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Load and resistance factor design of cold-formed steel : tentative recommendation - load and resistance factor design criteria for cold-formed steel structural members and commentary thereon

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Civil Engineering Study 80-1
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Sixth Progress Report

LOAD AND RESISTANCE FACTOR DESIGN OF COLD-FORMED STEEL

TENTATIVE RECOMMENDATIONS - LOAD AND RESISTANCE
FACTOR DESIGN CRITERIA FOR COLD-FORMED STEEL
STRUCTURAL MEMBERS AND COMMENTARY THEREON

by

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A Research Project Sponsored by American Iron and Steel Institute

March 1980

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FORWARD

This progress report contains the following two parts:

Part I: Tentative Recommendations - Load and Resistance
Factor Design Criteria for Cold-Formed Steel Structural
Members (pp. i-47).

Part II: Commentary on Tentative Recommendations - Load and
Resistance Factor Design Criteria for Cold-Formed
Steel Structural Members (pp. 48-86).

The tentative design recommendations are based on the statistical data presented in previous progress reports and the newly revised load factors used in Section 8.3.4 of the design criteria. The selections of ϕ factors are discussed in the Commentary for various types of structural members and connections.

This investigation was sponsored by American Iron and Steel Institute. The technical guidance provided by the AISI Task Group on Load and Resistance Factor Design (K.H. Klippstein, Chairman, D. H. Hall and R. L. Cary, members), the advisors for the AISI Task Group (R. Bjorhovde, C.W. Pinkham, R.M. Schuster, and G. Winter), former members of the AISI Task Group (N.C. Lind, R.B. Matlock, W. Mueller, F.J. Phillips, and D.S. Wolford), the AISI Staff (A.L. Johnson and D.P. Cassidy) and our consultant, M.K. Ravindra, is gratefully acknowledged.

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PART I

Tentative Recommendations
LOAD AND RESISTANCE FACTOR DESIGN CRITERIA
FOR
COLD-FORMED STEEL STRUCTURAL MEMBERS

PREFACE

The "Allowable Stress Design Method" has long been used for the design of cold-formed steel structural members. The "Load and Resistance Factor Design Method" has recently been developed from a research project sponsored by American Iron and Steel Institute. In this method, separate load and resistance factors are applied to specified loads and nominal resistance to ensure that the probability of reaching a limit state is acceptably small. These factors reflect the uncertainties of analysis, design, loading, material properties and fabrication. They are derived on the basis of the first order probabilistic methodology as used for the development of the LRFD recommendations for hot-rolled steel shapes for buildings.

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

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NOTATION

Symbol	Definition	Section
A	Cross-sectional area, in. ²	3,6, 3,7, 9.4
A	Area of cross section of stud, in. ²	5.1
A _{ef}	Effective area of the stiffener section, in. ²	2.3.1.2, 8.4.1
A _{eff}	Effective area as determined for the effective design widths, in. ²	9.4.1
A _I	Influence area, ft ²	8.3.3
A _n	Net area of the cross section, in. ²	9.2
A _n	Net area of the connected part, in. ²	10.3.3
A _{sA}	Stress area when threading is included in shear plane; gross area when threading is excluded from shear plane, in. ²	10.3.5
A _T	Tributary area, ft ²	8.3.3
A _w	Area of beam web, in. ²	9.3.3.1
a	Actual spacing of attachments of wall material to stud measured along the length of stud, (a = 1 for continuous attachment), in.	5.1
a	Length of bracing interval, in.	5.2
B _c	Term for determining the tensile yield point of corners	3.1.1.1, 9.1.1.1
b	Effective design width of stiffened elements, in.	2.3.1, 8.4.1.1 8.4.1.2
b _e	Effective design width of sub-element or element in.	2.3.1.2, 8.4.1.2
C	Ratio of total corner area to total cross-sectional area of full section or full flange section	9.1.1.1

C_b	Bending coefficient dependent upon the moment gradient	3.3, 9.3.2
C_c	$\sqrt{2\pi^2 E/F_y}$	3.6.1.1, 9.4.1
C_m	End moment coefficient in interaction formula	3.7, 9.5
C_{TF}	End moment coefficient in interaction formula	3.7.2, 9.5.2
C_w	Warping constant of torsion of the cross section, in. ⁶	3.6.1.2, 9.4.2
c	Distance from the centroidal axis to the fiber with maximum compression stress, negative when the fiber is on the shear center side of the centroid, in.	3.7.2, 9.5.2
c_f	Amount of curling, in.	2.3.3, 8.4.3
D	Mean diameter of cylindrical tube, in.	3.8, 9.6
D_n	Nominal dead load, psf	8.3.4
d	Depth of section, in.	2.3.3, 3.3, 3.7, 4.3, 5.2, 8.4.3, 9.3.2, 9.5
d	Diameter of bolt, in.	4.5, 10.3.3
d_l	Overall depth of lip, in.	4.3
d_{min}	Overall minimum depth required of simple lip, in.	2.3.2.1
E	Modulus of elasticity of steel = 29,500 ksi	3.3, 3.6.1, 3.7, 5.1, 9.3.2, 9.4
E_n	Nominal earthquake load	8.3.4
e	Eccentricity of the axial load with respect to the centroidal axis, negative when on the shear center side of the centroid, in.	3.7.2, 9.5.2

e	The distance 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, in.	10.3.2
F	Basic design stress on the net section of tension members and tension and compression on the extreme fibers of flexural members, ksi	3.1
F_a	Maximum average compression stress, ksi	3.6.2, 3.7.2
F_{ac}	Average allowable compression stress determined by both requirements (i) and (ivb) of 3.7.2 if the point of application of the eccentric load is at the shear center, ksi	3.7.2
F_{aE}	Average allowable compression stress determined by requirement (i) of 3.7.2 if the point of application of the eccentric load is at the shear center, ksi	3.7.2
F_{ao}	Allowable compression stress under concentric loading determined by 3.6.1.1 for $L = 0$, ksi	3.7
F_{as}	Allowable compression stress under concentric loading on bracing and secondary members, ksi	3.6.2
F_{a1}	Allowable compression stress under concentric loading, ksi	3.6.1.1, 3.7
F_{a2}	Allowable compression stress under concentric loading, ksi	3.6.1.2, 3.7.2
F_b	Maximum compression stress on extreme fibers of laterally unbraced beams, ksi	3.3
F_b	Maximum bending stress in compression that is permitted where bending stress only exist (3.1, 3.2, 3.3), ksi	3.7

F_{bw}	Maximum allowable compression stress in the flat web of a beam due to bending, ksi	3.4.2, 3.4.3
F'_e	$\frac{12\pi^2 E}{23(KL_b/r_b)^2}$, ksi	3.7.1, 3.7.2
F_{Exx}	Specified nominal tensile strength of the weld metal, ksi	10.2.1.2
F_{ya}	Average yield point of section, ksi	3.1, 9.1.1.1
F_{yc}	Tensile yield point of corners, ksi	3.1.1.1, 9.1.1.1
F_{yf}	Weighted average tensile yield point of the flat portions, ksi	3.1.1.1, 6.3.2 9.1.1.1
F_{bl}	Maximum bending stress in compression permitted where bending stress only exist and the possibility of lateral buckling is excluded (3.1, 3.2), ksi	3.7.1, 3.7.2
F_c	Maximum allowable compression stress on unstiffened elements, ksi	3.2, 4.4
F_r	Allowable compression stress in cylindrical tubular member, ksi	3.8
F_u	Ultimate tensile strength of virgin steel, ksi	3.1.1.1, 9.1.1.1
F_u	Nominal tensile strength of the connected part, ksi	10.3.2
F_v	Maximum allowable average shear stress on the gross area of a flat web, ksi	3.4.1, 3.4.3
F_y	Specified minimum yield point, ksi	2.2, 2.3, 3.4.4, 4.5, 5.1, 5.3, 8.2, 8.4, 9.1.1
\bar{F}_y	Yield stress determined by either the minimum specified yield point or the average yield stress, F_{ya} , when the increase in strength resulting from cold forming is utilized, ksi	8.5, 9.1, 9.1, 9.3, 9.4, 9.5, 9.6
f	Actual stress in the compression element computed on the basis of effective design width, ksi	2.3.1.1, 4.4
f_a	Axial stress = axial load divided by full cross sectional area of member, P/A , ksi	3.7

f_{av}	Average stress in the full unreduced flange width, ksi	2.3.3, 8.4.3
f_b	Maximum bending stress = Bending moment divided by section modulus of the member, ksi	3.7
f_{bw}	Actual compression stress at the junction of flange and web, ksi	3.4.3
f_v	Actual shear stress, ksi	3.4.3
f_{max}	Actual stress in the compression element computed on the basis of effective design width and the factored load, ksi	8.4.1.1
G	Shear modulus = 11,300 ksi	9.4.2
g	Vertical distance between two rows of connections near or at top and bottom flanges, in.	4.3
h	Clear distance between flanges measured along the plane of the web, in.	2.3.4, 3.4, 3.5, 8.4.4, 9.3
I_{min}	Minimum allowable moment of inertia of stiffener of any shape about its centroidal axis parallel to the stiffened element, in. ⁴	2.3.2.1, 8.4.2.1
I_s	Moment of inertia of a multiple-stiffened element, in. ⁴	2.3.2.2, 8.4.2.2
I_x	Moment of inertia about axis normal to the web, in. ⁴	4.3, 5.2
I_{xc}	Moment of inertia of the compression portion of a section about its axis of symmetry, in. ⁴	3.7.2, 9.5.2
I_{xy}	Product of inertia, in. ⁴	5.2.2
I_y	Moment of inertia about y-axis, in. ⁴	3.7.2, 9.5.2
I_{yc}	The moment of inertia of the compression portion of a section about the gravity axis of the entire section parallel to the web, in. ⁴	3.3, 9.3.2

