1/4 Hard	1/2 Hard	Transverse Compression Annealed & 1/16 Hard	1/4 Hard	1/2 Hard
0	1.00	1.00	1.00	1.00	1.00	1.00
4	1.00	1.00	1.00	1.00	1.00	1.00
8	1.00	1.00	1.00	1.00	1.00	1.00
12	1.00	1.00	1.00	1.00	1.00	1.00
16	0.60	1.00	1.00	1.00	1.00	1.00
20	0.45	1.00	1.00	1.00	1.00	1.00
24	0.34	0.63	0.96	0.66	1.00	1.00
28	0.25	0.50	0.74	0.38	1.00	1.00
32	0.16	0.42	0.64	0.21	1.00	1.00
36	0.10	0.36	0.56	0.09	1.00	1.00
40	0.05	0.31	0.51	0.04	0.98	1.00
44	0.01	0.27	0.46	0.02	0.86	1.00
48		0.23	0.41		0.78	1.00
52		0.20	0.36		0.71	1.00
56		0.18	0.33		0.65	0.94
60		0.15	0.29		0.59	0.88
64		0.13	0.25		0.54	0.82
68		0.11	0.22		0.49	0.77
72		0.10	0.19		0.44	0.73
76			0.17		0.39	0.68
80			0.16		0.34	0.64
84			0.14		0.29	0.60
88			0.13		0.25	0.56
92			0.12		0.20	0.53
96			0.11		0.16	0.49
100			0.11		0.13	0.46
104					0.10	0.43
108					0.07	0.39
112						0.36
116						0.32
120						0.29
124						0.26
128						0.23
132						0.20
136						0.16
140						0.13
144						0.10
148						0.07

1 ksi = 6.895 MPa

TABLE A11

Plasticity Reduction Factors for Lateral Buckling Strength (Types 409, 430 and 439)

Stress	E_t/E_o			
	Type 409		Types 430 and 439	
ksi	Longitudinal Compression	Transverse Compression	Longitudinal Compression	Transverse Compression
0	1.00	1.00	1.00	1.00
4	1.00	1.00	1.00	1.00
8	1.00	1.00	0.99	1.00
12	0.99	1.00	0.92	1.00
16	0.93	1.00	0.71	0.99
20	0.66	0.93	0.43	0.89
24	0.29	0.47	0.22	0.41
28	0.09	0.08	0.11	0.08
32	0.03	0.01	0.06	0.02

1 ksi = 6.895 MPa

TABLE A12

Plasticity Reduction Factors for Shear Strength

Stress ksi	G_s/G_o Types 201, 301, 304 and 316: Annealed & 1/16 Hard	1/4 Hard	1/2 Hard	Type 409	Types 430 and 439
0	1.00	1.00	1.00	1.00	1.00
4	1.00	1.00	1.00	1.00	1.00
8	0.98	1.00	1.00	1.00	0.99
12	0.84	1.00	1.00	0.83	0.70
16	0.58	0.98	1.00		
20	0.24	0.95	0.99		
24	0.03	0.90	0.97		
28		0.85	0.95		
32		0.78	0.93		
36		0.70	0.89		
40		0.61	0.85		
44		0.51	0.81		
48			0.77		
52			0.71		
56			0.65		

1 ksi = 6.895 MPa

TABLE A13

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

Stress ksi	Tangent Modulus, E_t, ksix10^3					
	Longitudinal Compression			Transverse Compression		
	Annealed & 1/16 Hard	1/4 Hard	1/2 Hard	Annealed & 1/16 Hard	1/4 Hard	1/2 Hard
0	28.0	27.0	27.0	28.0	28.0	28.0
4	28.0	27.0	27.0	28.0	28.0	28.0
8	28.0	27.0	27.0	28.0	28.0	28.0
12	28.0	27.0	27.0	28.0	28.0	28.0
16	16.7	27.0	27.0	28.0	28.0	28.0
20	12.5	27.0	27.0	28.0	28.0	28.0
24	9.5	17.0	26.0	18.5	28.0	28.0
28	7.0	13.5	20.0	10.7	28.0	28.0
32	4.6	11.3	17.2	5.9	28.0	28.0
36	2.7	9.7	15.2	2.5	28.0	28.0
40	1.4	8.4	13.7	1.2	27.3	28.0
44	0.4	7.2	12.4	0.6	24.0	28.0
48		6.3	11.0		21.7	28.0
52		5.5	9.8		19.9	28.0
56			8.8		18.1	26.4
60			7.7		16.6	24.5
64			6.8		15.1	23.0
68			6.0		13.7	21.5
72					12.3	20.4
76					10.9	19.1
80					9.5	18.0
84					8.2	16.9
88					6.9	15.8
92					5.7	14.8
96						13.8
100						12.8
104						11.9
108						10.9
112						10.0
116						9.0
120						8.1

1 ksi = 6.895 MPa

TABLE A14

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

Stress ksi	Tangent Modulus, E_t, ksix10^3			
	Type 409		Types 430 and 439	
	Longitudinal Compression	Transverse Compression	Longitudinal Compression	Transverse Compression
0	27.0	29.0	27.0	29.0
4	27.0	29.0	27.0	29.0
8	27.0	29.0	26.7	29.0
12	26.8	29.0	24.7	29.0
16	25.1	28.9	19.1	28.8
20	17.8	26.9	11.5	25.8
24	7.7	13.6	6.0	12.0
28	2.6	2.4	3.1	2.4
32	0.9	0.4	1.6	0.4

1 ksi = 6.895 MPa

TABLE A15

Tensile Strength of Weld Metal

AWS Classification	Tensile Strength, min. ksi
E209	100
E219	90
E240	100
E307	85
E308	80
E308H	80
E308L	75
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
E410NiMo	110
E430	65
E502	60
E505	60
E630	135
E16-8-2	80
E7Cr	60

1 ksi = 6.895 MPa

TABLE A16

Tensile Strength of Annealed Base Metal

Type	Product	Minimum Tensile Strength, ksi
201	Plate,Sheet,Strip & Flat Bar	90 (Class 1) 95 (Class 2)
301	Plate,Sheet,Strip & Flat Bar	90
304,316	Plate,Sheet,Strip & Flat Bar	75
409	Plate,Sheet,Strip	55
430	Plate,Sheet,Strip	65
439	Plate,Sheet,Strip	65

1 ksi = 6.895 MPa

TABLE A17

Allowable Shear and Tension Stresses in Bolts

Type	Diameter d (in.)	Minimum Tensile Requirements		Allowable Shear Stress, F_v, (ksi)		Allowable Tension Stress, F_t, (ksi)
		0.2% Yield Strength (ksi)	Tensile Strength (ksi)	No Threads in Shear Plane	Threads in Shear Plane	
201[a]	all	40.0	75.0	15.0	10.5	25.0
304,316[b]	all	30.0	75.0	15.0	10.5	25.0
304,316[c]	≤1/2	45.0	90.0	18.0	12.6	30.0
304,316[d]	≤3/4	100.0	125.0	25.0	17.5	42.0
430[a]	all	30.0	60.0	12.0	8.4	20.0

1 ksi = 6.895 MPa ; 1 in. = 25.4 mm

[a] Condition A in ASTM A276-85a, Hot- or Cold-Finished

[b] Condition A in ASTM A276-85a and Class 1 (solution treated) in ASTM A193/A193M-86, Hot-Finished

[c] Condition A in ASTM A276-85a, Cold-Finished

[d] Condition B(cold-worked) in ASTM A276-85a and Class 2 (solution treated and strain hardened) in ASTM A193/A193M-86, Cold-Finished

For Class 2: B8MN in ASTM A193/A193M-86, the allowable shear stress is 22.0 ksi when threading is excluded from the shear plane, or 15.0 ksi when threads are in the shear plane.

TABLE A18

Allowable Tension Stress, F'_t for Bolts
Subject to a Combination of Shear and Tension

Type	Threads Not Excluded from Shear Plane (ksi)	Threads Excluded from Shear Plane (ksi)
201*,304,316 Condition A, Hot-Finished	$31-1.8f_v \leq 25$	$31-1.4f_v \leq 25$
304,316 Condition A, Cold-Finished	$38-1.8f_v \leq 30$	$38-1.4f_v \leq 30$
304,316 Condition B, Cold-Finished	$53-1.8f_v \leq 42$	$53-1.4f_v \leq 42$
430* Condition A	$25-1.8f_v \leq 20$	$25-1.4f_v \leq 20$

1 ksi = 6.895 Mpa

* Hot- or Cold-Finished

TABLE A19

Ratio of the Effective Proportional Limit-to-Yield Strength

Type of Stress	Effective Proportional Limit/Yield Strength (F_{pr}/F_y)				
	Types 201, 301, 304, and 316			Type 409	Types 430 and 439
	Annealed & 1/16-Hard	1/4-Hard	1/2-Hard		
Longitudinal Tension	0.67	0.50	0.45	0.76	0.70
Transverse Tension	0.57	0.55	0.60	0.83	0.81
Transverse Compression	0.66	0.50	0.50	0.83	0.82
Longitudinal Compression	0.46	0.50	0.49	0.73	0.62

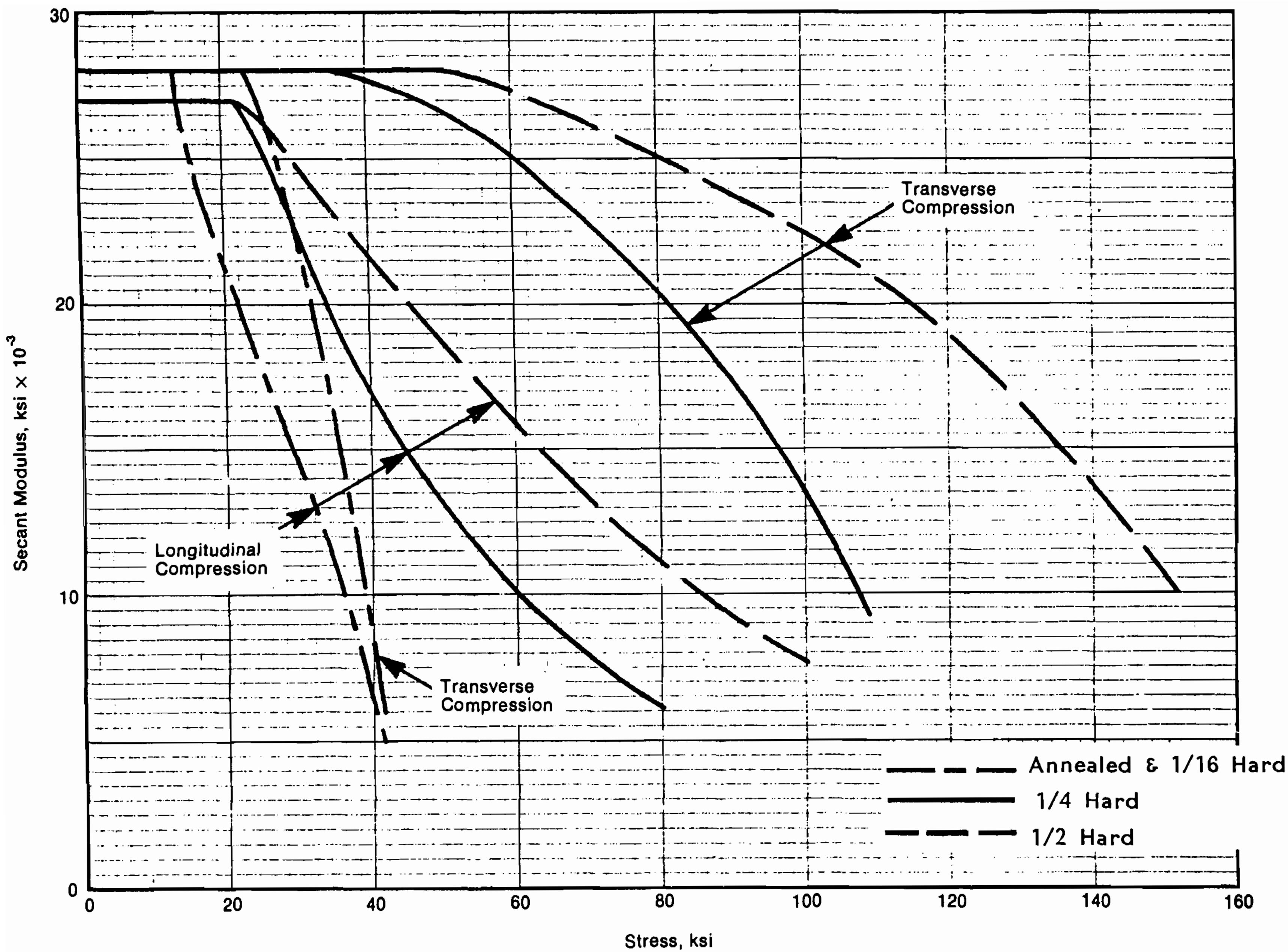

Figure A1 Secant Moduli for Deflection Calculations

(Types 201, 301, 304 and 316)

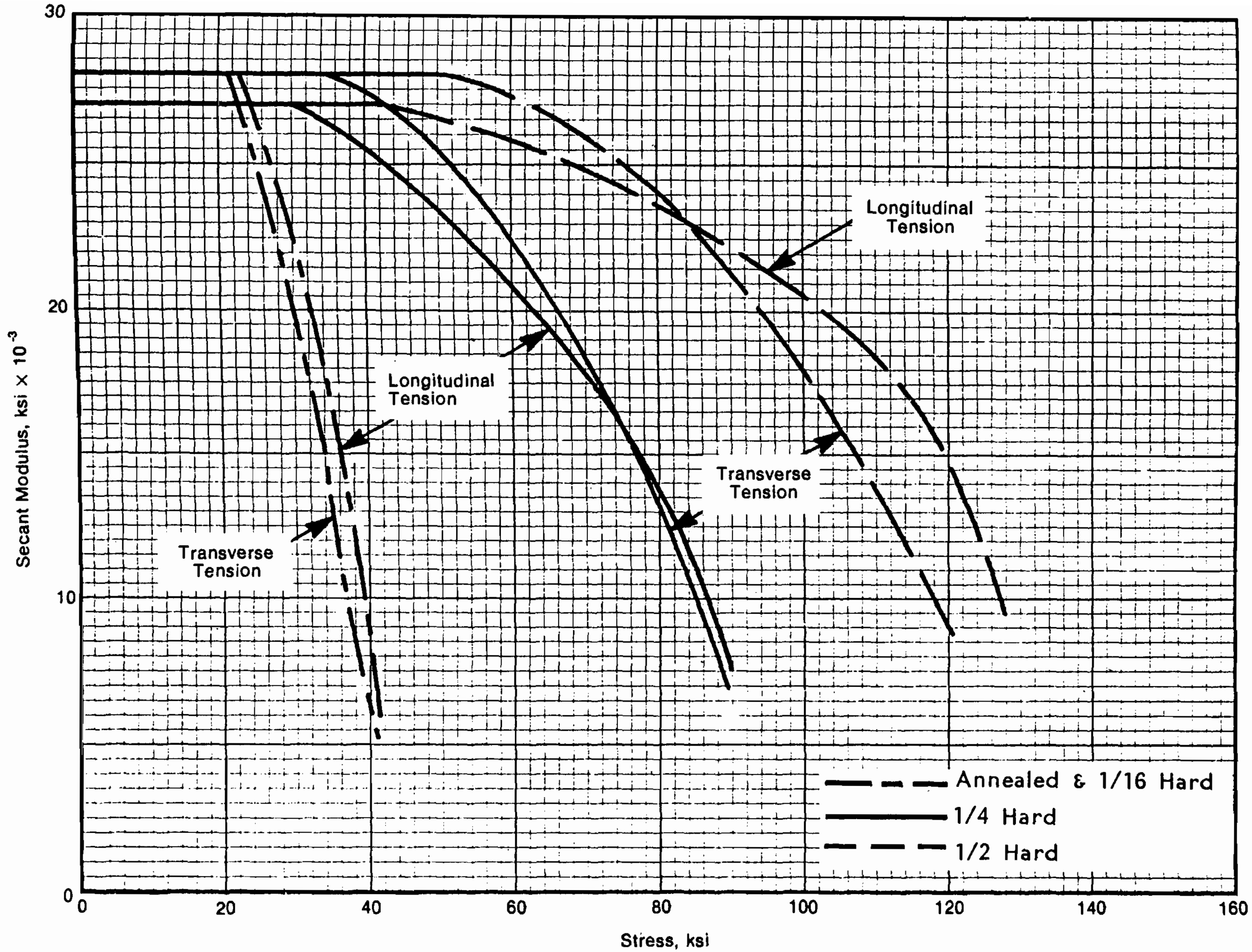

Figure A1 **Secant Moduli for Deflection Calculations** (cont'd)

(Types 201, 301, 304 and 316)