I_1	Moment of inertia of cross section of stud about its axis parallel to wall, in. ⁴	5.1
I_2	Moment of inertia of cross section of stud about its axis perpendicular to wall, in. ⁴	5.1
J	St. Venant torsion constant of the cross section, in. ⁴	3.6.1.2, 9.4.2
j	Section property, torsional-flexural buckling, in.	3.7.2, 9.5.2
K	Effective length factor	3.6.1, 3.7, 9.4, 9.5
K'	A constant; for channel = m/d , for Z-section = I_{xy}/I_x	5.2.2
K_w	Modulus of elastic support of wall material and its attachments, kip per in.	5.1
L	Full span for simple beams, distance between inflection points for continuous beams, twice the length of cantilever beams, in.	2.3.5, 4.3b 8.4.5
L	Unbraced length of member, in.	3.3, 3.6, 4.3a, 9.4.2, 9.4
L	Length of stud, in.	5.1
L	Weld length, in.	10.2.1.1
L_b	Actual unbraced length in the plane of bending, in.	3.7, 9.5
L_C	Basic live load, psf	8.3.3
L_n	Nominal live load, psf	8.3.3, 8.3.4
l_i	length of middle line of segment i, in.	3.6.1.2, 9.4.2
M_c	Elastic critical moment causing compression on the shear center side of the centroid, kip-in.	3.7.2, 9.5.2
M_D	Bending moment at or immediately adjacent to P_D computed on the basis of factored loads, kip-in.	9.3.3.3, 9.3.3.4, 9.5.2

M_T	Elastic critical moment causing tension on the shear center side of the centroid, kip-in.	3.7.2, 9.5.2
M_u	Nominal ultimate bending moment if moment only exist, kip-in.	9.3.3.4
M_{ubw}	Nominal maximum bending moment governed by the buckling strength of the beam web, kip-in.	9.3
M_{uc}	Beam capacity as determined from Sections 9.3.1 and 9.3.2, whichever is smaller, kip-in.	9.5
M_{us}	Beam strength as determined from Section 9.3.1, kip-in.	9.5
M_1	Smaller end moment	3.3, 3.7, 9.3.2 9.5.3
M_2	Larger end moment	3.3, 3.7, 9.3.2 9.5.3
m	Term for determining the tensile yield point of corners	3.1.1.1, 9.1.1.1
m	Distance of shear center of channel from mid - plane of the web, in.	4.3, 5.2.2
m	Number of shear planes per bolt	10.3.5
N	Actual length of bearing, in.	3.5, 9.3.3.4
P	Total load on compression members, kips	3.6.1.1
P	Concentrated load or reaction, kips	4.3, 5.2.2
P	Force transmitted by bolt, kips	4.5
P	Force which produces an elongation of e inches in a strip of wall material of width a and of length equal to the distance between adjacent studs, kips	5.1
P_D	Concentrated load or reaction based on factored load, kips	9.3.3.4
P_L	Force to be resisted by intermediate beam brace, kips	5.2

P_{\max}	Allowable concentrated load or reaction to avoid crippling of flat webs of beams, kips	3.5
P_{\min}	Lateral force which each single attachment of the wall material shall be capable of exerting on the stud in the plane of the wall, kips	5.1
P_s	Design load on stud, kips	5.1
P_n	Nominal ponding load	8.3.4
P_{ur}	AF_{cr} , kips	9.6.2
P_u	Nominal ultimate concentrated load or reaction, kips	9.3.3.4, 9.5
Q	Stress and/or area factor to modify allowable axial stress	3.6.1.1, 9.4.1
Q_a	Area factor to modify members composed entirely of stiffened elements	3.6.1.1, 9.4.1
Q_s	Stress factor to modify members composed entirely of unstiffened elements	3.6.1.1, 9.4.1
q	Intensity of loading on beam, kips per in.	4.3
R	Inside bend radius, in.	3.1.1.1, 3.5, 9.1.1.1, 9.3.3.4
r	Radius of gyration, in.	3.6, 9.4.1
r	Bolt force ratio	4.5.2, 10.3.3
r_b	Radius of gyration about axis of bending, in.	3.7, 9.5.3
r_{cy}	Radius of gyration of one channel about its centroidal axis parallel to web, in.	4.3
r_o	Polar radius of gyration of cross section about shear center, in.	3.6.1, 3.7.2, 9.4.2
r_x	Radius of gyration of cross section about centroidal principal axis, in.	3.6.1.2, 9.4.2
r_{xc}	Radius of gyration about the centroidal axis parallel to the web of that portion of the I-	3.7.2, 9.5.3

	section which is in compression when there is no axial load, in.	
r_y	Radius of gyration of cross section about centroidal principal axis, in.	3.6.1.2, 3.7.2 9.4.2
r_1	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, if any, in.	4.3
r_2	Radius of gyration of stud about its axis perpendicular to the wall, in.	5.1
S_n	Nominal snow load	8.3.4
S_c	Elastic section modulus of entire section about axis of bending, I/distance to extreme compression fiber, in. ³	9.3
S_{eff}	Elastic section modulus of effective section, in. ³	9.3
s_{max}	Maximum permissible longitudinal spacing of welds or other connectors joining two channels to form an I-section, in.	4.3
S_{xc}	Compression section modulus of entire section about major axis, in. ³	3.3, 9.3.2
S_{yc}	Compression section modulus of entire section about axis normal to axis of symmetry, in. ³	3.7.2, 9.5.3
s	Spacing of connections in line of stress, in.	4.4
s	Spacing of bolts perpendicular to the line of stress, in.	4.5.2, 10.3.3
T_s	Strength of connection in tension, kips	4.3
t	Base steel thickness at any element or section, in.	2.2, 2.3, 3.2, 3.4, 3.5, 3.8, 4.4, 4.5, 8.2, 8.4, 9.3, 9.6

t	Thickness of connected part, in.	10.3.2
t_w	Effective throat dimension of weld, in.	10.2.1.1
t_i	Steel thickness for member of segment i, in.	3.6.1.2, 9.4.2
t_s	Equivalent thickness of a multiple-stiffened element, in.	2.3.2.2, 8.4.2.2
V_u	Nominal maximum shear strength of the unreinforced flat beam webs, kips	9.3.3.1, 9.3.3.3
w	Flat width of element exclusive of fillets, in.	2.2, 2.3, 3.2, 8.2
W_n	Nominal wind load	8.3.4
w_f	Projection of flanges from inside face of web, in.	4.3
w_f	Width of flange projection beyond the web or half the distance between webs for box-or U-type sections, in.	2.3.3, 2.3.5, 8.4.3, 8.4.5
w_s	Whole width between webs or web to edge stiffener in.	2.3.2.2, 8.4.2.2
x	Distance from concentrated load to brace, in.	5.2
x_o	Distance from shear center to centroid along the principal x-axis, in.	3.6.1.2, 9.4.2
x_o	x coordinate of the shear center, negative, in.	3.7.2, 9.5.3
α	Reduction factor for computing effective area of stiffener section	2.3.1.2, 8.4.1.2
β	$1 - (x_o/r_o)^2$	3.6.1.2, 9.4.2
ϕ	Resistance factor for the appropriate limit state	8.3.1, 8.3.5, 9.2, 9.3, 9.4, 9.5, 9.6, 10.2, 10.3
σ_{bC}	Maximum compression bending stress caused by M_c , ksi	3.7.2, 9.5.2
σ_{bT}	Maximum compression bending stress caused by M_T , ksi	3.7.2, 9.5.2

σ_{b1}	$\sigma_{TF} ec/r_y^2 =$ maximum compression bending stress in the section caused by σ_{TF} , ksi	3.7.2, 9.5.2
σ_{b2}	$\sigma_{TF} x_o c/r_y^2$, ksi	3.7.2, 9.5.2
σ_e	$\pi^2 E/(KL_b/r_b)^2$, ksi	3.7.2, 9.5.2
σ_{ex}	$\pi^2 E/(KL/r_x)^2$, ksi	3.6.1.2, 3.7.2, 9.5.2 9.4.2
σ_t	Torsional buckling stress, ksi	3.6.1.2, 3.7.2, 9.5.2 9.4.2
σ_{TF}	Average elastic torsional-flexural buckling stress, ksi	3.7.2, 9.5.2
σ_{TFO}	Elastic torsional-flexural buckling stress, ksi	3.6.1.2, 3.7.2 9.5.2, 9.4.2

LOAD AND RESISTANCE FACTOR DESIGN CRITERIA FOR
COLD-FORMED STEEL STRUCTURAL MEMBERS

SECTION 7 - GENERAL

7.1 Scope

This Specification shall apply to the design of structural members cold-formed to shape from carbon or low-alloy steel sheet, strip, plate or bar not more than one inch in thickness and used for load-carrying purposes in buildings. It may also be used for structures other than buildings provided appropriate allowances are made for thermal and/or dynamic effects.

7.2 Material

7.2.1 General

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

Steel Sheet, Zinc-coated (Galvanized) by the Hot-Dip Process, Structural (Physical) Quality, ASTM A446 -72

Hot-Rolled Carbon Steel Sheet and Strip, Structural Quality, ASTM A570 -72

Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High Strength, Low Alloy, with Improved Corrosion Resistance, ASTM A606 -71

Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High Strength, Low Alloy Columbium and/or Vanadium, ASTM A607 -70

Steel, Cold-Rolled Sheet, Carbon, Structural, ASTM A611 -72

Structural Steel, ASTM A36 -70a

High-Strength Low-Alloy Structural Steel, ASTM A242 -70a

High-Strength Low-Alloy Structural Manganese Vanadium Steel, ASTM A441-70a

High-Strength Low-Alloy Columbium-Vanadium Steels of Structural

Quality, ASTM A572-73

High-Strength Low-Alloy Structural Steel with 50,000 psi Minimum Yield Point to 4 in. Thick, ASTM A588-71

Structural Steel with 42,000 psi Minimum Yield Point ($\frac{1}{2}$ in. Maximum Thickness), ASTM A529-72

7.2.2 Other Steels

The listing in Section 7.2.1 does not exclude the use of steel up to and including one inch in thickness ordered or produced to other than the listed specifications provided such steel conforms to the chemical and mechanical requirements of one of the listed specifications or other published specifications 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.

7.3 Delivered Minimum Thickness

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

SECTION 8 - DESIGN PROCEDURE

8.1 Procedure

All computations for load effects (axial force, bending moment, shear force, stress due to the factored loads) shall be in accordance with conventional methods of structural analysis except as otherwise specified herein. The method of design shall conform to the Load and Resistance Factor Design criteria as defined in Sec. 8.3.

8.2 Definitions

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

(a) Stiffened Compression Elements. A 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 conforming to the requirements of Section 8.4.2.

(b) Unstiffened Compression Elements. An unstiffened compression element is a flat 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 and which conform to the requirements of Section 8.4.2.2. A sub-element is the portion between adjacent stiffeners or between web and intermediate stiffener or between edge and intermediate stiffener.

(d) Flat-Width Ratio. The flat-width ratio, w/t , of single flat element is the ratio of the flat width, w , exclusive of edge fillets, to the thickness, t . In the case of sections such as I-, T-, channel- and Z-shaped sections, the width, w , is the width of the flat projection of flange from web, exclusive of fillets and of any stiffening lip that may be at the outer edge of the flange. In the case of multiple-web sections such as hat-, U- or box-shaped sections, the width, w , is the flat width of flange between adjacent webs, exclusive of fillets.

(e) Effective Design Width. Where the flat width, w , of an element is reduced for design purposes, the reduced design width, b , is termed the effective width or the effective design width, and is determined in accordance with Sections 8.4.1 and 8.4.5.

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

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

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

(i) Yield Point. Yield point, F_y , as used in this Specification shall mean yield point or yield strength.

(j) Stress. Stress as used in this Specification means force per unit area and is expressed in kips per square inch, abbreviated throughout as ksi.

8.3 Load and Resistance Factor Design

Load and Resistance Factor Design is a method of proportioning cold-formed structural steel elements (i.e., members, connectors and connections) such that any applicable limit state is not exceeded when the structure is subjected to any appropriate load combinations.

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

8.3.1 Limit State - Strength

The design is satisfactory when the computed load effects, as determined from the assigned nominal loads which are multiplied by appropriate load factors, are smaller than or equal to the factored nominal strength of each structural element.

The factored nominal strength is equal to ϕR_n , where ϕ is a resistance factor and R_n is the nominal strength determined according to the formula given in Section 9 for members and in Section 10 for connections. Values of resistance factors ϕ are given in Section 8.3.5 for the appropriate limit states governing member and connection strength.

8.3.2 Limit State - Serviceability

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

8.3.3 Nominal Loads

The nominal loads acting on the structure are the loads specified in ANSI A58.1-1980, "Building Code Requirements for Minimum Design Loads in Buildings and Other Structures".

8.3.4 Load Factors and Load Combinations

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

$$1.4 D_n$$

$$1.2 D_n + 1.6 (L_n \text{ or } S_n)$$

$$1.2 D_n + 0.5 (L_n \text{ or } S_n) + 1.3 W_n$$

$$1.2 D_n + 0.5 (L_n \text{ or } S_n) + 1.5 E_n$$

$$1.2 D_n + 0.5 L_n + 1.6 S_n$$

$$1.2 D_n + 1.6 S_n + 0.8 W_n$$

$$0.9 D_n - 1.3 W_n$$

$$0.9 D_n - 1.5 E_n$$

$$1.2 D_n + 1.2 P_n$$

where D_n = nominal dead load

L_n = nominal live load

S_n = nominal snow load

W_n = nominal wind load

E_n = nominal earthquake load

P_n = nominal ponding load

8.3.5 Resistance Factors

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

<u>Type of Strength</u>	<u>Resistance Factor, ϕ</u>
(a) Tension members	0.85
(b) Flexural members	
Section strength	0.90
Laterally unbraced beams	0.80
Web design	
Shear strength*	0.85
Flexural strength	0.75
Web crippling	
Beams having single, unreinforced webs	
End reactions or concentrated loads on outer ends of cantilevers	
One-flange loading	0.80
Two-flange loading	0.65
Reactions of interior supports or concentrated loads located anywhere on the span	
One-flange loading	0.85
Two-flange loading	0.80
I-beams	
End reactions or concentrated loads on outer ends of cantilevers	
One-flange loading	0.78
Two-flange loading	0.42
Reactions of interior supports or concentrated loads located anywhere on the span	
One-flange and two-flange loading	0.55

* When $h/t \leq 374/\sqrt{F_y}$, $\phi = 1.0$

<u>Type of Strength</u>	<u>Resistance Factor, ϕ</u>
(c) Axially loaded compression members	0.80
(d) Beam - columns	
ϕ_c	0.80
ϕ_s	0.90
(e) Cylindrical tubular members	
Flexural strength	0.90
Axial compression	0.80
(f) Welded Connections	
Groove welds	0.85
Fillet welds	0.50
Resistance welds	0.65
(g) Bolted Connections	
Minimum spacing and edge distance	
$F_u/F_y \geq 1.15$	0.70
$F_u/F_y < 1.15$	0.65
Tension strength on net section	
With washers	
Double shear connection	0.60
Single shear connection	0.50
Without washers	0.60
Bearing strength	
See Tables 10.3.4 (A) and 10.3.4 (B)	0.50 - 0.70
Shear strength of bolts	
A307 bolts	0.80
A325 and A449 bolts	0.80
A490 and A354 Grade BD bolts	0.75

8.4 Properties of Sections

Properties of section (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 section where the use of a net section is applicable) except where the use of a reduced cross section, or effective design width, is required by the provisions of Sections 8.4.1 and 8.4.5 of this Specification.

8.4.1 Properties of Stiffened Compression Elements

In computing properties of sections of flexural members and in computing values of Q (Section 9.4.1) for compression members, the flat width, w , of any stiffened compression element having a flat-width ratio larger than $(w/t)_{lim}$ as hereinafter defined shall be considered as being reduced for design purposes to an effective design width, b or b_e , determined in accordance with the provisions of Sections 8.4.1.1 or 8.4.1.2, whichever is applicable, and subject to the limitations of Section 8.4.5 where applicable. That portion of the total width which is considered removed to arrive at the effective design width shall be located symmetrically about the center line of the element.

8.4.1.1 Elements Without Intermediate Stiffeners

The effective design widths of stiffened compression elements which are not subject to the provisions of Section 8.4.1.2 shall be determined from the following formulas:*

*It is to be noted that where the flat-width ratio exceeds $(w/t)_{lim}$ the properties of the section must frequently be determined by successive approximations or other appropriate methods, since the stress and the effective design width are interdependent.

Flanges are fully effective up to

$$(w/t)_{\text{lim}} = 221 / \sqrt{f_{\text{max}}}$$

For flanges with w/t larger than $(w/t)_{\text{lim}}$

$$\frac{b}{t} = \frac{326}{\sqrt{f_{\text{max}}}} \left[1 - \frac{71.3}{(w/t) \sqrt{f_{\text{max}}}} \right] \quad (8.4.1.1-1)$$

Exception: Flanges of closed square and rectangular tubes

are fully effective up to $(w/t)_{\text{lim}} = 237 / \sqrt{f_{\text{max}}}$. For

flanges with w/t larger than $(w/t)_{\text{lim}}$

$$\frac{b}{t} = \frac{326}{\sqrt{f_{\text{max}}}} \left[1 - \frac{64.9}{(w/t) \sqrt{f_{\text{max}}}} \right] \quad (8.4.1.1-2)$$

In the above,

w/t = flat-width ratio

b = effective design width, in.

f_{max} = actual stress in the compression element computed on the basis

of the effective design width. Use factored load for load determination and use nominal load for deflection determination, ksi.