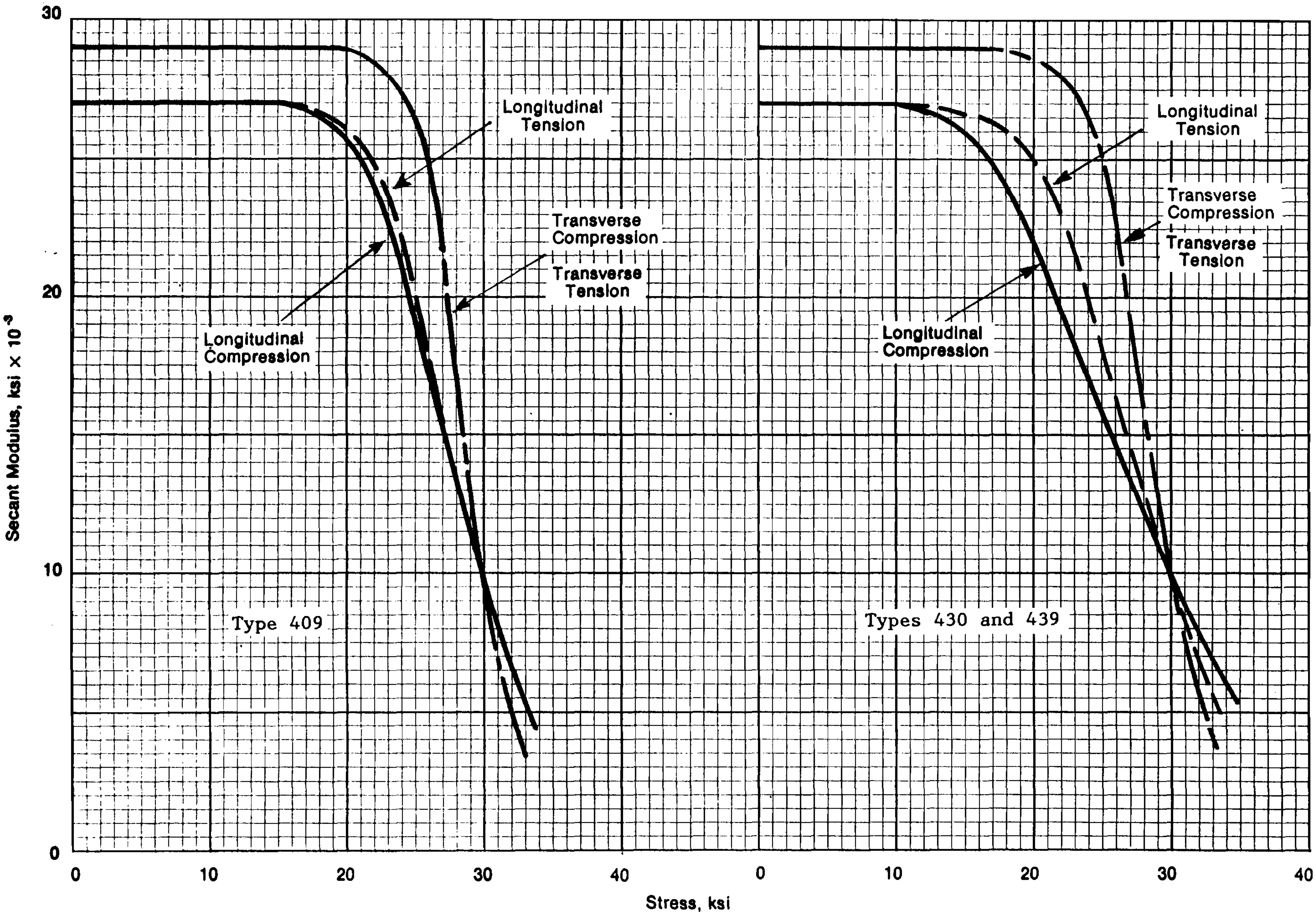

Figure A2 Secant Moduli for Deflection Calculations

(Types 409, 430 and 439)

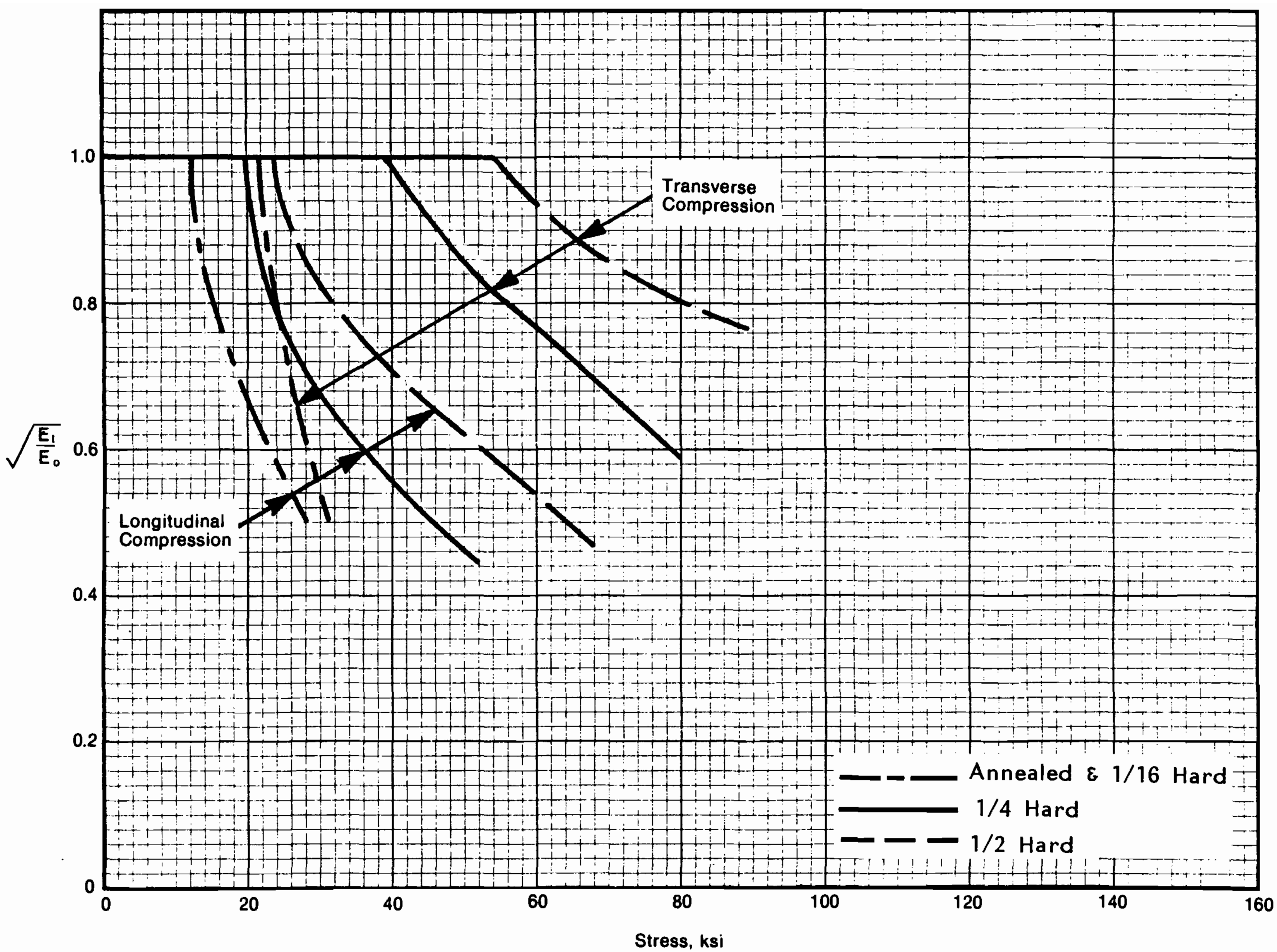

Figure A3   Plasticity Reduction Factors for Stiffened Compression Elements (Types 201, 301, 304 and 316)

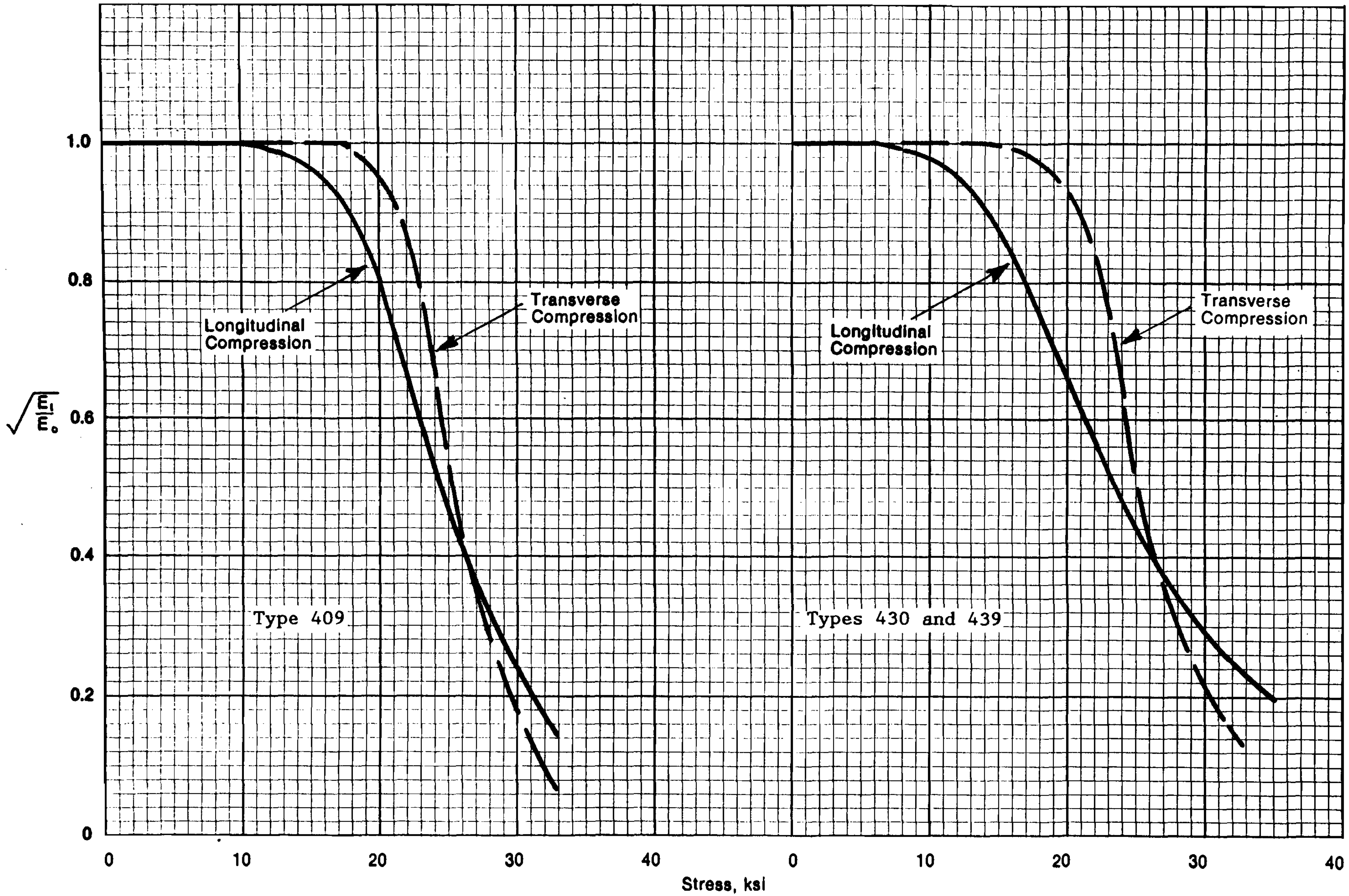

Figure A4 Plasticity Reduction Factors for Stiffened Compression Elements (Types 409, 430 and 439)

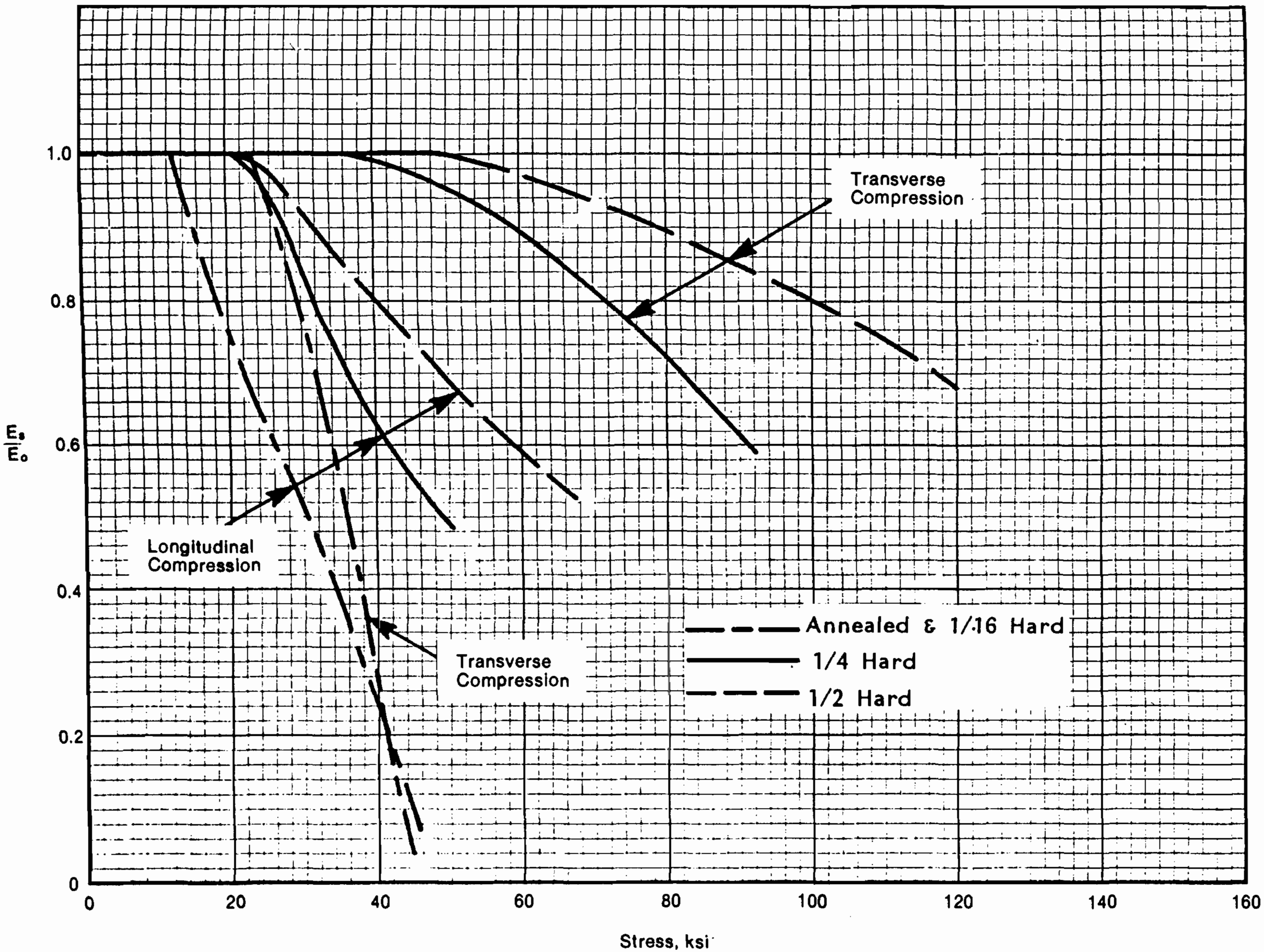

Figure A5 Plasticity Reduction Factors for Unstiffened Compression Elements (Types 201, 301, 304 and 316)

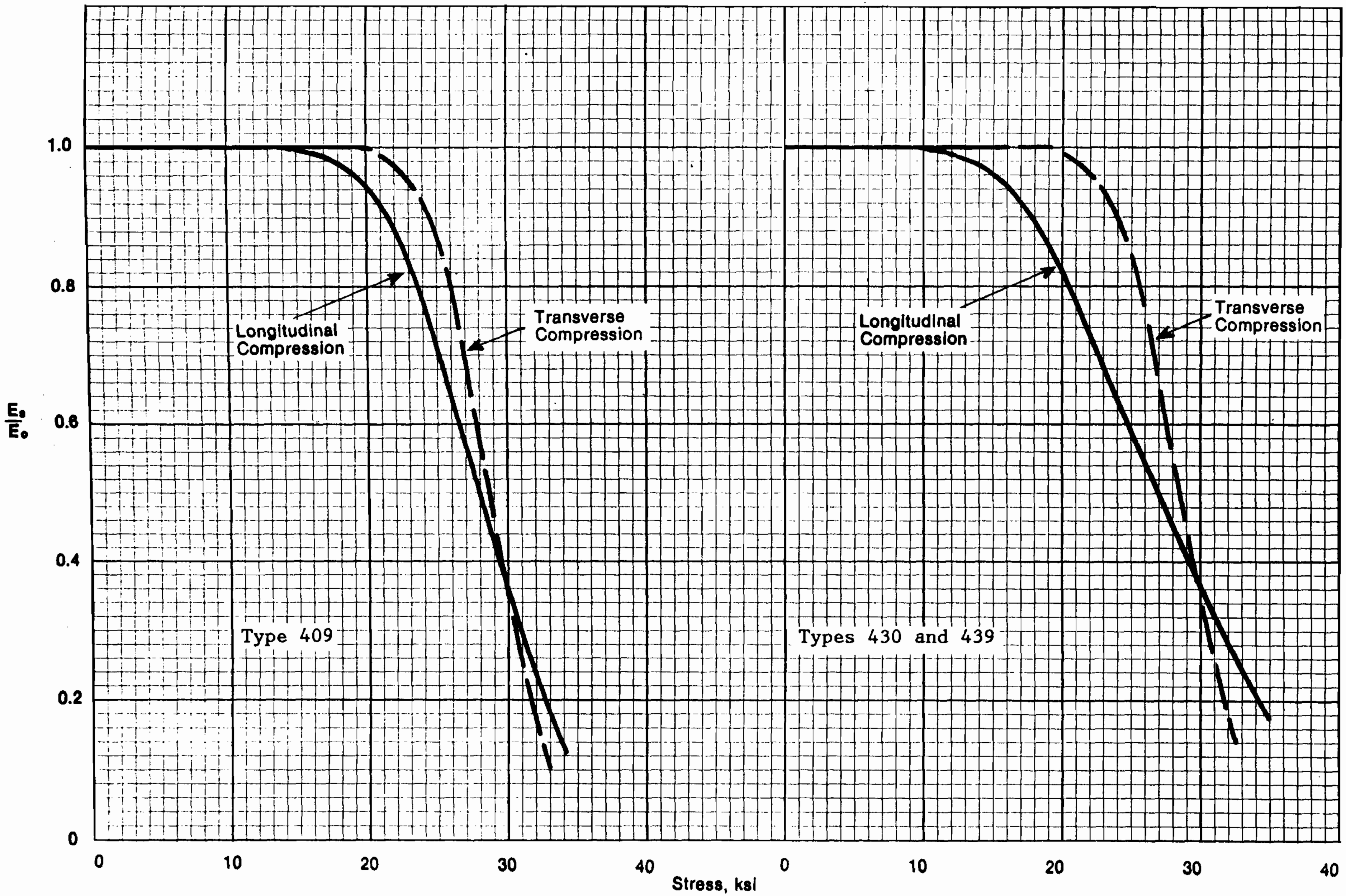

Figure A6 Plasticity Reduction Factors for Unstiffened Compression Elements (Types 409, 430 and 439)

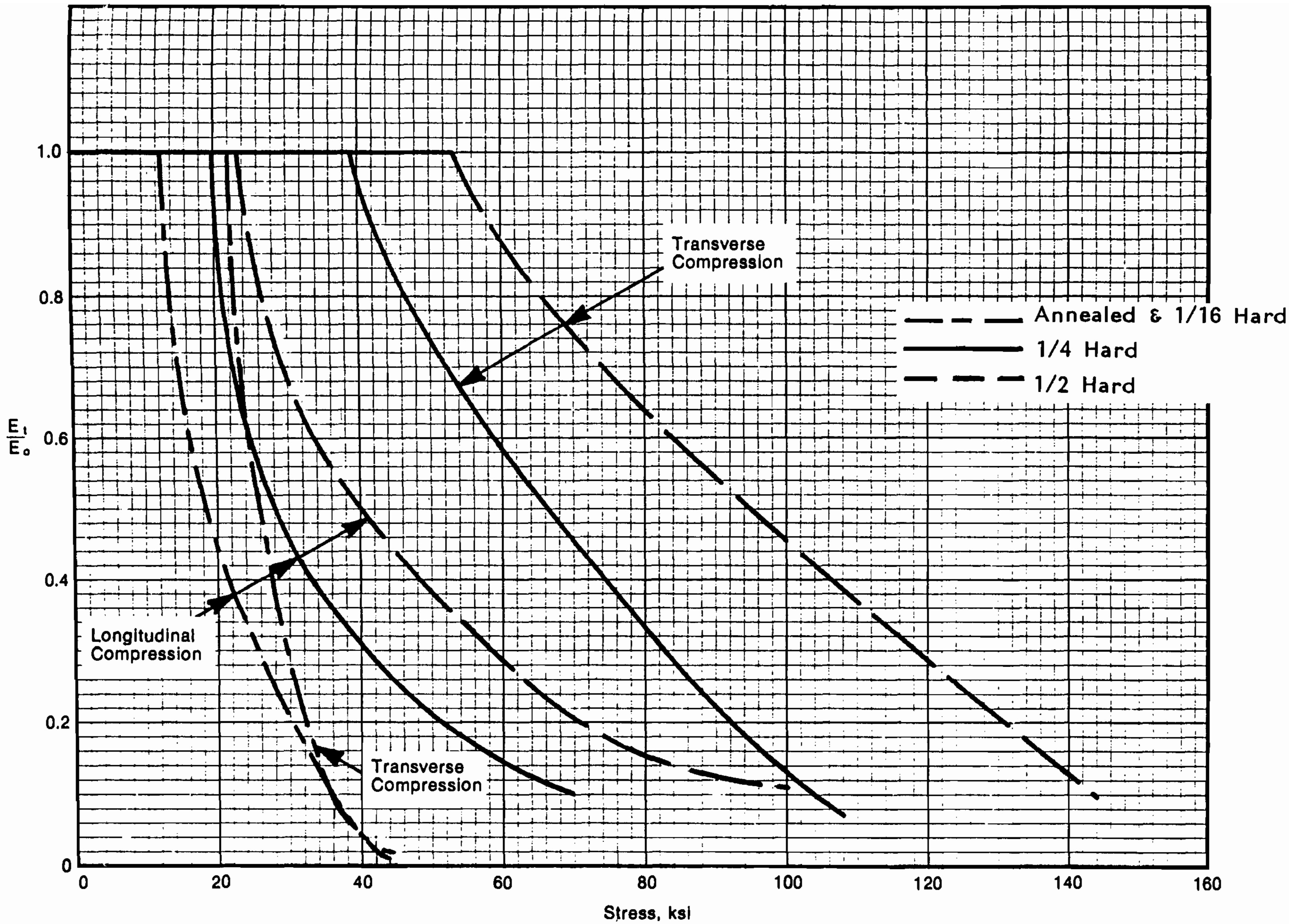

Figure A7 Plasticity Reduction Factors for Design of Laterally Unbraced Single Web Beams (Types 201, 301, 304 and 316)

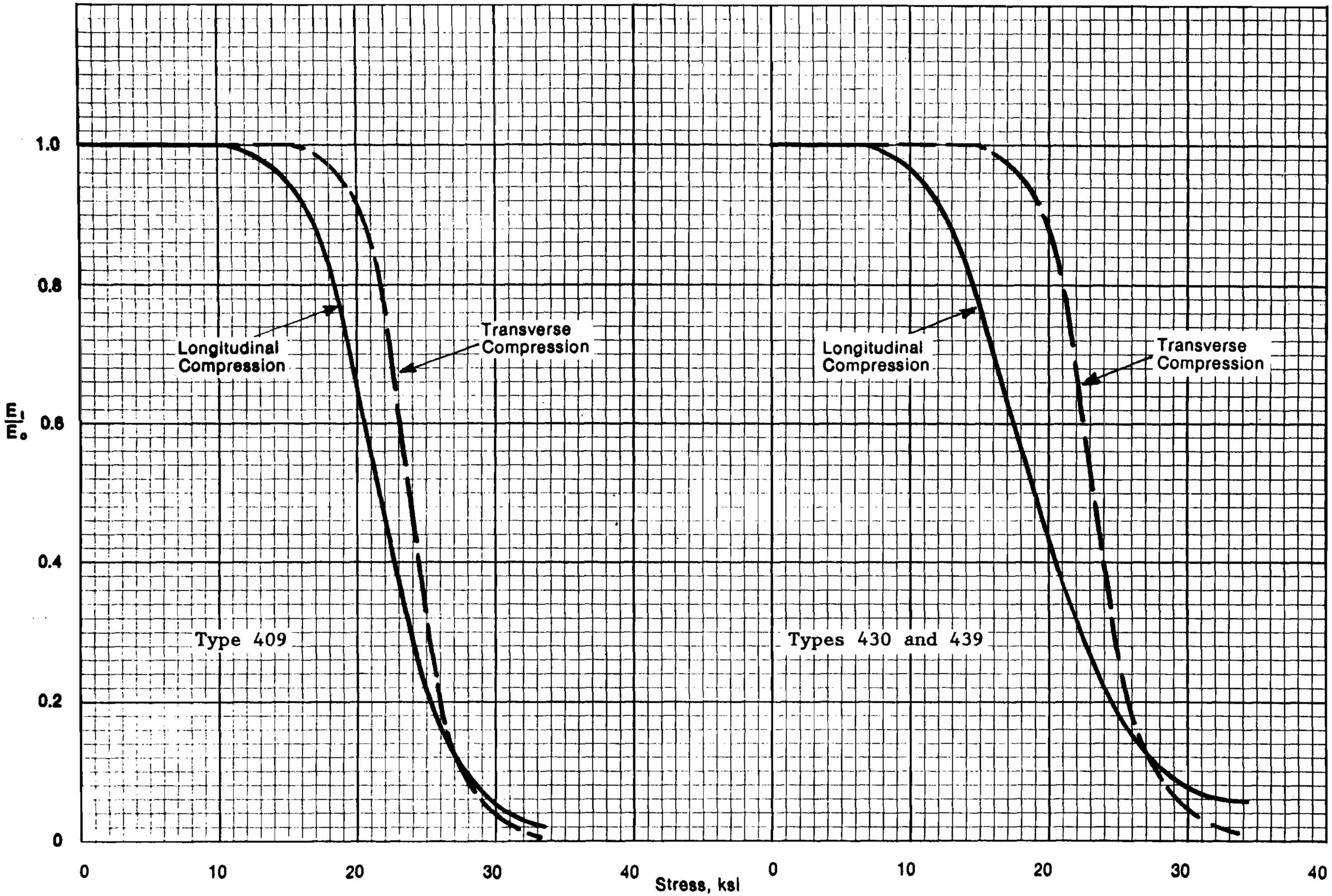

Figure A8 Plasticity Reduction Factors for Design of Laterally Unbraced Single Web Beams (Types 409, 430 and 439)

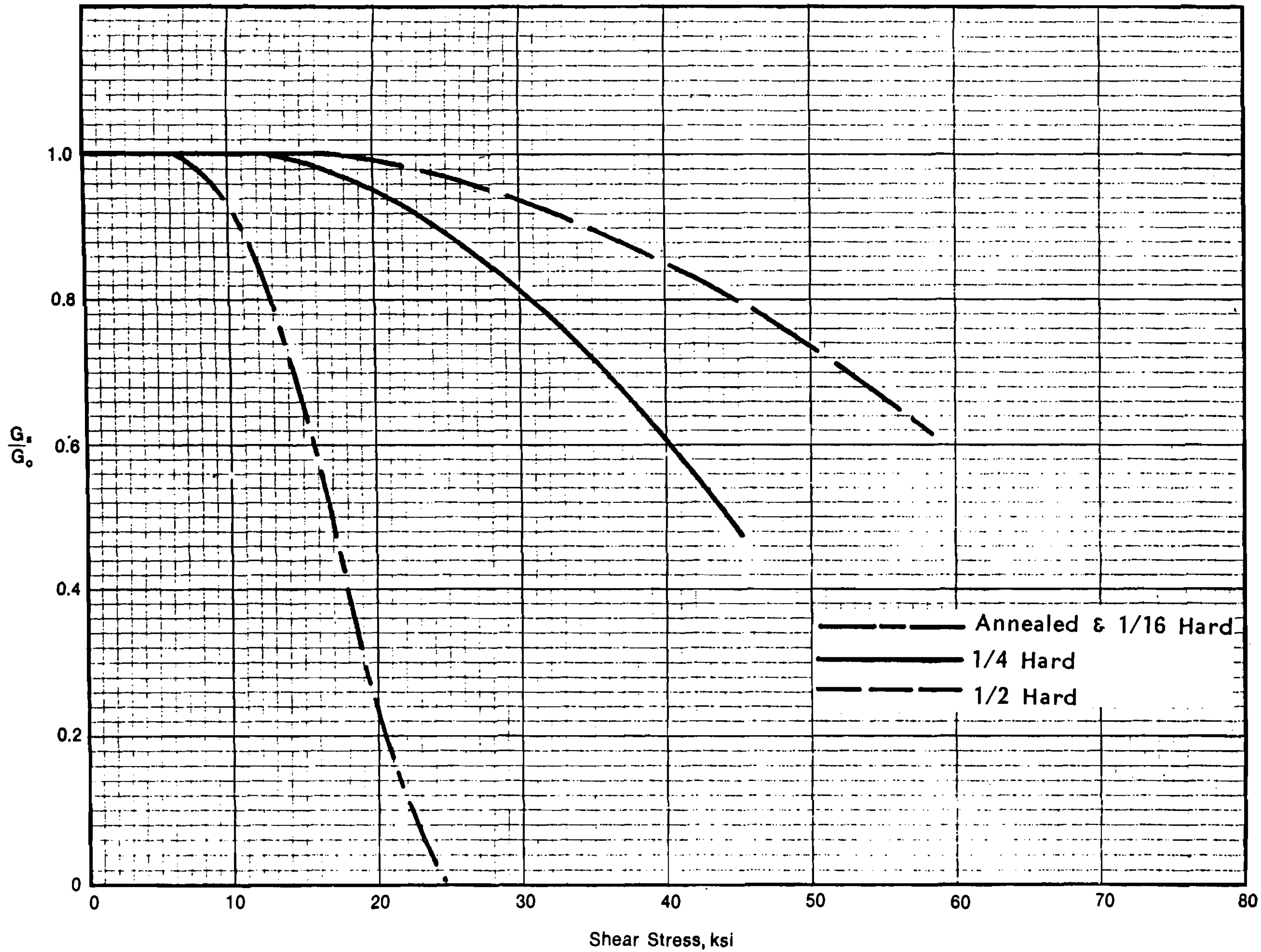

Figure A9 Plasticity Reduction Factors for Shear Stresses in Webs (Types 201, 301, 304 and 316)

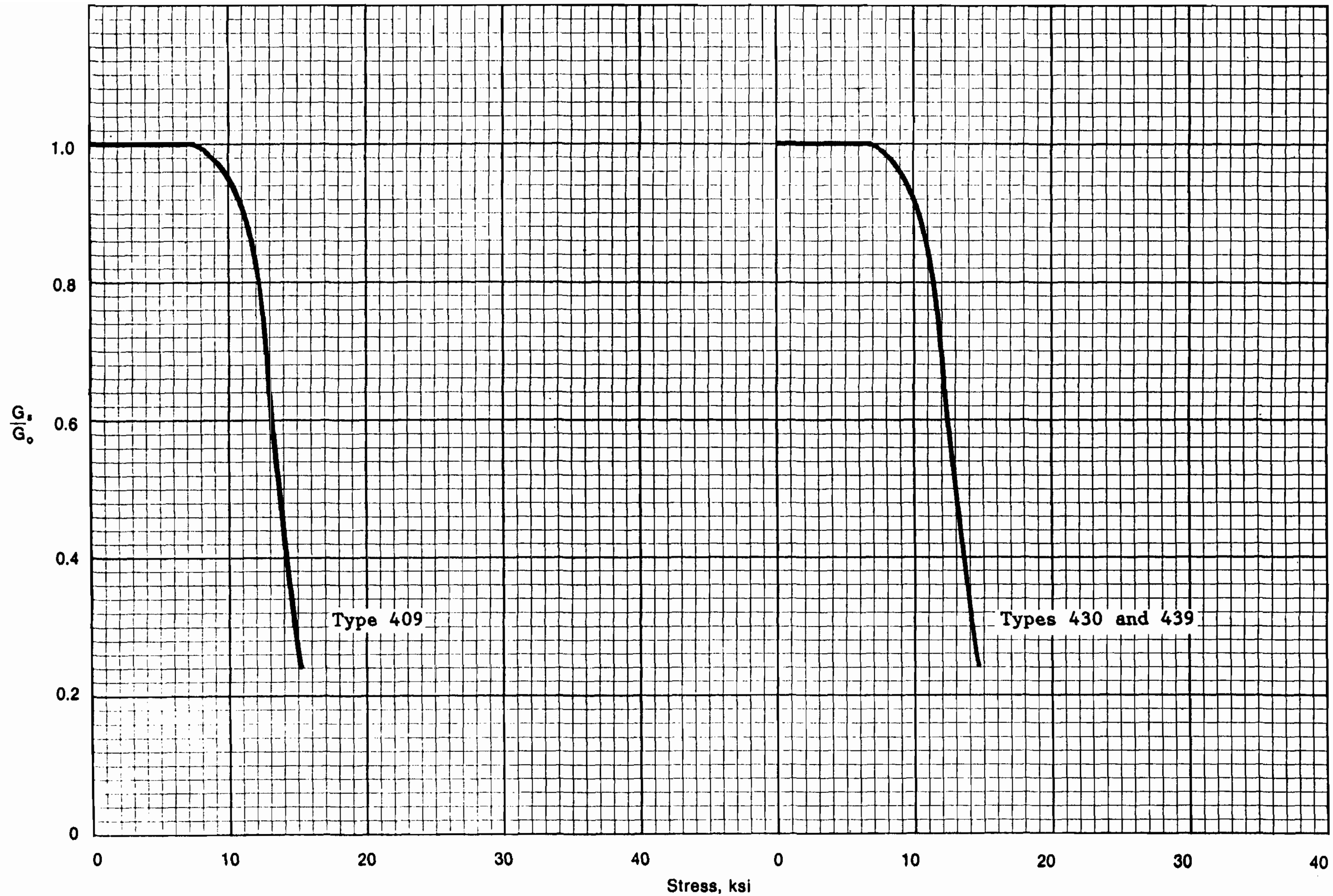

Figure A10 Plasticity Reduction Factors for Shear Stresses in Webs (Types 409, 430 and 439)