8.4.1.2 Multiple-Stiffened Elements and Wide Stiffened Elements With Edge Stiffeners

Where the flat-width ratio of a sub-element of a multiple-stiffened compression element or of a stiffened compression element which does not have intermediate stiffeners and which has only one longitudinal edge connected to a web does not exceed 60, the effective design width, b , of such sub-element shall be determined in accordance with the provisions of Section 8.4.1.1. Where such flat-width ratio exceeds 60, the effective design width, b_e , of the sub-element or element shall be determined from the following formula:*

$$\frac{b_e}{t} = \frac{b}{t} - 0.10 \left(\frac{w}{t} - 60 \right) \quad (8.4.1.2-1)$$

* See Section 8.4.3 (a) for limitations on the allowable flat-width ratio of a compression element stiffened at one edge by other than a simple lip.

where

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

b = effective design width determined in accordance with the provisions of Section 8.4.1.1, in.

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

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 w/t between 60 and 90:

$$A_{ef} = \alpha A_{st} \quad (8.4.1.2-2)$$

where

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

For w/t greater than 90:

$$A_{ef} = (b_e/w) A_{st} \quad (8.4.1.2-3)$$

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.

* See Section 8.4.2.2 for limitations on number of intermediate stiffeners which may be considered effective and their minimum moment of inertia.

8.4.2 Stiffeners for Compression Elements

8.4.2.1 Edge Stiffeners

In order that a flat compression element may be considered a stiffened compression element it shall be stiffened along each longitudinal edge parallel to the direction of stress by a web, lip, or other stiffening means, having not less than the following minimum moment of inertia:

$$I_{\min} = 1.83t^4 \sqrt{(w/t)^2 - 4,000/F_y} \quad (8.4.2.1-1)$$

but not less than $9.2t^4$

where w/t = flat-width ratio of stiffened element

I_{\min} = minimum allowable moment of inertia of stiffener (of any shape) about its own centroidal axis parallel to the stiffened element, in.⁴

Where the stiffener consists of a simple lip bent at right angles to the stiffened element, the required overall depth, d_{\min} , of such lip may be determined as follows:

$$d_{\min} = 2.8t \sqrt{(w/t)^2 - 4,000/F_y}, \text{ but not less than } 4.8t \quad (8.4.2.1-2)$$

A simple lip shall not be used as an edge stiffener for any element having a flat-width ratio greater than 60.

8.4.2.2 Intermediate Stiffeners

In order that a flat compression element may be considered a multiple stiffened element, it shall be stiffened between webs, or between a web and a stiffened edge, by means of intermediate stiffeners parallel to the direction of stress, and the moment of inertia of each such intermediate stiffener shall be not less than twice the minimum allowable moment of inertia specified for edge stiffeners in Section 8.4.2.1, where w is the width of the sub-element. The following limitations also shall apply:

- (a) If the spacing of stiffeners between two webs is such that the flat-width ratio of the sub-element between stiffeners is larger than $(w/t)_{\lim}$ in Section 8.4.1, only two intermediate stiffeners

(those nearest each web) shall be considered effective.

- (b) If the spacing of stiffeners between a web and an edge stiffener is such that the flat-width ratio of the sub-element between stiffeners is larger than $(w/t)_{lim}$ in Section 8.4.1 only one intermediate stiffener shall be considered effective.
- (c) If intermediate stiffeners are spaced so closely that the flat-width ratio between stiffeners does not exceed $(w/t)_{lim}$ in Section 8.4.1 all the stiffeners may be considered effective. Only for the purposes of computing the flat-width ratio of the entire multiple-stiffened element, such element shall be considered as replaced by an element without intermediate stiffeners whose width w_s is the whole width between webs or from web to edge stiffener, and whose equivalent thickness t_s is determined as follows:

$$t_s = \sqrt[3]{\frac{12I_s}{w_s}} \quad (8.4.2.2)$$

where I_s = moment of inertia of the full area of the multiple-stiffened element, including the intermediate stiffeners, about its own centroidal axis.

8.4.3 Maximum Allowable Flat-Width Ratios

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

- (a) Stiffened compression element having one longitudinal edge connected to a web or flange element, the other stiffened by:
- | | |
|-----------------------------|----|
| Simple lip | 60 |
| Any other kind of stiffener | 90 |
- (b) Stiffened compression element with both longitudinal edges connected to other stiffened elements
- | | |
|--|-----|
| | 500 |
|--|-----|
- (c) Unstiffened compression element
- | | |
|--|----|
| | 60 |
|--|----|

Note: Unstiffened compression elements that have flat-width ratios exceeding approximately 30 and stiffened compression elements that have flat-width ratios exceeding approximately 250 are likely to develop noticeable deformation under the nominal loads, without affecting the strength of the member.

Stiffened elements having flat-width ratios larger than 500 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.

- (d) Unusually Wide Flanges: Where a 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:

$$w_f = \sqrt{\frac{1,800 t d}{f_{av}}} \times \sqrt{\frac{100 c_f}{d}} \quad (8.4.3-1)$$

where

- w_f = the width of flange projecting beyond the web; or half of the distance between webs for box- or U-type beams, in.
- t = flange thickness, in.
- d = depth of beam, in.
- c_f = the amount of curling, in.
- f_{av} = the average stress in the full, unreduced flange width, ksi. (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.) Use nominal load for computing f_{av} .

8.4.4 Maximum Allowable Web Depth

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

following limitations:

- (a) For members with unstiffened webs:

$$(h/t)_{\max} = 150$$

- (b) For members which are provided with adequate means of transmitting concentrated loads and/or reactions into the web:

$$(h/t)_{\max} = 200$$

In the above,

h = clear distance between flanges measured along the plane of web, in.

t = web thickness, in.

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

8.4.5 Unusually Short Spans Supporting Concentrated Loads

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

TABLE 8.4.5

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

In Table 8.4.5 above;

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

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, in.

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.

8.5 Critical Stress for Unstiffened Compression Elements

The critical stress, F_{cr} , in kips per square inch, to be used for computing the section strength of flexural members (Section 9.3.1) and the form factor, Q , for axially loaded compression members (Section 9.4.1) shall be determined as follows:

(a) For $w/t \leq 63.3/\sqrt{\bar{F}_y}$:

$$F_{cr} = \bar{F}_y \quad (8.5-1)$$

(b) For $63.3/\sqrt{\bar{F}_y} < w/t \leq 144/\sqrt{\bar{F}_y}$:

$$F_{cr} = \bar{F}_y (1.28 - 0.0044(w/t)\sqrt{\bar{F}_y}) \quad (8.5-2)$$

(c) For $144/\sqrt{\bar{F}_y} < w/t \leq 25$:

$$F_{cr} = 13,300/(w/t)^2 \quad (8.5-3)$$

(d) For $25 < w/t \leq 60$:

$$F_{cr} = 33.0 - 0.467 (w/t) \quad (8.5-4)$$

except that for angle struts

$$F_{cr} = 13,300/(w/t)^2 \quad (8.5-5)$$

where w/t = flat-width ratio of the unstiffened compression elements

\bar{F}_y = yield point of steel specified in Section 9.1, ksi

SECTION 9 - DESIGN OF MEMBERS

9.1 Yield Point

The stress \bar{F}_y to be used herein is the minimum specified yield point F_y , or the average stress, F_{ya} , when the increase in steel strength resulting from cold forming is utilized according to Section 9.1.1.

9.1.1 Utilization of Cold Work of Forming

Utilization, for design purposes, of any increase in steel strength that results from a cold forming operation is permissible provided that the methods and limitations prescribed in Section 9.1.1.1 are observed and satisfied.

9.1.1.1 Methods and Limitations

Utilization of cold work of forming shall be on the following basis:

- (a) The yield point of axially loaded compression members when Q equals unity, and the flanges of flexural members whose proportions are such that when treated as compression members the quantity Q (Section 9.4.1) is unity, shall be determined on the basis of either (i) full section tensile test (See paragraph (a) of Section 6.3.1); or (ii) stub column test (See paragraph (b) of Section 6.3.1); or (iii) computed as follows:

$$F_{ya} = CF_{yc} + (1-C)F_{yf} \quad (9.1.1.1-1)$$

where F_{ya} = average tensile yield point of the full section of compressive members, or full flange section of flexural members, ksi

C = ratio of total corner area to the total cross-sectional area of the full section of compression members, or full flanges of flexural members

F_{yc} = tensile yield point of corners, ksi, $B_c F_y / (R/t)^m$.

The formula does not apply where F_u/F_y is less than 1.2, R/t exceeds 7, and/or maximum inclined angle exceeds 120°

F_{yf} = weighted average tensile yield point of the flat portions established in accordance with Section 6.3.2 or virgin yield point if tests are not made

$$B_c = 3.69(F_u/F_y) - 0.819(F_u/F_y)^2 - 1.79$$

$$m = 0.192(F_u/F_y) - 0.068$$

R = inside bend radius, in.