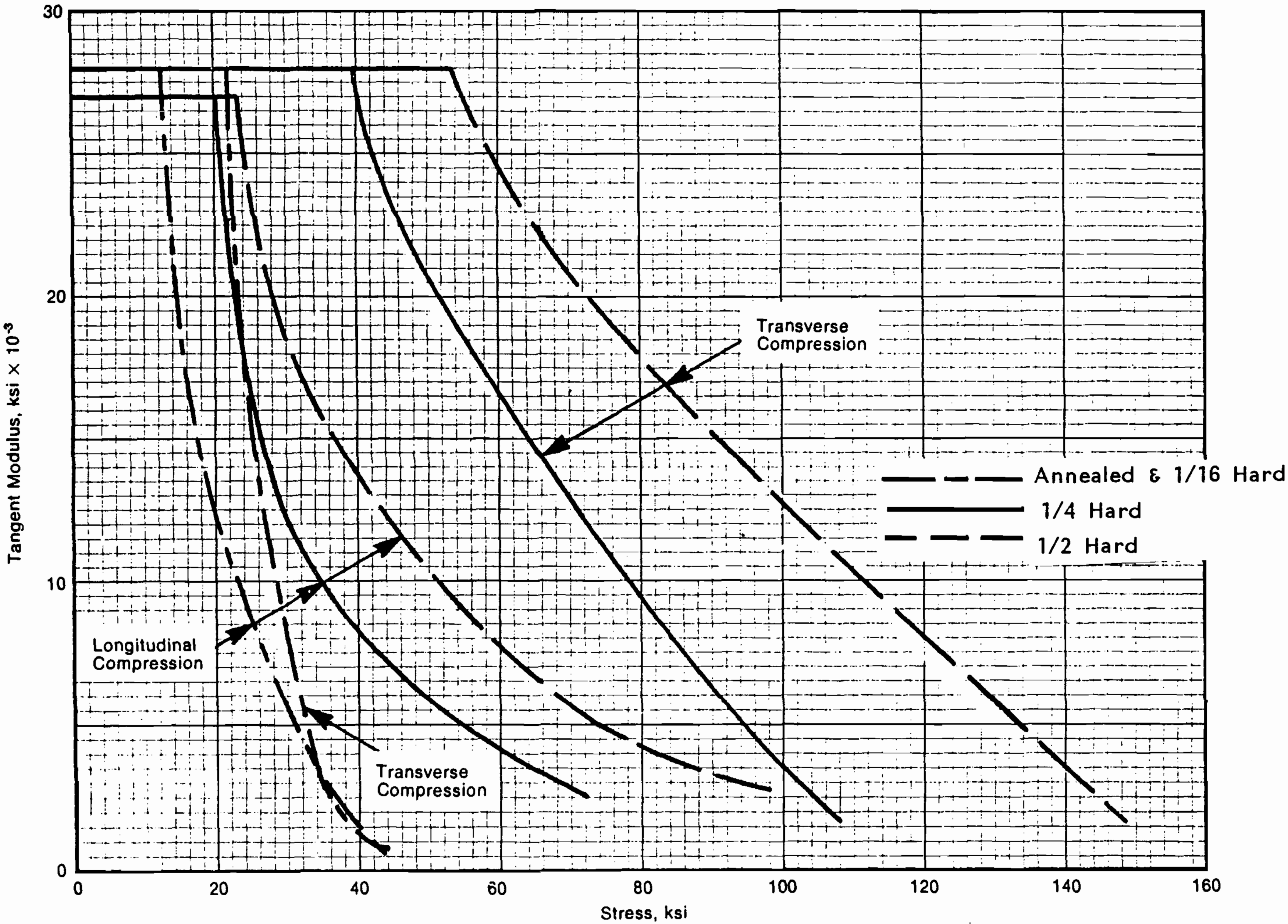

Figure A11 Tangent Moduli for Design of Columns

(Types 201, 301, 304 and 316)

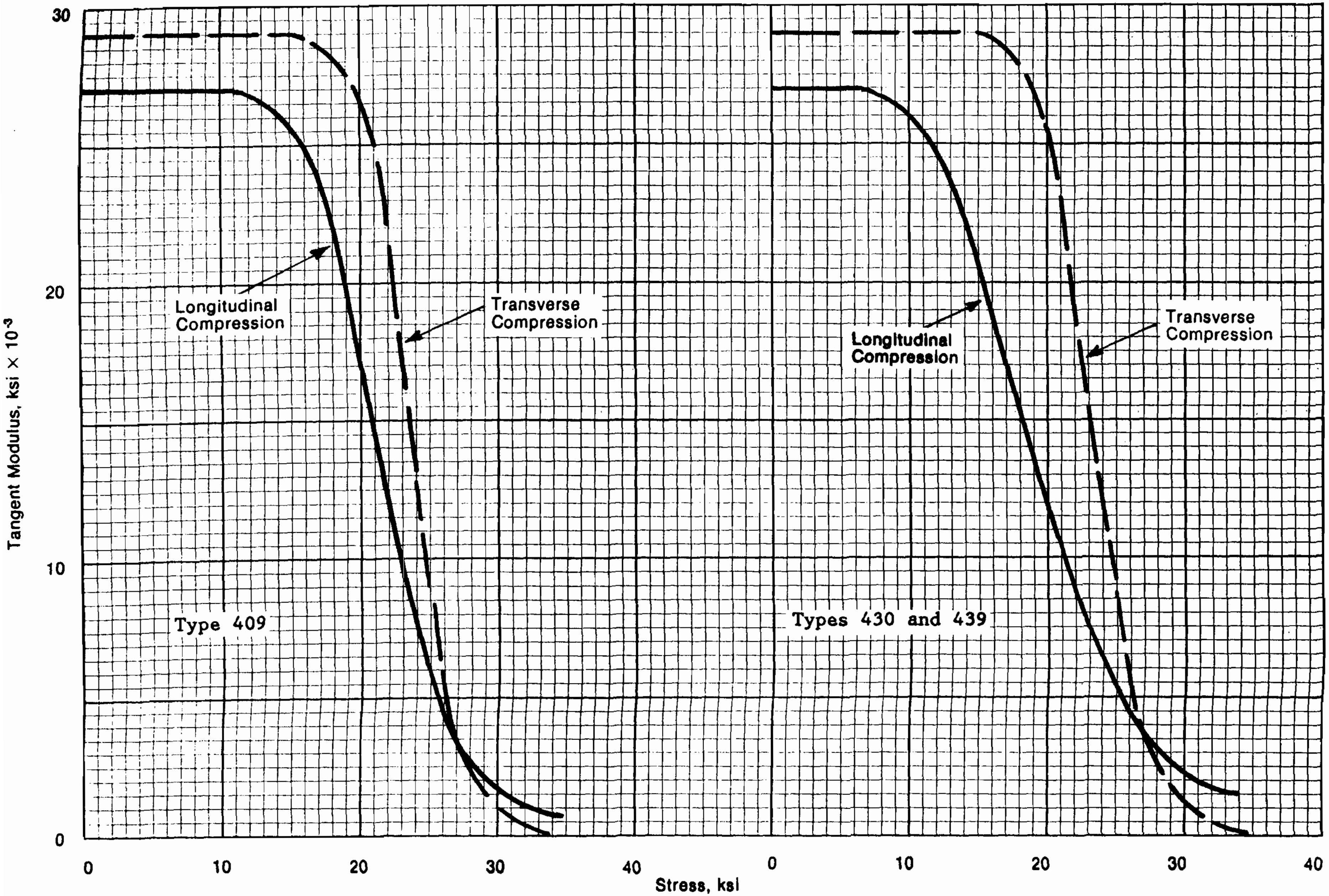

Figure A12 Tangent Moduli for Design of Columns

(Types 409, 430 and 439)

Appendix B <u>Flange Curling</u>

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

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

where

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

t = Flange thickness

d = Depth of beam

E_o = Initial modulus of elasticity, as given in Tables A4 and A5

c_f = Amount of curling*

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

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* The amount of curling that can be tolerated will vary with different kinds of sections and must be established by the designers.

Appendix C Shear Lag Effects

Where the span of the beam is less than $30w_f$(w_f as defined below) and it carries one concentrated load, or several loads spaced farther apart than $2w_f$, the effective design width of any flange, whether in tension or compression, shall be limited to the followings:

TABLE C1

Short, Wide Flanges -- Maximum Allowable Ratio

of Effective Design Width to Actual Width

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

where

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

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

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

Appendix D Stiffeners

D.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. The concentrated load or reaction shall not exceed the smaller of the allowable loads, P_a, given by (a) and (b) as follows:

(a) $P_a = P_n/\Omega_{st}$ (Eq. D-1)

where

$P_n = F_{yw}A_c$ (Eq. D-2)

$\Omega_{st} = 2.00$

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

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

F_{yw} = Lower value of the yield strength in beam web F_y or stiffener section F_{ys}

(b) $P_a = P_n/\Omega_c$ (Eq. D-5)

where

P_n = Nominal axial load evaluated according to Section 3.4(a) with A_e replaced by A_b

Ω_c = Factor of safety for axial compression = 2.15

$A_b = b_1t+A_s$, for transverse stiffeners at interior and under concentrated load (Eq. D-6)

$A_b = b_2t+A_s$, for transverse stiffeners at end support (Eq. D-7)

A_s = Cross-sectional area of transverse stiffeners

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

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

L_{st} = Length of transverse stiffener

t = Base thickness of beam web

The w/t_s ratio for the stiffened and unstiffened elements of cold-formed stainless 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} is the yield strength of stiffener steel and t_s is the thickness of the stiffener steel.

D.2 Shear Stiffeners

Where shear stiffeners are required, the spacing shall be such that the web shear force shall not exceed the allowable shear force, V_a, permitted by Section 3.3.2, and the ratio a/h shall not exceed $[260/(h/t)]^2$ nor 3.0.

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

$I_{smin} = 5ht^3[h/a - 0.7(a/h)] \geq (h/50)^4$ (Eq. D-10)

The gross area of shear stiffeners shall be not less than

$A_{st} = 0.5(1-C_v)[a/h - (a/h)^2/(a/h+\sqrt{1+(a/h)^2})]YDht$ (Eq. D-11)

where

$C_v = 45{,}000k_v/[F_y(h/t)^2]$ when $C_v \leq 0.8$ (Eq. D-12)

$C_v = [190/(h/t)](\sqrt{k_v/F_y})$ when $C_v > 0.8$ (Eq. D-13)

$k_v = 4.00 + 5.34/(a/h)^2$ when $a/h \leq 1.0$ (Eq. D-14)

$k_v = 5.34 + 4.00/(a/h)^2$ when $a/h > 1.0$ (Eq. D-15)

a = Distance between transverse stiffeners

Y = Ratio of the yield strength of web steel to yield strength of stiffener steel

D = 1.0 for stiffeners furnished in pairs

D = 1.8 for single-angle stiffeners

D = 2.4 for single-plate stiffeners

t and h are as defined in Section 2.1.2

D.3 Non-Conforming Stiffeners

The allowable load carrying capacity of members with transverse stiffeners that do not meet the requirements of Appendix D, such as stamped or rolled-in transverse stiffeners shall be determined by tests in accordance with Section 6 of this Specification.

PART II

COMMENTARY ON THE PROPOSED SPECIFICATION FOR THE DESIGN OF COLD-FORMED STAINLESS STEEL STRUCTURAL MEMBERS

(Second Draft)

January 1988

COMMENTARY ON THE ASCE STANDARD SPECIFICATION FOR THE DESIGN OF COLD-FORMED STAINLESS STEEL STRUCTURAL MEMBERS

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COMMENTARY ON THE ASCE STANDARD SPECIFICATION FOR THE DESIGN OF COLD-FORMED STAINLESS STEEL STRUCTURAL MEMBERS

INTRODUCTION

Cold-formed stainless steel sections have long been used architecturally in building construction due to their superior corrosion resistance, ease of maintenance, and pleasing appearance. Typical applications include curtain wall panels, mullions, door and window framing, roofing and siding, fascia, gutters, railing, and stairs. Other uses include arches, street lamp poles, railroad passenger and specialty cars.[1] Sometimes, stainless steel sheets may be used as facing material for composite sections.

Prior to 1968, the application of cold-formed stainless steel sections as structural members in building construction has been limited due to the lack of an appropriate design specification. This is because the AISI Specification[2] for carbon steel design does not apply to the design of stainless steel members due to the differences in strength properties, modulus of elasticity, and the shape of the stress-strain curve. For this reason an extensive research project has been conducted at Cornell University under the direction of Dr. George Winter. The objectives of the research were to investigate the structural behavior of stainless steel members and to develop factual information needed for the formulation of a separate specification.

The potential for use of stainless steel sections as structural members for load-carrying purposes has been increased since the publication of the First Edition of the Specification for the Design of Light

Gage Cold-Formed Stainless Steel Structural Members by American Iron and Steel Institute in 1968.[3] However, the scope of the 1968 Specification was limited only to the design of annealed AISI Types 201, 202, 301, 302, 304 and 316. The design provisions included in the First Edition of the Specification were based mainly on the extensive investigation conducted by Johnson and Winter at Cornell University[4,5] and the accumulated experience in the design of cold-formed carbon steel structural members.[2] The mechanical properties of stainless steels used in structural applications are summarized in Reference 6.

Because the 1/4- and 1/2- hard temper grades of stainless steels are often used in various applications due to their higher strength than annealed grades, additional research work has been conducted by Wang, Errera, Tang and Popowich at Cornell University[7,8,9] to further investigate the performance of structural members cold formed from cold-rolled austenitic stainless steels. The strengths of bolted and welded connections in stainless steel have also been studied in detail. Based on the findings of these continued studies, the Second Edition of the AISI Specification[10] was prepared at the University of Missouri-Rolla and issued by the Institute in 1974, in which the scope was extended to include the design of structural members cold formed from sheet, strip, plate or flat bar stainless steels, annealed and cold rolled in 1/4- and 1/2- hard tempers.

In 1987, the ASCE Standard Specification for the Design of Cold-Formed Stainless Steel Structural Members was prepared at the University of Missouri-Rolla under the sponsorship of American Society of Civil Engineers. The current Standard is essentially a revised version of the 1974 Edition of the AISI Specification updated on the basis of the results of recent investigations conducted in the field of cold-formed carbon steel structures [11] and the reevaluation of previous tests using stainless steels. It provides allowable design rules for AISI Types 201, 301, 304, 316, 409, 430, and 439 stainless steels, annealed and cold rolled in 1/16-, 1/4-, and 1/2- hard tempers.

This Commentary is intended to facilitate the use, and provide an understanding of the background, of the Standard Specification for the Design of Cold-Formed Stainless Steel Structural Members. This is accomplished through the integrated bibliography as well as brief substantive discussions. Some discussions are based on References 10 and 11 with appropriate modifications. In this Commentary, the individual sections, equations, figures and tables are identified by the same notation as in the Specification and the material is presented in the same sequence.

1. GENERAL

1.1 Limits of Applicability and Terms

1.1.1 Scope and Limits of Applicability

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

Some of the significant differences in material properties between cold-fromed stainless steels and 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.

The design provisions are developed primarily for cold-formed stainless steel structural members used for buildings. It may also be used for structures other than buildings provided that appropriate allowances are made for dynamic effects. The general information on impact loading of thin-walled beams and columns can be found in References 12 to 15.

It should be noted that the Specification for the Design of Cold-Formed Steel Structural Members issued by the American Iron and Steel Institute and the Specification for the Design, Fabrication and Erection of Structural Steel for Buildings[16] issued by the American Institute of Steel Construction do not apply to the design of stainless steel structural members.

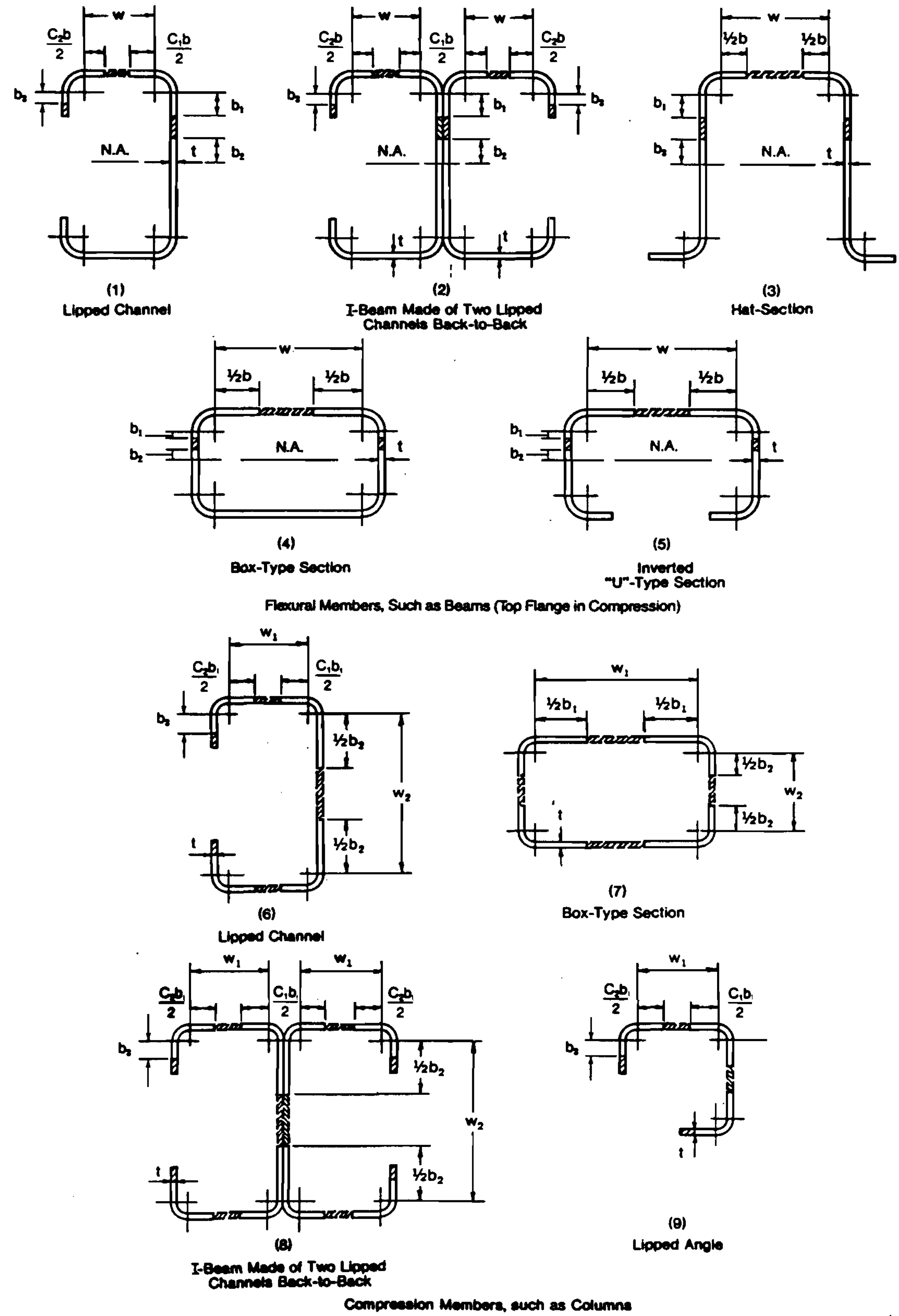

Figure C1 Members with Stiffened Compression Elements

1.1.2 Terms

The notations and symbols used in the Specification for the Design of Cold-Formed Stainless Steel Structural Members are mostly the same as those used in the AISI 1986 Specification[11] for the design of cold-formed carbon steels, except that some minor editorial changes have been made in this edition of the Specification for stainless steel design.

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 has 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 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

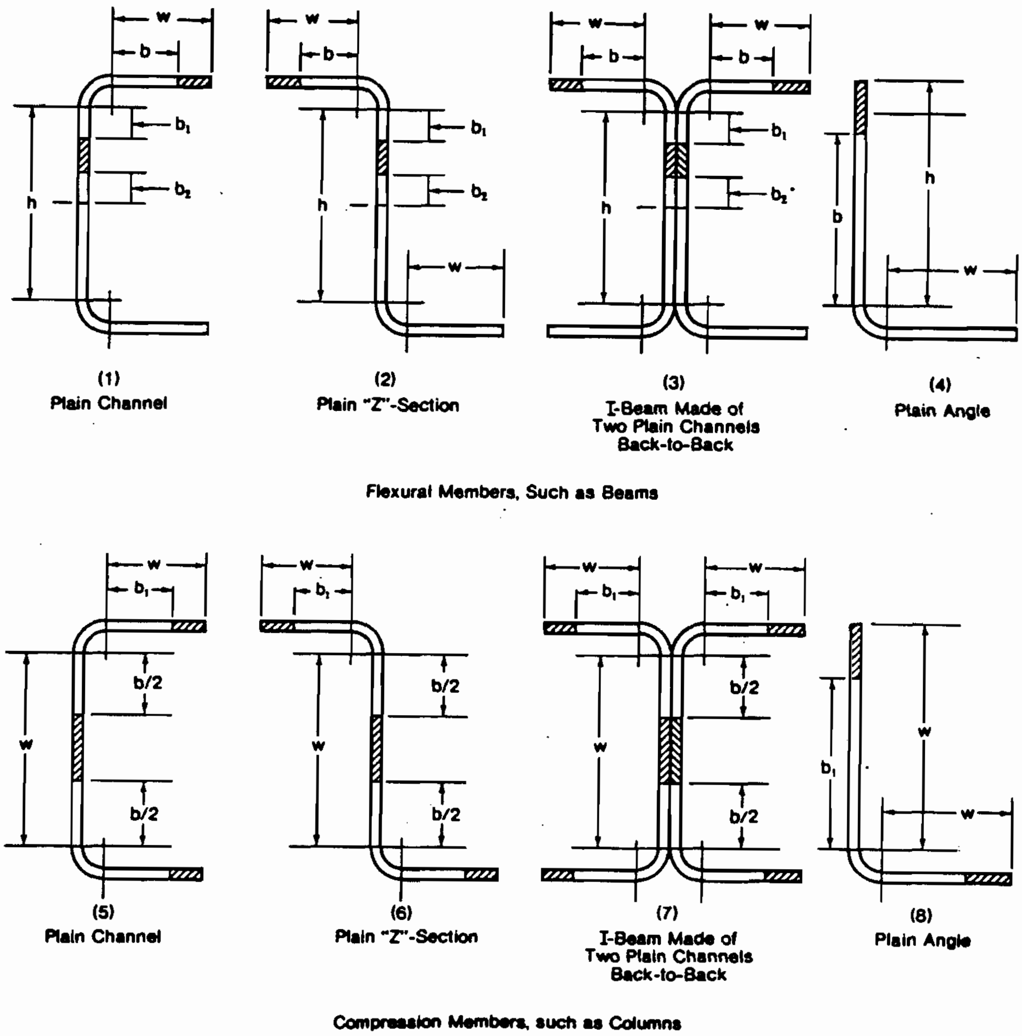

Figure C2 Members with Unstiffened Compression Elements which contain unstiffened compression elements. Shapes (1), (2), and (3) have only a web to support the compression flange element. The legs of Shape (4) provide mutual stiffening action to each other along their common edge. Shapes (5), (6), and (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.

(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 falnges is also handled by using an effective design width. These matters are treated in Appendix C 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 Specification provisions is intended to facilitate design in any compatable system of units (U.S. customary, SI, or metric). It will also simplify the conversion to a load and resistance factor design format.

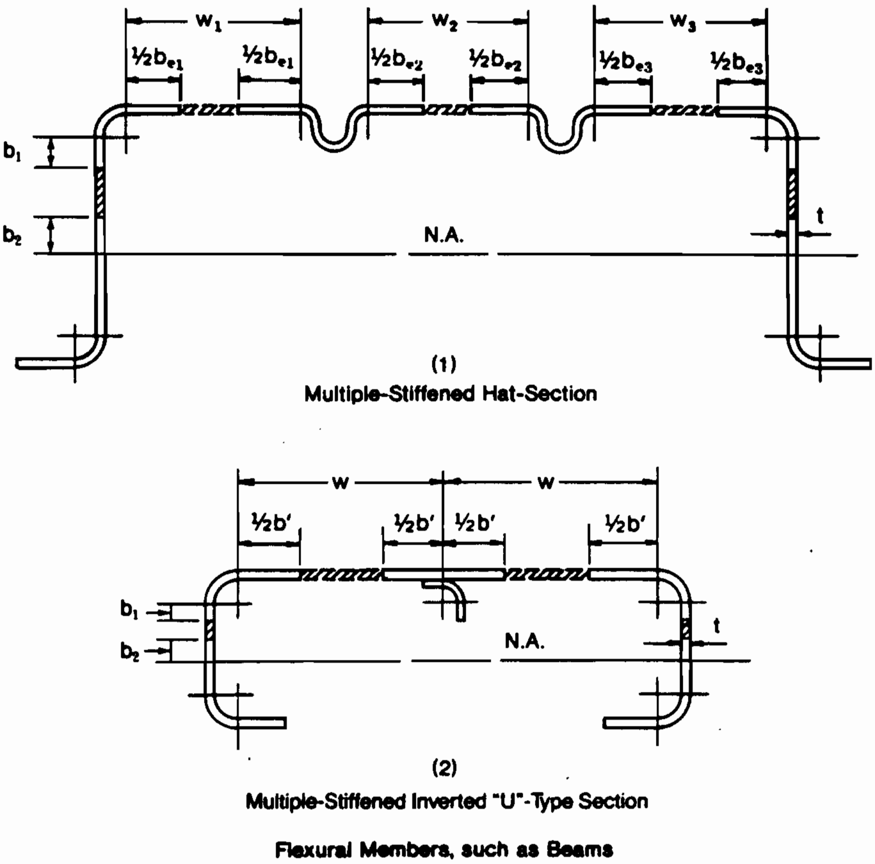

Figure C3 Members with Multiple-Stiffened Compression Elements

1.2 Non-Conforming 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 current Specification is now applicable for Types 201, 301, 304, 316, 409, 430, and 439 stainless steels, among which Types 201, 301, 304, and 316 are austenitic stainless steels and Types 409, 430, and 439 are 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 structural properties for design are: yield strength, tensile strength, proportional limit, initial modulus of elasticity, tangent modulus, secant modulus, and ductility.

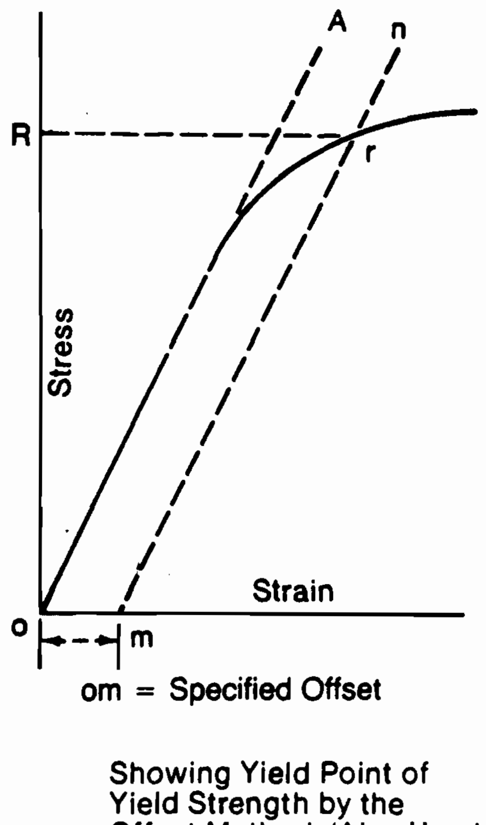

Figure C4 Stress-Strain Diagram Showing Yield Strength Determination

Standard methods for testing of steels and steel products are given in ASTM A370.[23] Determination of yield strengths or proportional limits for gradual yielding stainless steels can be done simply by using the offset method as shown in Figure C4. The distance om equals to 0.002 in./in. or 0.0001 in./in. for determining yield strength or proportional limit, respectively.

The ASTM Specification A176-85a[17] 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[18] are also included in Table C1. Type 439 is a stabilized version of Type 430 for weldability.

The ASTM Specification A276-85a[19] 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[20] 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 this operation is done by roller leveling or, in the case of

TABLE C1

ASTM Requirements for Mechanical Properties of Types 409, 430 and 439[17,18]

UNS Designation	AISI Type	Tensile Strength min. (ksi)	Yield * Strength min. (ksi)	Elongation in 2 in. min. (%)
S40900	409	55	30	22.0
S43000	430	65	30	22.0
S43035	439	65	30	22.0

1 ksi = 6.895 MPa; 1 in. = 25.40 mm

* The yield strength is determined by the 0.2% offset method described in Methods and Definitions, ASTM A370.

TABLE C2

ASTM Requirements for Mechanical Properties

of Types 201, 301, 304 and 316[20]

UNS Designation	AISI Type	Tensile Strength min. (ksi)	Yield Strength min. (ksi)	Elongation in 2 in. min. (%)
		Annealed		
S20100 Class 1	201-1	90	30	40.0
S20100 Class 2	201-2	95	45	40.0
S30100	301	90	30	40.0
S30400	304	75	30	40.0
S31600	316	75	30	40.0
		1/16 Hard		
S20100 PSS *	201	95	45	40.0
FB **		75	40	40.0
S30100	301	90	45	40.0
S30400 PSS	304	80	45	35.0
FB		90	45	40.0
S31600 PSS	316	85	45	35.0
FB		90	45	40.0
		1/4 Hard		
S20100	201	125	75	25.0
S30100	301	125	75	25.0
S30400	304	125	75	10.0
S31600	316	125	75	10.0
		1/2 Hard		
S20100	201	150	110	15.0
S30100	301	150	110	15.0
S30400	304	150	110	6.0
S31600	316	150	110	6.0

1 ksi = 6.895 MPa; 1 in. = 25.40 mm

* PSS means plate, strip, sheet.
** FB means flat bar.

Note:
This specificaiton defines minimum properties only and does not imply a range.

cut length sheets, by stretching. This flattening operation results in a small reduction in thickness.

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.[4] Table C3 lists the tested ultimate and yield strengths of the strain flattened condition of the annealed austenitic stainless steels.

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.[7] Tables C4 and C5 list the tested ultimate and yield strengths and the effective proportional limits for these stainless steels.

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 A666-84 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 indicates that the tensile yield strength in transverse direction is a value between the compressive yield strengths in the transverse and longitudinal directions. The compression yield strength in longitudinal direction is the lowest value as compared with other cases.

The tested ultimated and yield strengths of Types 409 and 430, annealed and cold rolled stainless steels, given in Tables C6 and C7, respectively, were established on the basis of Reference 21. From these two tables, it is noted that Types 409 and 430 ferritic stainless steels

TABLE C3

Mechanical Properties of Annealed and Strain Flattened Type 304 Austenitic Stainless Steel[4,10]

Type of Stress	Tested Ultimate Strength (ksi)	Tested Yield Strength (ksi)	Recommended Yield Strength (ksi)	Effective Proportional Limit (ksi)
Longitudinal Tension	94.6	38.0	37.0	24.7
Transverse Tension	91.1	36.0	37.0	21.6
Transverse Compression	---	37.5	37.0	24.4
Longitudinal Compression	---	34.5	34.0	15.5

1 ksi = 6.895 MPa

TABLE C4

Mechanical Properties of 1/4-Hard Temper

Type 301 Austenitic Stainless Steel[7,20]

Type of Stress	Tested * Ultimate Strength (ksi)	Tested ** Yield Strength (ksi)	Recommended Yield Strength (ksi)	Effective Proportional Limit (ksi)
Longitudinal Tension	137.9	77.5	75.0	37.5
Transverse Tension	137.0	78.0	75.0	41.3
Transverse Compression	---	94.5	90.0	45.0
Longitudinal Compression	---	52.5	50.0	25.0

1 ksi = 6.895 MPa

* For 1/4-hard Type 301 stainless steel, Reference 7 recommends a value of 130 ksi. Reference 20 requires a minimum strength of 125 ksi for plate, sheet, strip and flat bar of Types 201, 301, 304 and 316.

** Based on 95 % probability.

TABLE C5

Mechanical Properties of 1/2-Hard Temper

Type 301 Austenitic Stainless Steel[7,20]

Type of Stress	Tested * Ultimate Strength (ksi)	Tested ** Yield Strength (ksi)	Recommended Yield Strength (ksi)	Effective Proportional Limit (ksi)
Longitudinal Tension	167.0	108.2	110.0	49.5
Transverse Tension	168.1	100.0	100.0	60.0
Transverse Compression	---	121.3	120.0	60.0
Longitudinal Compression	---	70.5	65.0	31.5

1 ksi = 6.895 MPa

* For 1/2-hard Type 301 stainless steel, Reference 7 recommend a value of 150 ksi. Reference 20 requires a minimum strength of 150 ksi for plate, sheet, strip and flat bar of Types 201, 301, 304 and 316.

** Based on 95 % probability.

TABLE C6

Mechanical Properties of Type 409 Ferritic Stainless Steel[21]

Type of Stress	Tested * Ultimate Strength (ksi)	Tested ** Yield Strength (ksi)	Recommended Yield Strength (ksi)	Effective Proportional Limit (ksi)
Longitudinal Tension	59.7	36.7	30.0	22.8
Transverse Tension	60.6	38.6	30.0	24.9
Transverse Compression	---	39.2	30.0	24.9
Longitudinal Compression	---	38.1	30.0	21.9

1 ksi = 6.895 MPa

* For Type 409 ferritic stainless steel, Reference 17 requires a minimum strength of 55 ksi for plate, sheet and strip.

** Based on 95 % probability.

TABLE C7

Mechanical Properties of Type 430 Ferritic Stainless Steel[21]

Type of Stress	Tested * Ultimate Strength (ksi)	Tested ** Yield Strength (ksi)	Recommended Yield Strength (ksi)	Effective Proportional Limit (ksi)
Longitudinal Tension	70.4	43.9	30.0	21.0
Transverse Tension	75.7	48.3	30.0	24.3
Transverse Compression	---	50.2	30.0	24.6
Longitudinal Compression	---	43.4	30.0	18.6

1 ksi = 6.895 MPa

* For Type 430 ferritic stainless steel, Reference 17 requires a minimum strength of 65 ksi for plate, sheet and strip.

** Based on 95 % probability.

provide a relatively high ratio of proportional limit-to-yield strength in both transverse tension and compression than those listed in Tables C3, C4 and C5 for austenitic stainless steels. Table C8 summarizes the mechanical properties of Types 409 and 439 based on the statistical data obtained from Allegheny Ludlum Steel.[22]

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 sizeable 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-inch (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 11 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.

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

TABLE C8

Summary of Mechanical Properties for Types 409 and 439[22]

Type	Number of Tests		Yield Strength (ksi) Mean	Yield Strength (ksi) Std. Dev.	Tensile Strength (ksi) Mean	Tensile Strength (ksi) Std. Dev.	Elongation in 2 in. (%) Mean	Elongation in 2 in. (%) Std. Dev.
409	In	2597	39.6	3.09	64.1	2.59	33.6	2.78
	Out	2154	39.3		64.3		33.9	
439	In	869	48.4	3.11	72.8	2.62	29.6	2.75
	Out	138	48.9		73.8		29.6	

1 ksi = 6.895 MPa; 1 in. = 25.40 mm

Thickness Mean = 0.042 inch
Range = 0.016 inch- 0.125 inch

thickness tolerances are over (+) and nothing under (-). If the stainless steel is ordered to nominal thickness, the thickness torlerances 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 be at least 95 percent of the design thickness. Thus, a portion of the factor of safety may be considered to cover minor negative thickness torlerances.

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 manufacture 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 that used for the design of cold-formed carbon steels. This provision is adopted from Reference 11. 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.

For load combinations involved with wind or earthquake load, the Specification recognizes the generally accepted practice of increasing the allowable stress by 33 1/3 percent.[24] This is accomplished by permitting a 25 percent reduction in design loads as included in Section 1.4.4 of the Specification. When the deflection of structural members is a critical factor in design, the increased allowable stress should not exceed the proportional limit of stainless steels included in Table A19 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. Design guidance can be obtained from Section 1.13.3 of the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings.[16]

1.5 <u>Structural Analysis and Design</u>

1.5.1 <u>Design Basis</u>

This Specification uses allowable moments and loads instead of allowable stresses. A safe load is determined by applying a factor of safety to the maximum strength, buckling or yielding, calculated for the member or connection in question. The fudamental nature of the factor of safety is to compensate for uncertainties inherent in the design, fabrication, and erection of structural components as well as in the applied load. Table C9 summarizes the factors of safety adopted in the Specification. It should be noted that these factors of safety used for stainless steel

TABLE C9

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-sections
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 against tension failure on connected parts
Bearing	5.3.3	2.40 for single and double shear connections
Bolt Shear	5.3.4	3.0 against shear failure of bolts
Wind or Earthquake Loads	1.4.4	A 25 % reduction of nominal safety factor is permissable provided that the member or connection thus designed is not less than that required for combination of dead and live load

design are relatively larger than those used in Reference 11 for cold-formed carbon steels. In part, this reflects the lack of accumulated experience in the design of stainless steel structural members. In addition, this larger safety factor has been chosen to minimize the possibility of permanent deformations when members are stressed above the proportional limit, and to reduce the necessity for inelastic deflection calculation.