F_y = tensile yield point of virgin steel* specified
by Section 7.2 or established in accordance with
Section 6.3.3, ksi

F_u = ultimate tensile strength of virgin steel specified
by Section 7.2 or established in accordance with
Section 6.3.3, ksi

- (b) The yield point of axially loaded compression members with Q less than unity, and the flanges of flexural members whose proportions are such that when treated as compression members the quantity Q (Section 9.4.1) is less than unity, may be taken as (i) the tensile yield point of virgin steel* specified by applicable ASTM Specifications, (ii) the tensile yield point of the virgin steel established in accordance with Section 6.3.3 or (iii) the weighted average tensile yield point of flats established in accordance with Section 6.3.2.
- (c) The yield point of axially loaded tension members shall be determined by either method (i) or method (iii) prescribed in paragraph (a) of this Section.
- (d) Application of the provisions of section 9.1.1.1(a) shall be confined to the following Sections of Specification.
- 9.3 Tension Members
 - 9.4 Flexural Members
 - 9.5 Axially Loaded Compression Members
 - 9.6 Beam - Columns
 - 9.7 Cylindrical Tubular Members in Compression or Bending

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

Application of all provisions of the specification may be based upon the properties of flat steel before forming or on Sections 9.1.1.1 (b) or (c) as applicable.

- (e) The effect on mechanical properties of any welding that is to be applied to the member shall be determined on the basis of test of full section specimens containing within the gauge length, such welding as the manufacturer intends to use. Any necessary allowance for such effect shall be made in the structural design of the member.

9.2 Tension Members

For axially loaded tension members, the factored nominal tensile strength ϕR_{nt} shall be determined according to the following formulas:

$$\phi = 0.85$$

$$R_{nt} = A_n \bar{F}_y \quad (9.2-1)$$

where ϕ = resistance factor for tension

R_{nt} = nominal strength of the member when loaded in tension, kips

A_n = net area of the cross section, in.²

\bar{F}_y = specified minimum yield point of steel, F_y , or the average yield point of the full section, F_{ya} , ksi

9.3 Flexural Members

Flexural members subjected to bending moment and shear force shall be checked for the limit states of a) section strength (Section 9.3.1), b) lateral-torsional buckling in case of unbraced I, channel, or Z-sections (Section 9.3.2), c) web strength (Section 9.3.3), and d) serviceability (Section 9.3.4).

9.3.1 Section Strength

The factored nominal section strength, ϕM_u , shall be determined with $\phi = 0.90$ and

(a) For members with stiffened compression flange

$$M_u = S_{eff} \bar{F}_y \quad (9.3.1-1)$$

where S_{eff} = elastic section modulus of effective section
determined according to Section 8.4, in.³

$\bar{F}_y = F_y$ or F_{ya} , as appropriate (Section 9.1), ksi

(b) For members with unstiffened compression flange

$$M_u = S_c F_{cr} \quad (9.3.1-2)$$

where S_c = elastic section modulus of entire section
about axis of bending; I divided by distance
to extreme compression fiber, in.³

F_{cr} = critical stress determined according to Section
8.5, ksi

9.3.2 Laterally Unbraced Beams*

The factored nominal strength of laterally unbraced I, channel, or Z-shaped members, ϕM_u , shall be determined with $\phi = 0.80$ and

$$(a) \text{ For } M_y/M_e \leq 0.36, \quad M_u = M_y \quad (9.3.2-1)$$

$$(b) \text{ For } 0.36 \leq M_y/M_e \leq 1.8, \quad M_u = M_y \left(\frac{10}{9} \right) \left(1 - \frac{10}{36} (M_y/M_e) \right) \quad (9.3.2-2)$$

$$(c) \text{ For } M_y/M_e \geq 1.8, \quad M_u = M_e \quad (9.3.2-3)$$

where $M_y = S_{xc} \bar{F}_y$

M_e = critical moment determined as follows:

- (i) For bending about the centroidal axis perpendicular to the web for either I-shaped sections symmetrical about an axis in the plane of the web, or symmetric channel-shaped sections.

$$M_e = \frac{\pi^2 E C_b d I_{yc}}{L^2} \quad (9.3.2-4)$$

- (ii) For point-symmetrical Z-shaped sections bent about the centroidal axis perpendicular to the web

$$M_e = \frac{\pi^2 E C_b d I_{yc}}{2L^2} \quad (9.3.2-5)$$

*The provisions of this section apply to I-, Z-, or channel-shaped flexural members (Not including multiple-web deck, U- and closed box type members and curved or arch members). The provisions of this section do not apply to laterally unbraced compression flanges of otherwise laterally stable sections.

In the above,

L = the unbraced length of the member, in.

I_{yc} = the moment of inertia of the compression portion of a section about the gravity axis of the entire section parallel to the web, in.⁴

S_{xc} = compression section modulus of entire section about major axis, I_x divided by the distance to extreme compression fiber, in.³

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

$$C_b = 1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.30\left(\frac{M_1}{M_2}\right)^2$$

but not more than 2.3,

where M_1 is the smaller and M_2 is the larger

bending moment at the ends of the unbraced length,

taken about the strong axis of the member, and

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 the

ratio C_b shall be taken as unity.

For members subjected to combined axial and bending stress (Section 9.5.1a), C_b shall be 1.

E = modulus of elasticity = 29,500 ksi

d = depth of section, in.

\bar{F}_y = yield point of steel specified in Section 9.1, ksi

9.3.3 Web Strength

9.3.3.1 Shear Strength of Beam Webs

For unreinforced flat beam webs, the factored nominal shear strength $\phi_v V_u$ shall be determined as follows:

(a) For $h/t \leq 374/\sqrt{F_y}$

$$\phi_v = 1.0$$

$$V_u = A_w F_y / \sqrt{3} \quad (9.3.3.1-1)$$

(b) For $374/\sqrt{F_y} < h/t \leq 561/\sqrt{F_y}$

$$\phi_v = 0.85$$

$$V_u = 254 A_w \sqrt{F_y} / (h/t) \quad (9.3.3.1-2)$$

(c) For $h/t > 561/\sqrt{F_y}$

$$\phi_v = 0.85$$

$$V_u = 143,000 A_w / (h/t)^2 \quad (9.3.3.1-3)$$

where

ϕ_v = resistance factor for shear

V_u = nominal shear strength of the unreinforced flat beam web, kips

A_w = area of beam web (ht), in.²

F_y = specified minimum yield point of steel, F_y , ksi

h = clear distance between flanges measured along the plane of the web, in.

t = web thickness, in.

Where the web consist of two or more sheets, each sheet shall be considered as a separate member carrying its share of the shear.

9.3.3.2 Flexural Strength of Beams Governed by Webs

The flexural strength of beams shall also be limited by the factored web strength $\phi_b M_{ubw}$ determined from $\phi_b = 0.75$ and

(a) For members with stiffened compression flange

$$M_{ubw} = S_{eff} F_{cr} \quad (9.3.3.2-1)$$

(b) For members with unstiffened compression flange

$$M_{ubw} = S_c F_{cr} \quad (9.3.3.2-2)$$

where ϕ_b = resistance factor for bending

M_{ubw} = nominal maximum bending moment governed by the post-buckling strength of the beam web, kip-in.

$$F_{cr} = 867,000 / (h/t)^2 \leq F_y$$

F_y = specified minimum yield point of steel, ksi

S_c = elastic section modulus of the full section about the axis of bending, in.³

S_{eff} = elastic section modulus of the effective section determined according to section 8.4 by using

$$f_{max} = F_{cr}, \text{ in.}^3$$

9.3.3.3 Combined Bending and Shear in Webs

For beam webs subjected to a combination of bending and shear, the members shall be so proportioned that the factored shear force and the factored bending moment computed on the basis of the factored loads do not exceed the values specified in Sections 9.3.3.1 and 9.3.3.2 and the following requirement be satisfied:

$$\left(\frac{V_D}{\phi_v V_u}\right)^2 + \left(\frac{M_D}{\phi_b M_{ubw}}\right)^2 \leq 1.0 \quad (9.3.3.3-1)$$

where

V_D = factored shear force computed on the basis of the factored loads, kips

M_D = factored bending moment computed on the basis of the factored loads, kip-in.

ϕ_v = resistance factor for shear (See Section 9.3.3.1)

ϕ_b = resistance factor for bending (See Section 9.3.3.2)

V_u = nominal maximum shear strength determined according to Section 9.3.3.1 except that the equation

$$V_u = 254 A_w \sqrt{F_y} / (h/t) \quad \text{shall be used for } h/t \leq 374 / \sqrt{F_y},$$

kips

M_{ubw} = nominal maximum bending moment determined according to Section 9.3.3.2 except that for the computation of F_{cr} the limit of F_y shall not apply, kip-in.

9.3.3.4 Web Crippling of Beams

To avoid crippling of unreinforced beam webs having a flat width ratio, h/t , equal to or less than 150, the concentrated loads and reactions determined according to the factored design loads shall not exceed the values of $\phi_w P_u$ given below. Webs of beams for which the ratio, h/t , is greater than 150 shall be provided with adequate means of transmitting concentrated loads and/or reactions directly into the web.