1.5.2 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.[7] Tables 4 and 7 of Reference 5 revealed that for annealed and strain-flattened stainless steels, the potential increase in member strength caused by cold forming ranges from 5 to 11 percent for flexural strength, and 14 to 24 percent for column strength for the particular sections investigated.

Section 1.5.2.2 of the Specification permits utilization of the cold work of forming under certain conditions of Sections 1.5.2.2.1 and

1.5.2.2.2. Because Karren's formula (Reference 25) was originally developed on the basis of the test data for carbon steels and the equation may not be applicable for determining the yield strength of corners for stainless steels, any use of the strength increased from cold work must be based on full section tests. Therefore, the requirements of Section 1.5.2.2 are the same as those used in the AISI 1974 Specification with some minor editorial changes.[10]

1.5.3. Serviceability

Further information may be found in the applicable building code and contract documents.

1.5.4 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 welded and bolted connections are also included.

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.

2. ELEMENTS

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

2.1 Dimensional Limits and Considerations

Because of the relatively large flat-width-to-thickness ratio (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 unchanged from the previous edition of Specification.[10] 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 adopted from Reference 11 for the design of 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 Specifcation 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 Sections 3.3.1.1 and 3.4, respectively.

In addition, Reference 26 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. B-1) is included in Specification Appendix B to determine the maximum permissible flange width, w_f, for a given amount of tolerable curling, c_f. This formula is similar to that included in Reference 11 except that, in Eq. B-1, 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 11 suggests that curling on the order of 5 percent 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 closely control out-of-plane distortions of unusually wide flanges for the sake of appearance.

(c) Shear Lag Effects

The Specification Appendix C 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[27] as follows: "In metal beams of the usual shapes, the normal stresses are induced in the flanges through shear stresses transfered 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 non-uniform 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 non-uniformly 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 1974 Specification[10] is retained in this edition of the 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 flnage, as in 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 Appendix C 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 differ from the 1974 Edition of the AISI Specification. The limit of h/t for flexural members having unreinforced webs has been increased from 150, as previouly used, 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. When webs are provided with transverse stiffeners which satisfying the requirements of Specification Appendix D.1, the depth-to-thickness ratios are limited to 260 for webs using bearing stiffeners and 300 for webs using bearing and intermediate stiffeners. Previous Specification stipulated that the web depth-to-thickness ratio, h/t, shall not exceed 200 for member webs with stiffeners.[10] These limits are the same as those used in the current AISI Specification[11] for the design of cold-formed carbon steel sections.

It should be noted that the definition for h, the depth of the flat portion of the web measured along the plane of the web, differs from the 1974 Edition of the AISI Specification.

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 C5 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 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 general equation for the critical buckling stress of an isotropic sheet element is

$$f_{cr} = \frac{\Pi^2 k \eta E_o}{12(1-\mu^2)(w/t)^2} \qquad \text{(Eq. C-1)}$$

where

k = Buckling coefficient

E_o = Initial modulus of elasticity

μ = Poisson's ratio in the elastic range

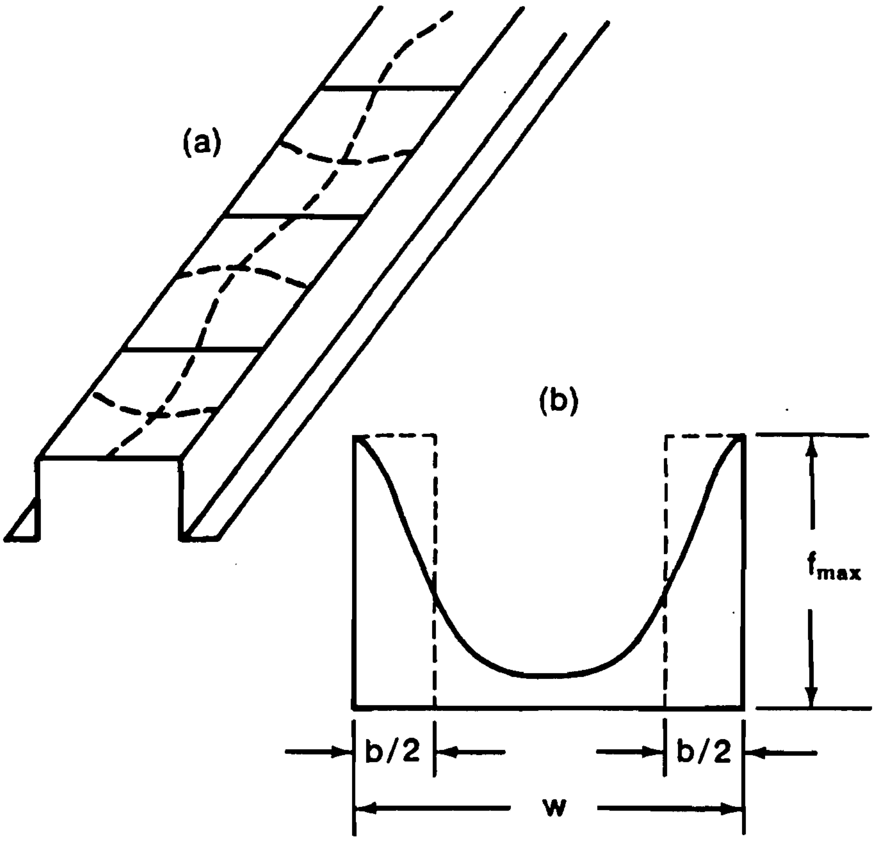

Figure C5 Local Buckling and Post Buckling Strength of Stiffened Compression Element

η = Plasticity reduction factor

w = Flat width

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 postbuckling strength of compression elements into account. Figure C5(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.

The 1974 Edition of the AISI Specification utilized the effective design width approach only for the uniformly compressed stiffened elements. In the current 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-2 was originally derived by Winter for the design of cold-formed carbon steel members.[28] It has been verified by Johnson and Wang for using stainless steel members.[4,7]

$$b/t = 1.9 \sqrt{E_o/f_{max}}\left[1-0.475\sqrt{E_o/f_{max}}/(w/t)\right] \qquad \text{(Eq. C-2)}$$

where

b = Effective design width

E_o = Initial modulus of elasticity

f_{max} = Maximum stress at the edge

w = Flat width

t = Thickness

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

$$b/t = 1.9 \sqrt{E_o/f_{max}}\left[1-0.415\sqrt{E_o/f_{max}}/(w/t)\right] \qquad \text{(Eq. C-3)}$$

Consequently, an additional study has been made by Wang on the suitability of Eq. C-3 for stainless steel structural members.[7] Because Eq. C-3 compares favorably with the experimental data obtained from numerous beam and column tests as shown in Figure C6, this equation was used in the 1974 Edition of the AISI Specification for the design of stainless steel structural members.[10]

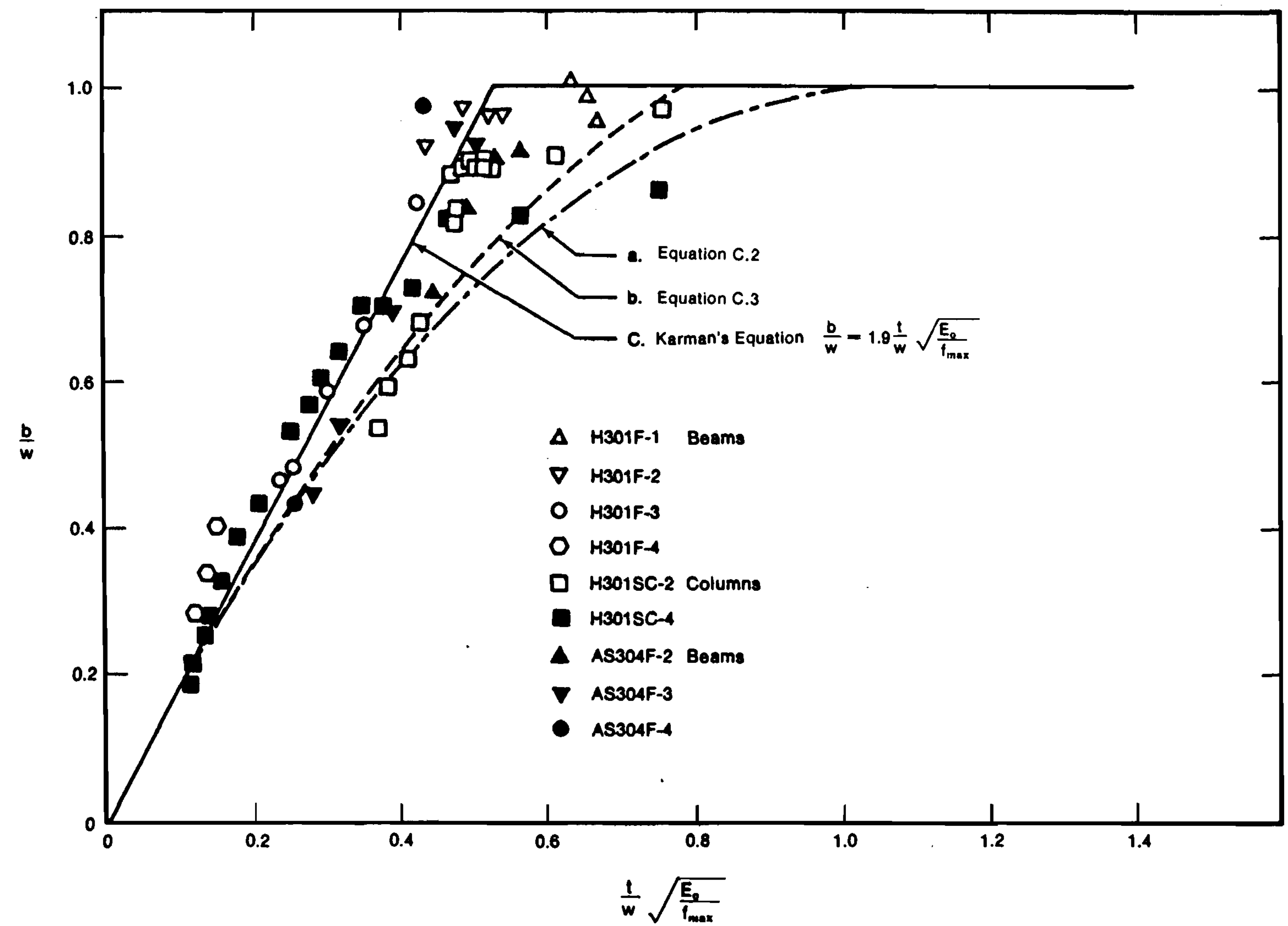

Figure C6 Correlation Between the Effective Width Formula and Test Data [7]

In this Specification, Eq. C-3 is retained for determining the effective design width for uniformly compressed stiffened elements. This equation can be expressed in terms of the ratio of b/w as follows:

$$b/w = (1/\lambda)(1 - 0.22/\lambda) \qquad \text{(Eq. C-4)}$$

where λ is a slenderness factor determined as

$$\lambda = \sqrt{f_{max}/f_{cr}} \qquad \text{(Eq. C-5)}$$

f_{cr} is the critical buckling stress of an isotropic sheet element calculated from Eq. C-1.

In Specification Section 2.2, Eq. 2.2.1-4 for calculating the slenderness factor, λ, is obtained from Eq. C-5 by substituting the value of f_{cr} from Eq. C-1 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.[8] 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.[4,7,29] 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 at 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 5.

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.[11]

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 Ref. 30 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 11 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.[29] 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.[8] In the 1974 Edition of the AISI Specification,[10] the buckling coefficient was conservatively taken as 0.5 for unstiffened compression elements. This value has been retained in this Specification. The plasticity reduction factor, η, used for the limit of local distortions may be taken as E_s/E_o for members having uniformly compressed unstiffened elements.[4,7,31,32]

2.3.2 Unstiffened Elements and Edge Stiffeners with Stress Gradient

Reference 30 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 capacities 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 11 except that the buckling coefficient, k, is equal to 0.5 for stainless steel members.

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

The 1974 Edition of the AISI Specification treated stiffeners as fully effective elements. However, in Reference 11, all compression elements including stiffeners are analyzed on the basis of effective widths or areas. Because the effective design width of stainless steel elements with an edge stiffeners or one intermediate stiffener has not been investigated in the past, Section 2.4 of the current Specification is adopted from the 1986 Edition of the AISI Specification.

2.4.1 Uniformly Compressed Elements with an Intermediate Stiffener

The 1974 Edition of the AISI Specification[10] 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 edge stiffener. The current provision adopted from Reference 11 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., (I) elements are fully effective as an stiffened element and do not need an intermediate stiffener, (II) elements can be fully effective as a stiffened element when it has an adequate stiffener, and (III) element is not fully effective even with an adequate stiffener.

2.4.2 Uniformly Compressed Elements with an Edge Stiffener

The 1974 AISI Cold-Formed Stainless Steel 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 current Specification contains three different cases for the determination of the effectiveness of compression elements and stiffeners.

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 30.

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

The current Specification retains Eq. 2.5-1 from the 1974 Edition of the AISI Specification 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. Addititonal 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

Design requirements for attached transverse stiffeners and for intermediate stiffeners are newly added to Appendix D of this Specification. Due to the lack of test results on cold-formed stainless steel members with stiffeners, the provisions are adopted from Reference 11. Equation D-1 of Appendix D serves to prevent end crushing of the transverse stiffeners, while Equation D-5 is to prevent column-type buckling for the web-stiffeners. The equations for computing the effetive areas (A_b and A_c) and the effective width (b_1 and b_2) were determined on the basis of Reference 33.

The equations for determining the minimum required moment of inertia (Eq. D-10) and the minimum required gross area (Eq. D-11) of attached intermediate stiffeners are based on the studies summarized in Reference 33. In this reference, test data show that even though the allowable 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. D-10, the minimum value of $(h/50)^4$ was selected from the AISC Specification.[16]

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

3. MEMBERS

This Section is different as compared with the 1974 Edition of the AISI Specification. In order to simplify the use of the Specification, all design provisions relative to a specific member type, e.g., beam, column or beam column, have been assembled in one location within the Specification. Also, design provisions are given in terms of allowable load or moment, instead of allowable stress. To clarify the phenomenon being evaluated, the nominal capacity and required safety factor are also explicitly stated.

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 maximum allowable load in tension is obtained by dividing the yield load by a safety factor of 1.85, which is the same as that used in the 1974 Edition of the AISI Specification.[10] The net section should be used when computing the capacity of a tension member.

When mechanical fasteners are used, the allowable tensile force should also be limited by Section 5.3.2 of the Specification.

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

The allowable bending moment is determined by dividing the nominal moment by a basic safety factor of 1.85. Flexural strength depends on the geometry of the member and the lateral support conditions. The provisions of Specification Section 3.3.1 encompass these considerations.

3.3.1.1 Nominal Section Strength

Beams not subject to lateral, torsional or torsional-flexural buckling, are designed on the basis of the initiation of yielding in either the extreme tension or compression fibers. This is the premise in Section 3.3.1.1, which is consistent with the 1974 Edition of the AISI Specification.[10]

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. 3.3.1.1-3 to 3.3.1.1-6 and the elastic section modulus of the full, unreduced section should be used to determine the permissible moment. The use of the full, unreduced section properties for this case is based on the 1974 Edition of the AISI Specification.[10] These compressive stresses are based on the considerations that if some barely perceptible distortions at the design load are allowed, the allowable stresses for stiffened and unstiffened compression elements are limited to 1.2 F_{cr} and F_{cr}, respectively. If no local distortions at the design load are permitted, the allowable stresses for stiffened and unstiffened compression elements should not exceed 0.9 F_{cr} and 0.75 F_{cr}, respectively.[4,7,10] In the above expression, F_{cr} is the critical buckling stress.

This provision is considered to be necessary for 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.

3.3.1.2 Lateral Buckling Strength

The design expressions for laterally unbraced segments of flexural members included in Specification Section 3.3.1.2 represent an improvement over the 1974 Edition of the AISI Specification.[10] This approach is adopted from Reference 11. It 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.

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. Equation 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.[34] Equation 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.[35]

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.[36]

3.3.2 Strength for Shear Only

This provision is primarily the same as that used in the 1974 Edition of the AISI Specification,[10] except that some editorial changes have been made in the current Specification. The definition of h has been changed to be the flat portion of the web instead of the clear distance between flanges.

The general equation for the critical stress of flat sheet or plate elements is given in Eq. C-1 of this Commentary. For shear stress, f_{cr} is replace by τ_{cr}. The buckling coefficient, k, is 5.35 for simply supported conditions. Various plasticity reduction factors are given by different authors. One of the simplest is that suggested by Gerard[37] 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 maximum allowable shear stress, 0.61 F_{yv}, is determined by dividing the shear yield strength by a safety factor of 1.64. This safety factor has been chosen to reflect the less serious nature of shear yielding (which is localized at the point of maximum shear in the web) in comparison with yielding in tension or compression. This choice is based on the reasoning similar to that behind the AISI Specification[11] for carbon steel.

3.3.3 Strength for Combined Bending and Shear

For cantilever beams and continuous beams, high bending stresses often combine with high shear stresses at the 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 27). The interaction formula now expressed in the ratios of moments and forces, instead of stresses, is suitable for the design of beams with unreinforced webs.

In addition, a new interaction equation (Eq. 3.3.3-2) is included in this Specification for beam webs with adequate transverse stiffeners. Safety factors and the inelastic response of stainless steel are accounted for in calculating M_a and V_a.