(a) Beams Having Single, Unreinforced Webs

- (1) For end reactions or for concentrated loads on outer ends of cantilevers:

For inside corner radius equal to or less than the thickness of the sheet

$$\phi_w = 0.80 \text{ for one-flange loading}$$

$$\phi_w = 0.65 \text{ for two-flange loading}$$

$$P_u = \frac{t^2 F_y}{10^3} \left\{ 5450 + 235 \left(\frac{N}{t} \right) - 1.2 \left(\frac{N}{t} \right) \left(\frac{h}{t} \right) - 0.6 \left(\frac{h}{t} \right) \right\} \times \left\{ 1.33 - 0.33 \left(\frac{F}{33} \right) \right\} \quad (9.3.3.4-1)$$

For other corner radii up to $4t$, the value P_u given by the above formula is to be multiplied by $(1.15 - 0.15 \frac{R}{t})$.

- (2) For reactions of interior supports or for concentrated loads located anywhere on the span:

For inside corner radius equal to or less than the thickness of the sheet.

$$\phi_w = 0.85 \text{ for one-flange loading}$$

$$\phi_w = 0.80 \text{ for two-flange loading}$$

$$P_u = \frac{t^2 F_y}{10^3} \left\{ 17,000 + 125 \left(\frac{N}{t} \right) - 0.5 \left(\frac{N}{t} \right) \left(\frac{h}{t} \right) - 30 \left(\frac{h}{t} \right) \right\} \times \left\{ 1.22 - 0.22 \left(\frac{F}{33} \right) \right\} \quad (9.3.3.4-2)$$

For other corner radii up to $4t$, the value P_u given

by the above formula is to be multiplied by $(1.06 - 0.06 \frac{R}{t})$.

- (3) For beam webs subjected to reactions of interior supports or concentrated loads located anywhere on the span with a relatively high bending moment at or immediately adjacent to the reaction or concentrated load, the member shall be proportioned to satisfy the following requirement:

$$\frac{P_D}{\phi_w P_u} + \frac{M_D}{\phi_b M_u} \leq 1.30 \quad (9.3.3.4-3)$$

In lieu of the above given interaction formula, the effect of bending moment on the possible reduction of the web crippling load may be determined either by a documented published method or by tests in accordance with Section 6 of this specification for the particular cross section and the material used for the member.

- (4) For corner radii larger than $4t$, tests shall be made in accordance with Section 12.
- (b) For I-beams made of two channels connected back to back or for 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) For end reactions or for concentrated loads on the outer ends of cantilevers:

$$\begin{aligned} \phi_w &= 0.78 \text{ for one-flange loading} \\ \phi_w &= 0.42 \text{ for two-flange loading} \\ P_u &= t^2 F_y (10 + 1.25\sqrt{N/t}) \end{aligned} \quad (9.3.3.4-4)$$

- (2) For reactions of interior supports or for concentrated loads located anywhere on the span:

$$\begin{aligned} \phi_w &= 0.55 \text{ for one-flange and two-flange loading} \\ P_u &= t^2 F_y (15 + 3.25\sqrt{N/t}) \end{aligned} \quad (9.3.3.4-5)$$

In all of the above, P_u represents the nominal ultimate load or reaction for one solid web sheet connecting top and bottom flanges. For

webs consisting of two or more such sheets, P_u shall be computed for each individual sheet and the results added for the built-up web.

For loads located close to the ends of beams, provisions (a-2) and (b-2) apply, provided for cantilevers the distance from the free end to the nearest edge of bearing, and for a load close to an end support the clear distance from the edge of end bearing to the nearest edge of load bearing is larger than $1.5h$. Otherwise provisions (a-1) and (b-1) apply.

In the above formulas,

ϕ_b = resistance factor for bending

ϕ_w = resistance factor for web crippling

F_y = specified minimum yield point of steel, ksi

h = clear distance between flanges measured along the plane of web, in

M_D = bending moment at or immediately adjacent to P_D computed on the basis of factored loads, kip-in.

M_u = nominal ultimate bending moment if bending moment only exist. The value of M_u shall be M_u (Section 9.3.1) or M_{ubw} (Section 9.3.3.2) whichever is smaller, kip-in.

N = actual length of bearing, except that in the above formula the value of N shall not be taken greater than h , in.

P_D = concentrated load or reaction based on factored loads, kips

P_u = nominal ultimate concentrated load or reaction, kips

R = inside bend radius, in.

t = web thickness, in.

9.3.4 Serviceability

Deflection of beams shall be computer for nominal loads and for the sectional properties defined in Section 8.4.1.1.

9.4 Axially Loaded Compression Members

9.4.1 Shapes Not Subject to Torsional-Flexural Buckling

For doubly-symmetric shapes, closed cross section shapes or cylindrical sections, and any other shapes* which can be shown not to be subject to torsional-flexural buckling, and for members braced against twisting, the factored axial strength, $\phi_c P_u$, shall be determined from $\phi_c = 0.80$ and

$$(a) \text{ For } KL/r \leq C_c/\sqrt{Q}, \quad P_u = A Q \bar{F}_y \left[1 - \frac{Q \bar{F}_y}{4 \pi^2 E} \left(\frac{KL}{r} \right)^2 \right] \quad (9.4.1-1)$$

$$(b) \text{ For } KL/r > C_c/\sqrt{Q}, \quad P_u = \frac{\pi^2 EA}{(KL/r)^2} \quad (9.4.1-2)$$

In the above,

$$C_c = \sqrt{2 \pi^2 E / \bar{F}_y}$$

A = full, unreduced cross-sectional area of the member, in.²

E = modulus of elasticity = 29,500 ksi

K = effective length factor**

L = unbraced length of member, in.

r = radius of gyration of full, unreduced cross section, in.

*See Section 3.6.1 of the Commentary for some sections including Z-sections not subject to torsional-flexural buckling.

**In frames where lateral stability is provided by diagonal bracing, shear walls, attachment to an adjacent structure having adequate lateral stability, or by 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 shall be taken as unity, unless analysis shows that a smaller value may be used. The effective length KL of compression members in a frame which depends upon its own bending stiffness for lateral stability, shall be determined by a rational method and shall not be less than the actual unbraced length.

$$\bar{F}_y = F_y \text{ or } F_{ya}, \text{ as appropriate}$$

Q = factor determined as follows:

- (a) For members composed entirely of stiffened elements

$$Q = Q_a = \frac{A_{\text{eff}}}{A} \quad (9.4.1-3)$$

where A_{eff} is the effective area as determined for the effective design widths from Section 8.4 for $f_{\text{max}} = \bar{F}_y$.

- (b) For members composed entirely of unstiffened elements

$$Q = Q_s = \frac{F_{\text{cr}}}{\bar{F}_y} \quad (9.4.1-4)$$

where F_{cr} is the critical stress for the weakest element of the cross section as determined from the formulas given in Section 8.5.

- (c) For members composed of both stiffened and unstiffened elements

$$Q = Q_a Q_s \quad (9.4.1-5)$$

except that the stress upon which Q_a is to be based shall be that value of the stress F_{cr} which is used in computing Q_s and the effective area to be used in computing Q_a shall include the full area of all unstiffened elements.

9.4.2 Singly-Symmetric and Nonsymmetric Shapes of Open Cross-Section
or Intermittently Fastened Singly-Symmetric Components of Built-
Up Shapes Which May be Subject to Torsional-Flexural Buckling

For singly-symmetric or nonsymmetric shapes* of open cross section
or intermittently fastened singly-symmetric components of built-up shapes
which may be subject to torsional-flexural buckling and which are not
braced against twisting,

$$\phi_c = 0.80$$

P_u is the smaller of the values determined from Section 9.4.1 and the
following formulas:

$$\text{For } \sigma_{TFO} > 0.5Q\bar{F}_y, \quad P_u = A Q \bar{F}_y \left(1 - \frac{Q \bar{F}_y}{4 \sigma_{TFO}} \right) \quad (9.4.2-1)$$

$$\text{For } \sigma_{TFO} \leq 0.5Q\bar{F}_y, \quad P_u = A \sigma_{TFO} \quad (9.4.2-2)$$

where Q is determined as in Section 9.4.1, and

σ_{TFO} = elastic torsional-flexural buckling stress under
concentric loading which shall be determined as
follows:

(a) Singly-Symmetric Shapes

For members whose cross-sections have one axis of symmetry
(x-axis), σ_{TFO} is less than both σ_{ex} and σ_t and is equal to:

$$\sigma_{TFO} = \frac{1}{2\beta} \left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t} \right] \quad (9.4.2-3)$$

where

$$\sigma_{ex} = \frac{\pi^2 E}{(KL/r_x)^2}, \text{ ksi}$$

* See Section 3.6.2 of the Commentary for some sections including Z-sections
not subject to torsional-flexural buckling.

$$\sigma_t = \frac{1}{Ar_o^2} \left[GJ + \frac{\pi^2 EC_w}{(KL)^2} \right], \text{ ksi}$$

$$\beta = 1 - (x_o/r_o)^2$$

A = cross-sectional area, in.²

$r_o = \sqrt{r_x^2 + r_y^2 + x_o^2}$ = polar radius of gyration of cross-section about the shear center, in.

r_x, r_y = radii of gyration of cross-section about centroidal principal axes, in.