3.3.4 Web Crippling Strength

This Section has been modified according to the current AISI Specification[11] for the design of cold-formed carbon steel members. However, the factors of safety used in Table 1 of the Specification are 2.0 and 2.20 for shapes having single webs and I-sections, respectively. Since no research work has been done in the Cornell project to study the problem of web crippling of beams made of high yield strength steels, a conservative approach has been taken by using the longitudinal compression yield strength of the stainless steel which is the lowest value of the four yield strengths as shown in Table A1 of the 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 C7, Equations 3.3.4-1, 3.3.4-2, and 3.3.4-3 are used for end one-flange loading; Equations 3.3.4-4 and 3.3.4-5 are used for interior one-flange loading; Equations 3.3.4-6 and 3.3.4-7 are used for end two-flange laoding; and Equations 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 carbon steel sections (Reference 38). The assumed distributions of loads or reactions in the beam webs are shown in Figure C8.

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 38 in conjunction with proper safety factors used for the design of 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 C9. 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 occuring under concentrated loads on single span beams. This post-buckling strength en-

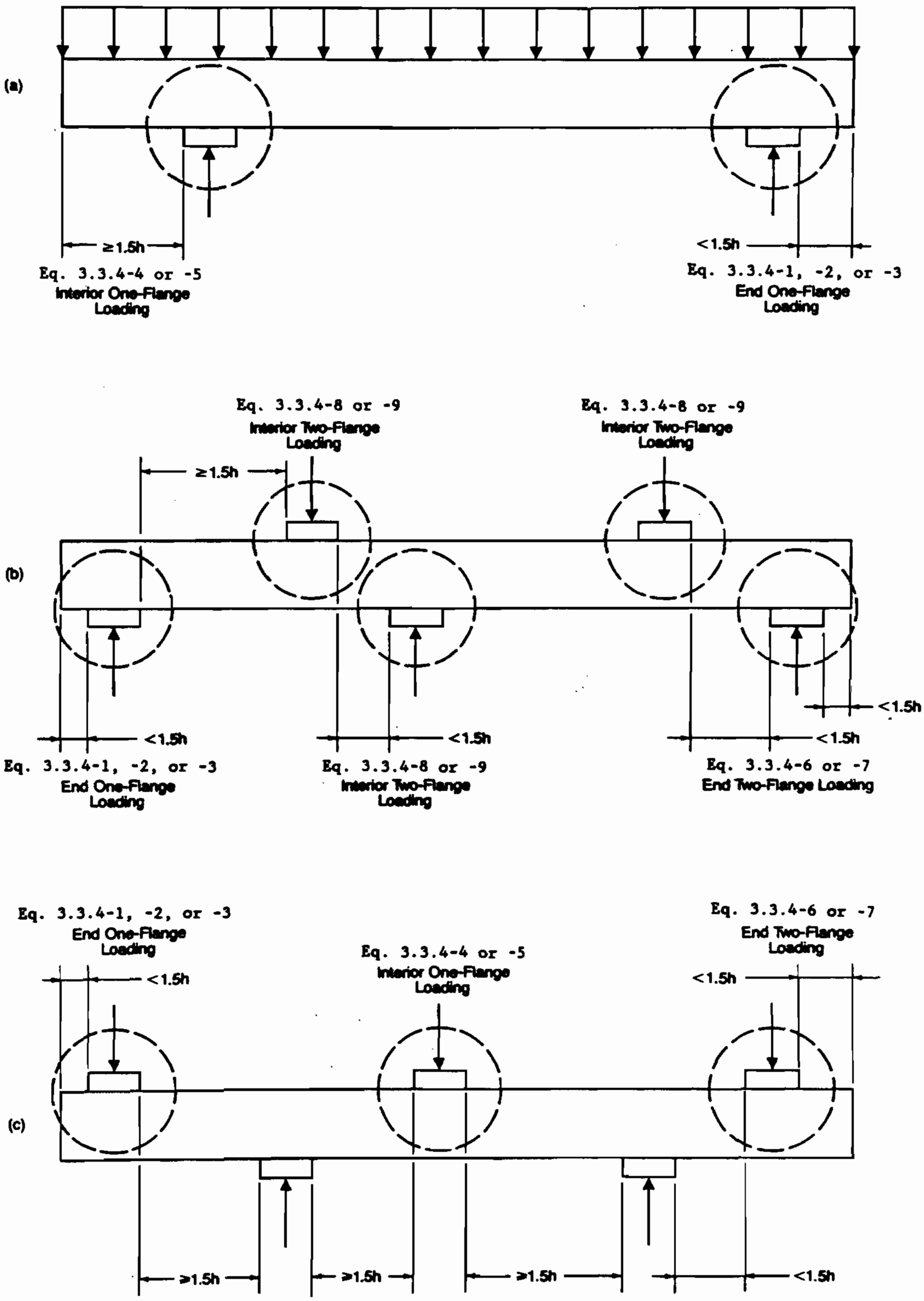

Figure C7 Application of Design Equations Listed in Table 1

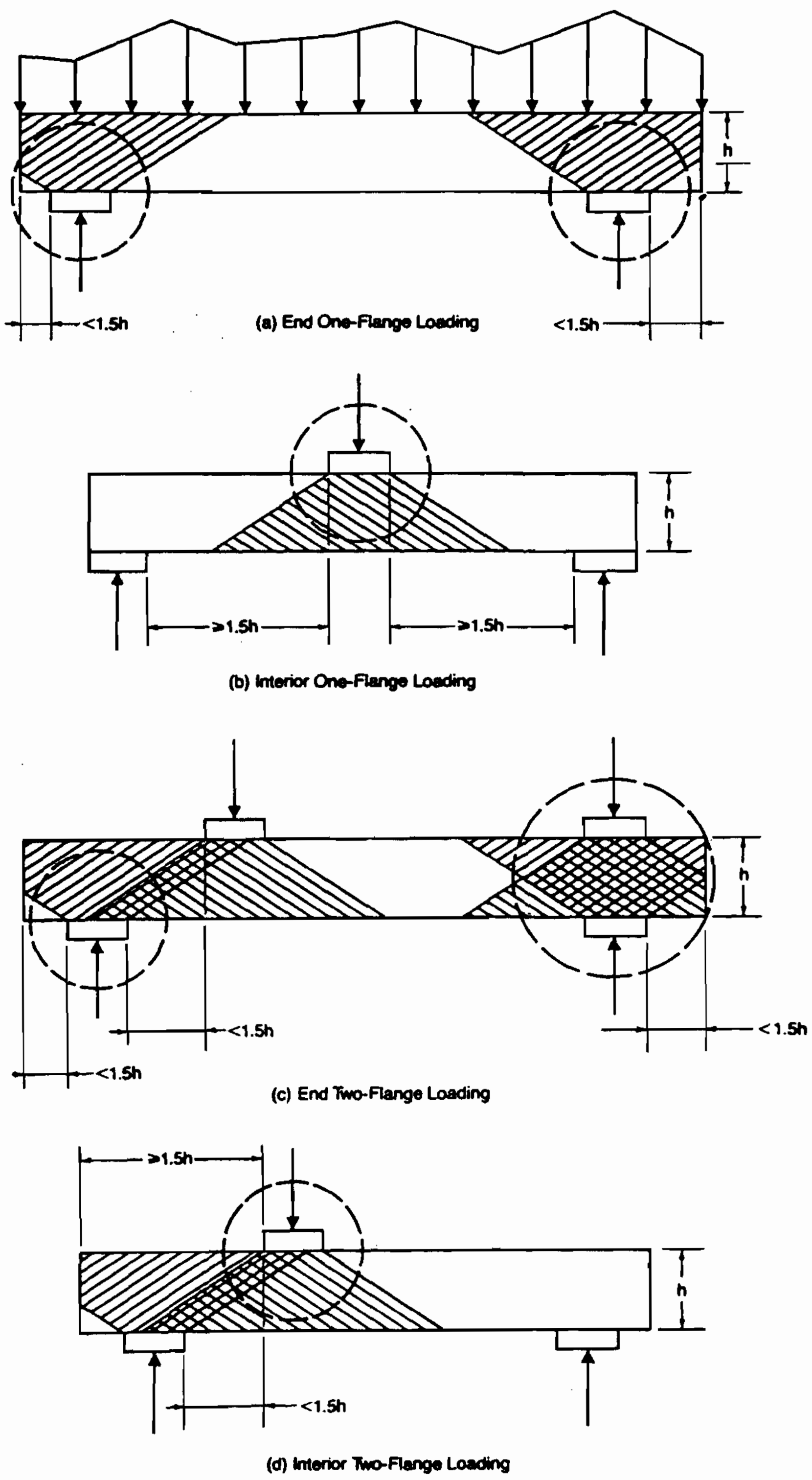

Figure C8 Assumed Distribution of Reaction or Load

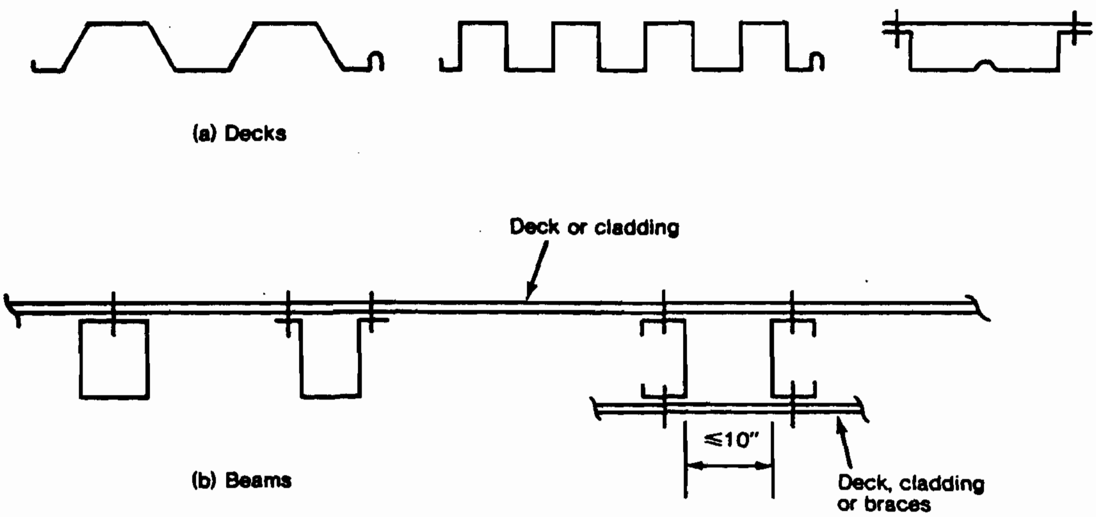

Figure C9 Sections Used for One Exception Clause of Section 3.3.5

ables the member to redistribute the moments in continuous spans. For this reason, Equation 3.3.5-1 is not applicable to the interaction between bending and the reaction at interior supports of continuous spans.

With regard to Equation 3.3.5-2, it was shown in Reference 38 that when the h/t ratio of an I-beam web does not exceed $2.33/\sqrt{F_y/E_o}$ and when $\lambda \leq 0.673$ (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 provisions of this Section of the Specification represent a significant improvement over the 1974 Edition of the AISI Specification for concentrically loaded compression members subject to flexural, torsional, or torsional-flexural buckling. The allowable load for compression members can be determined on the basis of Equation 3.4-1 of the Specification with due consideration given to various possible failure modes.

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 allowable 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.[11]

The safety factor applied for axially loaded compression members is 2.15, which is the same as that used in the 1974 Edition of the AISI Specification.[10] This safety factor used for cold-formed stainless steel compression members is slightly larger than that used for cold-formed carbon steel. It maintains a constant ratio of the corresponding safety factors between two specifications.[10,11]

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.

Equation 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 4, 5 and 7. The value of effective length factor, K, may be obtained from Reference 27.

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 new 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 Secitons 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, centrally loaded columns can buckle by (a) bending about one of the principal planes, (b) twisting about the shear center, or (c) bending and twisting simultaneously. Case (c) is the type of tortional-flexural buckling which can occur at a load lower than the flexural buckling load.

The new 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 crirical stress for torsional-flexural buckling. They are adopted from the AISI Specification for carbon steel with some necessary modifications.[11]

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 warrent computerization. Section 3.4.4 of the Specification provides that rational analysis shall be used, or tests according tó Section 6 shall be made, when dealing with nonsymmetrical open shapes.

3.5 Combined Axial Load and Bending

This provision is adopted from Reference 11 for the design of cold-formed carbon steel members, except that the tangent modulus, E_t, is used to calculate the critical buckling load. A recent study reported in Reference 30 indicates that the interaction equations (Equations 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 provisions for cylindrical tubular members have been revised to include the design guidelines for members subject to either flexural bending or axial compression.

3.6.1 Bending

This section is similar to that included in the 1974 Edition of the AISI Specification except for the following revisions: (a) the design expressions are used in terms of moment instead of stress and (b) the determination of buckling stress in the inelastic range is based on the actual ratio of the effective proportional limit-to-yield strength for different grades of stainless steels instead of $F_{pr}/F_y = 0.375$, which was used previously for the design of stainless steel cylindrical tubular members.[10]

It should be noted that the definition of D, outside diameter of the cylindrical tubular memebrs, differs from the previous Specification. The limit of D/t is the same as that used in Reference 10, except that it has been rewritten by using a non-dimensional term. The variable K_c is determined according to the ratio of the proportional limit-to-yield strength. It is a ratio of the nominal moment, M_n, to the yield moment, M_y, in the inelastic buckling range. Figure C10 shows the relationships between the ratio of the nominal moment to the yield moment, M_n/M_y, and

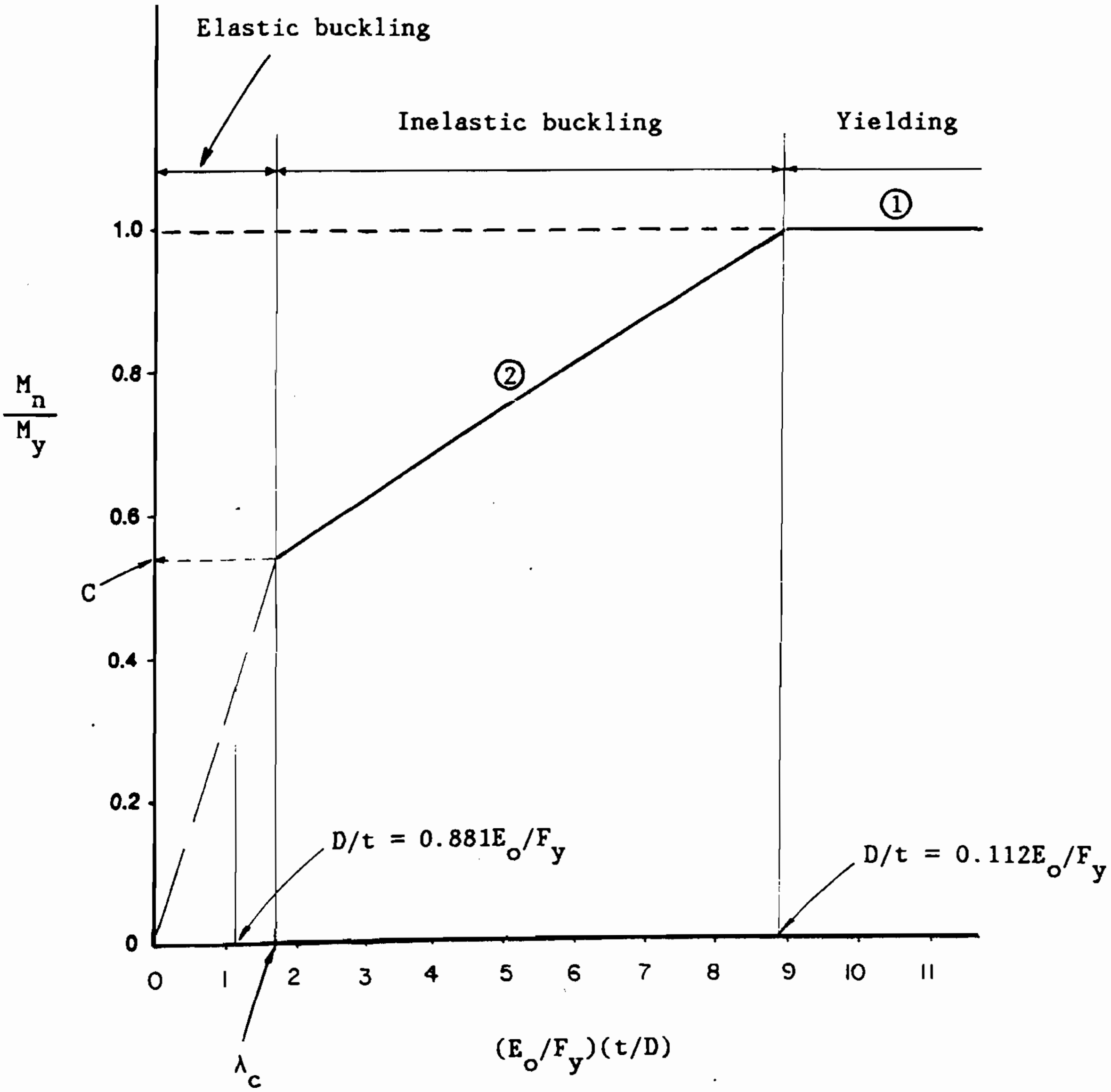

Figure C10 Ultimate Moment Capacity of Stainless Steel Cylindrical Tubes the ratio of $(E_o/F_y)(t/D)$. Line 1 shown in the figure represents the ultimate moment governed by yielding. Line 2 corresponds to the nominal moment in the inelastic buckling range.