E = modulus of elasticity = 29,500 ksi

G = shear modulus = 11,300 ksi

K = effective length factor

L = unbraced length of compression member, in.

x_o = distance from shear center to centroid along the principal x-axis, in.

J = St. Venant torsion constant of the cross section, in.⁴. For thin-walled sections composed of n segments of uniform thickness,

$$J = \frac{1}{3}(l_1 t_1^3 + l_2 t_2^3 + \dots + l_i t_i^3 + \dots + l_n t_n^3)$$

t_i = steel thickness of the member for segment i, in.

l_i = length of middle line of segment i, in.

C_w = warping constant of torsion of the cross-section, in.⁶

(b) Nonsymmetrical Shapes

For shapes whose cross sections do not have any symmetry, either about an axis or about a point, σ_{TFO} shall be determined by rational analysis.

Alternatively, compression members composed of such shapes may be tested in accordance with Section 6.

9.4.3 Maximum Slenderness Ratio

The slenderness ratio, KL/r , of compression members shall not exceed 200, except that during construction only, KL/r shall not exceed 300.

9.5 Beam - Columns

9.5.1 Shapes not Subject to Torsional-Flexural Buckling

The factored design forces P_D , M_{Dx} , and M_{Dy} shall satisfy the following interaction equations:

$$\frac{P_D}{\phi_c P_{uc}} + \frac{C_{mx} M_{Dx}}{(\phi M_{ucx}) \left(1 - \frac{P_D}{\phi_c P_{Ex}}\right)} + \frac{C_{my} M_{Dy}}{(\phi M_{ucy}) \left(1 - \frac{P_D}{\phi_c P_{Ey}}\right)} \leq 1.0 \quad (9.5.1-1)$$

$$\frac{P_D}{\phi_s P_{us}} + \frac{M_{Dx}}{\phi_s M_{usx}} + \frac{M_{Dy}}{\phi_s M_{usy}} \leq 1.0 \quad (9.5.1-2)$$

except that when $\frac{P_D}{\phi_c P_{uc}} \leq 0.15$, the following formula may be used in lieu

of the above two formulas:

$$\frac{P_D}{\phi_c P_{uc}} + \frac{M_{Dx}}{\phi M_{ucx}} + \frac{M_{Dy}}{\phi M_{ucy}} \leq 1.0 \quad (9.5.1-3)$$

where A_{eff} = effective area as determined from Section 8.4, in.²

C_m = a coefficient whose value shall be taken as follows:

1. For compression members in frames subject to joint translation (sidesway), $C_m = 0.85$
2. For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending,

$$C_m = 0.6 - 0.4 \frac{M_1}{M_2}, \text{ but not less than } 0.4$$

where M_1/M_2 is the ratio of the smaller to larger moments at the ends of that portion of the member unbraced in the plane of bending under consideration.

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 traverse 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, $C_m = 1.0$.

E = modulus of elasticity = 29,500 ksi

\bar{F}_y = F_y or F_{ya} , as appropriate (Section 9.1), ksi

I_x = moment of inertia of the section about the x-axis, in.⁴

I_y = moment of inertia of the section about the y-axis, in.⁴

K = effective length factor in the plane of bending

L = unbraced length of member, in.

M_D = factored design moment, kip-in.

ϕM_{uc} = factored nominal beam strength as determined from Sections 9.3.1 and 9.3.2, whichever is smaller, kip-in.

M_{us} = beam strength as determined from Section 9.3.1, kip-in.

P_D = factored design axial load, kips

$$P_{Ex} = \frac{\pi^2 EI_x}{(KL)_x^2}, \text{ kips}$$

$$P_{Ey} = \frac{\pi^2 EI_y}{(KL)^2}, \text{ kips}$$

P_{uc} = axial strength determined by Section 9.4.1, kips

$$P_{us} = A_{eff} \bar{F}_y, \text{ kips}$$

ϕ = 0.90 when the factored nominal beam strength is determined from Section 9.3.1; 0.80 when the factored nominal beam strength is determined from Section 9.3.2

$$\phi_c = 0.80$$

$$\phi_s = 0.90$$

9.5.2 Singly-Symmetric Shapes or Intermittently Fastened Singly-Symmetric Components of Built-Up Shapes Which May be Subject to Torsional-Flexural Buckling.

Singly-symmetric shapes subject to both axial compression and bending applied in the plane of symmetry shall be proportioned to meet the following four requirements as applicable:

$$(i) \quad \frac{P_D}{\phi_c P_{uc}} + \frac{C_m M_D}{\phi M_{us} \left(1 - \frac{P_D}{\phi_c P_{Ey}}\right)} \leq 1.0 \quad (9.5.2-1)$$

$$\text{and} \quad \frac{P_D}{\phi_s P_{us}} + \frac{M_D}{\phi_s M_{us}} \leq 1.0 \quad (9.5.2-2)$$

except that when $\frac{P_D}{\phi_c P_{uc}} \leq 0.15$, the following formula may be used in lieu of the above two formulas:

$$\frac{P_D}{\phi_c P_{uc}} + \frac{M_D}{\phi M_{us}} \leq 1.0 \quad (9.5.2-3)$$

(ii) If the point of application of the eccentric load is located on the side of the centroid opposite from that of the shear center, i.e. if e is

positive, then

$$P_D \leq \phi_c P_u \quad (9.5.2-4)$$

where for $\sigma_{TF} > 0.5Q\bar{F}_y$, $P_u = A Q \bar{F}_y \left(1 - \frac{Q \bar{F}_y}{4 \sigma_{TF}}\right)$ (9.5.2-5)

and for $\sigma_{TF} \leq 0.5Q\bar{F}_y$, $P_u = A \sigma_{TF}$ (9.5.2-6)

where σ_{TF} shall be determined according to the following formula:

$$\frac{\sigma_{TF}}{\sigma_{TFO}} + \frac{C_{TF} \sigma_{b1}}{\sigma_{bT} \left(1 - \frac{\sigma_{TF}}{\sigma_e}\right)} = 1.0 \quad (9.5.2-7)$$

- (iii) Except for T- or unsymmetrical I-sections, if the point of application of the eccentric load is between the shear center and the centroid, i.e., if e is negative, and if P_{uc1} is larger than P_{uc2} , where P_{uc1} is determined from Section 9.4.1 and P_{uc2} is determined from Section 9.4.2,

$$P_D \leq \phi_c P_u$$

where $P_u = P_{uc2} + \frac{e}{x_o} (P_{uE} - P_{uc2})$ (9.5.2-8)

- (iv) For T- and I-sections with negative eccentricities

- (a) If the point of application of the eccentric load is between the shear center and the centroid, and if P_{uc1} is larger than P_{uc2} ,

$$P_D \leq \phi_c P_u$$

where $P_u = P_{uc2} + \frac{e}{x_o} (P_{uac} - P_{uc2})$ (9.5.2-9)

- (b) If the point of application of the eccentric load is located on the side of the shear center opposite from that of the centroid, then

$$P_D \leq \phi_c P_u$$

where for $\sigma_{TF} > 0.5Q\bar{F}_y$,
$$P_u = A Q \bar{F}_y \left(1 - \frac{Q \bar{F}_y}{4 \sigma_{TF}} \right) \quad (9.5.2-10)$$

and for $\sigma_{TF} \leq 0.5Q\bar{F}_y$,
$$P_u = A \sigma_{TF} \quad (9.5.2-11)$$

where σ_{TF} shall be determined according to the formula:

$$\frac{\sigma_{TF}}{\sigma_{ex}} + \frac{C_{TF}}{\sigma_{bC}} \left(\frac{\sigma_{b1}}{1 - \frac{\sigma_{TF}}{\sigma_e}} - \sigma_{b2} \right) = 1.0 \quad (9.5.2-12)$$

In Section 9.5.2, x and y are centroidal axes and the x-axis is the axis of symmetry whose positive direction is pointed away from the shear center.

In the above,

C_{TF} = a coefficient whose value shall be taken as follows:

1. For compression members in frames subject to joint translation (sidesway), $C_{TF} = 0.85$
2. For restrained compression members in frames braced against joint translation and not subject to traverse loading between their supports in the plane of bending

$$C_{TF} = 0.6 - 0.4 \frac{M_1}{M_2}$$

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

c = distance from the centroidal axis to the fiber with maximum compressive stress, negative when

the fiber is on the shear center side of the centroid, in.

d = depth of section, in.

e = eccentricity of the axial load with respect to the centroidal axis, negative when on the shear center side of the centroid, in.

I_{xc} = moment of inertia of the compression portion of a section about its axis of symmetry, in.⁴

j = $\frac{1}{2I_y} \left[\int_A x^3 dA + \int_A xy^2 dA \right] - x_o$, in., where x is the axis of symmetry and y is orthogonal to x

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

M_c = $-A\sigma_{ex} \left[j + \sqrt{j^2 + r_o^2 (\sigma_t / \sigma_{ex})} \right]$, kip-in.

elastic critical moment causing compression on the shear center side of the centroid, kip-in.