3.6.2 Compression

The 1974 Edition of the AISI Specification did not include any design provisions for the interaction of local and column buckling. The provisions of Specification Section 3.6.2 use 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 11 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 the 1974 Edition of the AISI Specification,[10] except that the requirement is expressed in terms of the allowable moment instead of allowable stress.

Arc-and-tangent type corrugated sheets and panels are often used for roofing, siding and curtain walls. Previous tests[40] on carbon steel sections indicated that the moment-resisting capacities of such sections were about 20 percent 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 allowable moment of 0.60 F_yS_f is permitted in Section 3.7 for arc-and-tangent corrugated sheets.

The use of such a working moment would provide a load factor of approximately 2.0 based on ultimate strength of the section.

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 allowable 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 the current Specification are the same as that included in the 1974 Edition of the AISI Specification.

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.[27,41]

Even though Section 4.1 of the Specification refers only to I-sections, Equation 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 (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 C11 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 42.

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,[10] 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 C12 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.

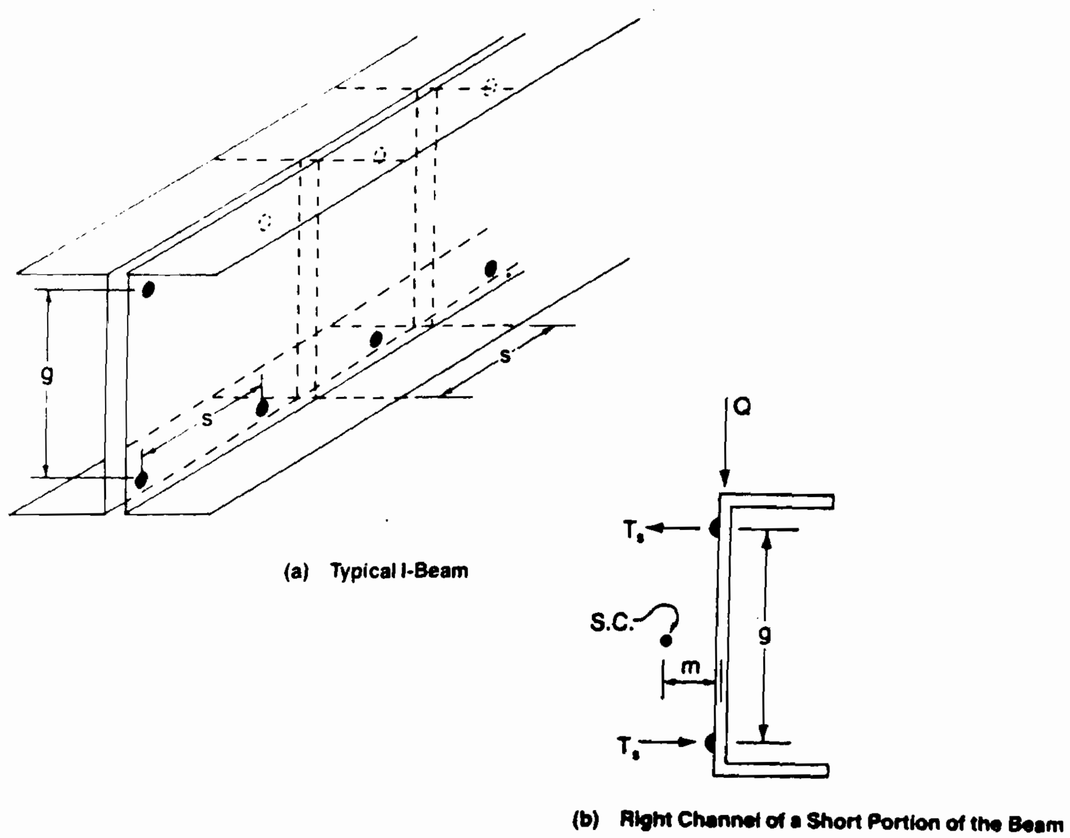

Figure C11 I-Beam Composed of Two Channels

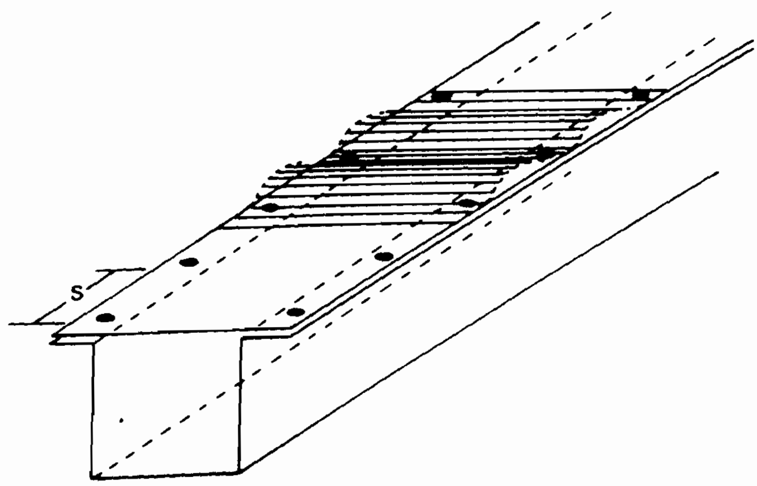

Figure C12 Spacing of Connections in Compression Elements

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 C12) 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, $\sigma_e = \pi^2 E/(KL/r)^2$, by substituting $\sigma_e = 1.85f$, $E_t = E$, $K = 0.6$, $L = s$, and $r = t/\sqrt{12}$.

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 11, 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 given in Reference 11. References 41 and 43 provide further information for this topic.

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[44-48] 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.[11]

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 Part II of Reference 43.

4.3.2.1.1 Type and Spacing of Braces

The new provision of Section 4.3.2.1.1 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. However, a factor of safety of 1.85 should be used, instead of that required by Section 6 for determining structural performance. This is because the beam member itself is analytically designed for a safety factor of 1.85 for cold-formed stainless steel structural members.

4.3.2.2 Neither Flange Connected to Sheathing

These provisions are similar to those included in the 1974 Edition of the AISI Specification,[10] 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 41.

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,[10] except that the requirement is expressed in a dimensionless unit. A brief discussion on lateral buckling of box beams can be found in Part II of Reference 43.

5. CONNECTIONS AND JOINTS

5.1 General Provisions

The Specification contains provisions only for welded and bolted connections. Other means of connection, e.g., rivets, screws and special devices are proprietary devices for which information on the strength of the connection must be obtained from 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 provisions contained in this Section are primarily based on the experimental evidence obtained from the test program at Cornell University for welded and bolted connections using stainless steels.[9,49]

5.2 Welded Connections

The design provisions for welded connections are developed on the basis of the research findings reported in References 9 and 49 and the AISI Specification for cold-formed carbon steel.[11] The 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 butt joints, twenty-four specimens have been tested and evaluated.[9,49] The allowable design stresses in tension and compression are similar to those included in Reference 10, except that the factor of safety used in this provision against fracture of the annealed based metal is 2.50 instead of 2.20 used in the 1974 Edition of the AISI Specification. This change is mainly for the sake of uniformity in safety factors between the Specifications for carbon and stainless steels.

The use of different factors of safety for butt welds is based on the consideration that a larger factor of safety should be provided for fracture than for overall yielding. Furthermore, a larger factor of safety is selected for fracture of weld metal because of the less certain properties of deposited weld metal.

5.2.2 Fillet Welds

For fillet welded connections, Reference 49 indicates that the design should be based on (a) fracture of the annealed base metal and (b) 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 have been included for fillet welds subjected to longitudinal direction and transverse direction loadings. The safety factor used for this provision is 2.50 against fracture. These changes are mainly based on References 11 and 49.

As compared with the 1974 Edition of the AISI Specification for stainless steel design, it can be seen that in the current Specification, the allowable load for longitudinal fillet welds due to fracture of the annealed base metal is slightly less than that used in the previous AISI Specification. However, for transverse loading, the allowable load due to fracture of the weld metal is slightly larger than that used in the previous AISI Specification.

The tensile strengths of the weld metal listed in Table A15 of the Specification are based on References 50 and 51. Table C10 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 20. For Types 409, 430 and 439, the tensile strengths of base metals listed in Table A16 are adapted from References 17 and 18.

5.2.3 Resistance Welds

The provisions of the current Specification are the same as those used in the 1974 Edition of the AISI Specification, except that for pulsation welding, the allowable shear strength is specified for 1/4- and 1/2- hard tempers, separately.

Table 2 of Section 5.2.3 includes allowable shear strengths for annealed, 1/4- and 1/2- hard temper materials. These data were adopted from the AWS Recommended Practices of Resistance Welding.[52] A safety factor of 2.50 was used to determine the allowable shear strength for spot welding. 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 3 of Specification Section 5.2.3 on pulsation welding, allowable 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 thickner than about 0.125 inch would not be available from most steel producers.

5.3 Bolted Connections

This provision contains some general requirements for bolt installation and maximum sizes of holes. Even though no specific requirements

TABLE C10

AWS Requirements for Mechanical Property of All-Weld-Metal[50,51]

A W S Classification	Elongation in 2 in. gage length min., percent	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

TABLE C10 (cont'd)

AWS Requirements for Mechanical Property of All-Weld-Metal[50,51]

A W S Classification	Elongation in 2 in. gage length min., percent	Heat Treatment
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

1 in. = 25.40 mm

a Specimen shall be heated to between 1550° and 1600° F (840° and 870° C), held for 2 hr., furnace-cooled at a rate not exceeding 100° F (55° 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 a rate not exceeding 100° F (55° 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.

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 shear and tension in bolts.

5.3.1 Spacing and Edge Distance

The provisions for minimum spacing and edge distance have been revised in the current Specification. The minimum edge distance of each individual connected part, e_{min}, is now determined by using the tensile strength of the connected part in the longitudinal direction (F_u), instead of the average yield strength ($F_{y,av}$). The design equation is based on the following basic formula established from the test results:[49]

$$e_{req'd} = P / F_u t \qquad \text{(Eq. C-6)}$$

in which $e_{req'd}$ is the required minimum edge distance to prevent shear failure of the connected part for the force, P, transmitted by one bolt, and t is the thickness of the thinnest connected part. The safety factor used in this provision is the same as that included in Reference 10.

The design requirements for bolted connections with standard, oversized and slotted holes are included in this provision.

5.3.2 Tension in Connected Part

In the revised design criteria, the formula 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. 5.3.2-3 and 5.3.2-4, F_u is taken as the tensile strength in the longitudinal direction instead of the basic design stress used in Reference 10. The safety factor used to determine the allowable tensile load on net section is 2.40 for double shear connections and single shear connections. It provides a consistent safety factor for the design of cold-formed stainless steels.

5.3.3 Bearing

In the 1974 Edition of the AISI Specification, the allowable bearing stress was determined on the basis of the average yield strength of the connected parts. In the current design provision, it is based on the tensile strength of the connected part in the longitudinal direction.[49] Based on the test results presented in Reference 49, different nominal bearing stresses are specified for single and double shear connections. The safety factor used in this provision is 2.40 instead of 2.57 used in Reference 10.

5.3.4 Shear and Tension in Bolts

In the current Specification, the allowable shear stresses for stainless steel bolts are given in Table A17 of the Specification. This table was prepared on the basis of References 9, 19 and 53. The allowable shear stress for bolts with no threads in the shear plane was taken as 60 percent of the minimum tensile strength divided by a safety factor of 3.0. This allowable shear stress provides a minimum safety factor of about 1.2 against shear yielding of bolt material. When threading is included in the shear plane, 70 percent of the nominal allowable shear stress is used due to the fact that the actual shear stress in bolts is to be calculated on the basis of the gross cross-sectional area or 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 1/4 to 3/4 inch. This practice is comparable to that for high-strength carbon steel structural bolts. For the bolts not listed in Table A17, the allowable shear stresses can be determined in the same manner.

In the same table, the allowable tension stresses for stainless steel bolts are added. When bolts are subjected to a combination of shear and tension, the reduced allowable tension stresses, F'_t, are given in Table A18.

The allowable tension strengths for bolts are determined on the basis of the minimum tensile strength of materials divided by a safety factor of 3.0. The design equations given in Table A18 for determining F'_t are derived from the following equation:

$$F'_t = 1.25\ F_t - A\ f_v \qquad \text{(Eq. C-7)}$$

where

F'_t = Reduced allowable tension stress for bolts subjected to a combination of shear and tension

F_t = Allowable tension stress for bolts subject only tension

A = 1.8 for threads not excluded from shear planes

= 1.4 for threads excluded from shear planes

f_v = Shear stress in bolt

5.4 Shear Rupture

This provision is adopted from Reference 11. It contains the allowable shear force due to shear rupture failure of cold-formed stainless steel structural members. The factor of safety used in this provision for stainless steel beams is 2.22 instead of 2.00 used for cold-formed carbon steel members.

6. TESTS

This section covers the requirements of tests to determine material properties and structural performance for cold-formed stainless steel structural members. These provisions do not apply to the panels used as shear diaphragms. A general discussion of structural diaphragms is given in References 41 and 54.

6.1 Determination of Stress-Strain Relationships

This provision is the same as that included in the 1974 Edition of the AISI Specificaiton,[10] 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 this Section, or for obtaining detailed mechanical properties for special cases when necessary. Discussions of the statistical approaches required are contained in References 4 and 7.

6.2 Tests for Determining Structural Performance

This provision is adopted from Reference 11, except that the dead load factor used in Specification Eq. 6.2-2 is 2.0 instead of 1.5, which is used in Reference 11. 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 provision of this Specification.

Many cold-formed stainless steel structural applications have different composition or configuration, which are not covered by the provisions of this Specification; their performance and adequacy, therefore, cannot be demonstrated by the Specification. For example, apart from those methods of connection coverd in the Specification, a number of other means of connecting are in use. The fact that these are not specifically covered in the Specification is not intended to exclude their use. However, since structures so connected cannot be calcualted according to the Specification (at least as to strength of connections), tests according to Section 6 are the only means of supplying proof of structural adequacy. Other similar examples could be cited.

Provision (b) prescribes that the structure, when tested, shall support the design loads increased by load factors, without failure or harmful distortions. This is to provide reasonable safety factors against failure or harmful distortions. The safety factor thus obtained is somewhat larger than the basic factor of safety of 1.85 on which the body of the Specification is based. This takes into account the uncertainties in translating test results into reliable carrying capacities as mentioned above. Additionally, in some cases, the factors of safety in the Specification are larger than 1.85 as can be seen in Table C8 of the Commentary. For connections, the factors of safety implied in the Specification are significantly larger than 2. Therefore, provision (b) is also aimed at ensuring factors of safety consistent with those implied in the Specification for connections.

Provision (c) applies when the strength of the test specimen is greater than the minimum specified steel 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 Confirming Structural Performance

Members, connections and assemblies which can be designed according to the provisions of Sections 1 through 5 of the Specification, need no confirmation of calculated results by test. However, special situations may arise where it is desirable to confirm by test the results of calculations. Tests may be called for by the manufacturer, the engineer, or a third party.

Since design was in accordance with the Specification, the higher load factors of Specification Section 6.2 do not apply. All that is needed is that the tested specimen or assembly perform according to the safety factor implied in the applicable specification provision. The appropriate value of the applicable safety factor can generally be obtained from the Specification or Commentary Table C8.

6.4 Tests for Determining Mechanical Properties of Full Sections

Method for utilizing the effects of cold work are incorporated in Section 1.5.2.2 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 and 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 1.5.2.2. For details of testing procedures which have been used for such purpose, but which in no way should be regarded as mandatory, see References 55 and 56.

APPENDICES

Appendix A Design Tables and Figures

In the current 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, 316, 409, 430, and 439 stainless steels. In addition, design tables for welded and bolted 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 stainless steels for transverse tension are based on the values specified in ASTM A666-84.[20] Other yield strengths for longitudinal tension, transverse compression, and longitudinal compression are derived from the yield strengths recommended in Tables C3, C4 and C5 of the Commentary.

For Type 409 stainless steel, the yield strengths for different types of stress are adapted from ASTM 176-85a.[17] These values have been verified by the test data provided in Refs. 21 and 22.

For Types 430 and 439 stainless steels, the yield strengths listed in Table A1 are adapted from ASTM 240-86.[18] The values specified in the ASTM Specification are the minimum tensile yield strengths in the transverse direction. The test data given in Tables C7 and C8 of the Commentary are larger than those specified in Ref. 18. These ASTM specified minimum strengths are used in the Specification for design purposes due to the consistency with other types of stainless steels.

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 4.

Tables A2 through A14 and Figures A1 through A12 provide 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 the 1974 Edition of the AISI Specification.[10] For Types 409, 430 and 439, the mechnical properties are based on Refs. 21, 22 and 57.

For the design of welded connections, the tensile strengths of weld metals listed in Table A15 are based on Refs. 50 and 51.

The tensile strengths of annealed base metals listed in Table A16 for Types 201, 301, 304 and 316 are adapted from ASTM A666-84.[20] In the same table, the tensile strengths of base metals for Types 409, 430 and 439 are based on ASTM 176-85a and ASTM A240-86.[17,18]

Table A17 gives the allowable shear and tension stresses for stainless steel bolts. Table A18 provides the reduced allowable tension stresses, F'_t, for bolts subjected to a combination of shear and tension. Discussions of the shear and tension stresses in stainless steel bolts are described in Commentary Section 5.3.4.

Table A19 lists the ratios of the effective proportional limit-to-yield strength. The vlaue of F_{pr}/F_y for Types 201, 301, 304 and 316 stainless steels are the same as that used in the 1974 Edition of the AISI Specification. For Types 409, 430 and 439, the ratios of F_{pr}/F_y are determined on the basis of Ref. 21.

Appendix B Flange Curling

For detailed discussions, see Commentary Section 2.1.1(b).

Appendix C Shear Lag Effects

For detailed discussions, see Commentary Section 2.1.1(c).

Appendix D Stiffeners

For detailed discussions, see Commentary Section 2.6.

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