M_T = $-A\sigma_{ex} \left[j - \sqrt{j^2 + r_o^2 (\sigma_t / \sigma_{ex})} \right]$, kip-in.

elastic critical moment causing tension on the shear center side of the centroid, kip-in.

P_{uac} = ultimate load determined by both requirements (i) and (iv b) if the point of application of the eccentric load is at the shear center, i.e., the calculated values of P_D in requirement (i) and P_u in requirement (iv b) for $e = x_o$, kips

P_{uE} = ultimate load determined by requirement (i) if the point of application of the eccentric load is at the shear center, i.e., the calculated value of P_D for $e = x_o$; kips

r_b = radius of gyration about the axis of bending, in.

r_{xc} = radius of gyration about the centroidal axis
parallel to the web of that portion of the
I-section which is in compression when there is
no axial load, in.

S_{yc} = compression section modulus of entire section
about axis normal to the axis of symmetry,
 I_y /distance to extreme compression fiber, in.³

x_o = x coordinate of the shear center, negative, in.

$\sigma_{bC} = \frac{M_c}{I_y}$ = maximum compression bending stress caused
by M_c , ksi. For I-sections with unequal flanges
 σ_{bC} may be approximated by $\frac{\pi^2 E d I_{xc}}{L^2 S_{yc}}$

$\sigma_{bT} = \frac{M_T}{I_y}$ = maximum compression bending stress caused
by M_T , ksi. For I-sections with unequal flanges
 σ_{bT} may be approximated by $\frac{\pi^2 E d I_{xc}}{L^2 S_{yc}}$

$\sigma_{b1} = \sigma_{TF} \frac{ec}{r_y} =$ maximum compressive bending stress in the
section caused by σ_{TF} , ksi

$\sigma_{b2} = \sigma_{TF} \frac{x_c}{r_y} =$ ksi

$\sigma_e = \frac{\pi^2 E}{(KL_b/r_b)^2}$, ksi

σ_{TF} = average elastic torsional-flexural buckling stress,
i.e., axial load at which torsional-flexural
buckling occurs divided by the full cross-
sectional area of the member, ksi

$A, r_o, \sigma_{ex}, \sigma_t, \sigma_{TFO}$ are as defined in Section 9.4.

9.5.3 Singly-Symmetric Shapes or Intermittently Fastened Singly-Symmetric Components of Built-Up Shapes Having $Q < 1.0$ Which May be Subject to Torsional-Flexural Buckling

When singly-symmetric shapes or intermittently fastened singly-symmetric components of built-up shapes having $Q < 1.0$ and subject to both axial compression and bending applied in the plane of symmetry, their strength may be determined by tests in accordance with Section 6. Q is defined in Section 9.4.1.

9.5.4 Singly-Symmetric Shapes Which are Unsymmetrically Loaded

Singly-symmetric shapes subject to both axial compression and bending applied out of the plane of symmetry must be designed according to Section 6.2 "Tests for Determining Structural Performance".

9.6 Cylindrical Tubular Members

9.6.1 Flexural Strength

For cylindrical tubular members used as beams, the factored design moment M_D should be less than or equal to ϕM_{us} , where $\phi = 0.90$ and M_{us} is determined below:

$$M_{us} = S_c F_{cr}$$

where F_{cr} is computer as follows:

$$(a) \text{ For } D/t \leq \frac{3300}{\bar{F}_y}$$

$$F_{cr} = \bar{F}_y \quad (9.6.1-1)$$

$$(b) \text{ For } \frac{3300}{\bar{F}_y} < D/t \leq \frac{13000}{\bar{F}_y}$$

$$F_{cr} = \frac{1104}{(D/t)} + 0.666\bar{F}_y \quad (9.6.1-2)$$

in which

D = mean diameter of the cylindrical tube, in.

t = wall thickness of the tube, in.

$$\bar{F}_y = F_y \text{ or } F_{ya} \text{ as appropriate, (Section 9.1), ksi}$$

S_c is defined in Section 9.3.1.

9.6.2 Axial Load in Compression

For cylindrical tubes used as axially loaded compression members, the factored design load P_D shall not exceed ϕP_{uc} nor ϕP_{ur}

where $\phi = 0.80$

P_{uc} is determined according to Section 9.4.1 for $Q = 1.0$,
kips

$$P_{ur} = AF_{cr}, \text{ kips}$$

A = cross-sectional area, in.²

F_{cr} = stress computed according to Section 9.6.1

SECTION 10 - CONNECTIONS

10.1 General

Connections shall be designed to transmit the maximum forces resulting from the factored loads acting on the structure with proper regard for eccentricity. In the case of members subject to reversal of forces, except if caused by factored wind or earthquake loads, the connection shall be proportioned for the sum of the forces.

The allowable stress design provisions of Section 4.3 of this Specification concerning the maximum permissible longitudinal spacing of connectors joining two channels to form an I-section and the requirements of Section 4.4 of this Specification for the spacing of connections in compression elements shall also apply to the connections designed in accordance with the LRFD criteria specified herein.

10.2 Welded Connections

10.2.1 Fusion Welds

10.2.1.1 Complete Penetration Groove Welds

The factored nominal strength ϕR_n of the complete penetration groove welds in tension or compression normal to the effective area or parallel to the axis of the weld shall be determined as follows, provided that the yield strength of the fillet metal is equal to or greater than the yield strength of the base metal.

$$\phi = 0.85$$

$$R_n = L t_w F_y \quad (10.2.1.1-1)$$

where

ϕ = resistance factor for welded connections

R_n = nominal ultimate strength of the complete penetration groove welds, kips

F_y = specified minimum yield point of base metal, ksi

L = length of weld, in.

t_w = effective throat dimension of weld, in.

10.2.1.2 Fillet Welds

The factored nominal shear strength ϕR_n of the fillet welds shall be determined as follows:

$$\phi = 0.50$$

$$R_n = 0.6L t_w F_{Exx} \quad (10.2.1.2-1)$$

where F_{Exx} = specified nominal tensile strength of the weld metal, ksi

ϕ , R_n , L , and t_w are defined in Section 10.2.1.1.

10.2.2 Resistance Welds

The factored nominal shear strength ϕR_n of spot welding shall be determined as follows:

$$\phi = 0.65$$

R_n = tabulated value given in Table 10.2.2, kips.

Table 10.2.2

Nominal Shear Strength of Spot Welding

Thickness of Thinnest Outside Sheet, in.	Shear Strength per Spot kips
0.010	0.125
0.020	0.313
0.030	0.563
0.040	0.875
0.050	1.310
0.060	1.810
0.080	2.690
0.094	3.440
0.109	4.130
0.125	5.000
0.188	10.000
0.250	15.000

10.3 Bolted Connections

10.3.1 Scope

The following LRFD design criteria govern bolted connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is less than 3/16 inch. For bolted connections in which the thickness of the thinnest connected part is equal to or greater than 3/16 inch, refer to the latest edition of the American Institute of Steel Construction's "Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings."

10.3.2 Minimum Spacing and Edge Distance in the Line of Stress

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

$$\begin{aligned} \text{(a) when } F_u/F_y &\geq 1.15 \\ \phi &= 0.70, \quad R_n = teF_u \end{aligned} \quad (10.3.1-1)$$

$$\begin{aligned} \text{(b) when } F_u/F_y &< 1.15 \\ \phi &= 0.65, \quad R_n = teF_u(0.9) \end{aligned} \quad (10.3.1-2)$$

where

ϕ = resistance factor

R_n = nominal resistance per bolt, kips

e = the distance measured in the line of force from the center of a standard hole* to the nearest edge of an adjacent hole or to the end of the connected part, ksi

F_u = nominal tensile strength of the connected part, ksi

F_y = yield point of the connected part, ksi

*The diameter of a standard hole is 1/16 in. larger than the bolt diameter for 1/2 in. and larger bolts, and is 1/32 in. larger than the bolt diameter for bolts less than 1/2 in. in diameter.

t = thickness of the connected part, in.

The ratio of e/d shall not be less than 1.5, where d is the diameter of the bolt.

10.3.3 Tensile Strength on Net Section

The factored nominal tensile strength, ϕR_n , on the net section of the connected part shall be determined as follows:

(a) With washers under both bolt head and nut

$$R_n = (1.0 - 0.9r + 3rd/s) F_u A_n \leq F_u A_n \quad (10.3.3-1)$$

$\phi = 0.60$ for double shear connection

$\phi = 0.50$ for single shear connection

(b) Without washers under both bolt head and nut, or with only one washer

$\phi = 0.60$

$$R_n = (1.0 - r + 2.5rd/s) F_u A_n \leq F_u A_n \quad (10.3.3-2)$$

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

$\phi = 0.85$

$$R_n = F_y A_n \quad (10.3.3-3)$$

where

A_n = net area of the connected part, in²

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 zero.

d = diameter of bolt, in.

s = spacing of bolts perpendicular to line of force, in. In case of a single bolt, s is equal to the width of the sheet.

The symbols ϕ , R_n , F_y , and F_u are defined in Section 10.3.2.

Table 10.3.4 (A)

Bolted Connections With Washers Under Both Bolt Head and Nut

Thickness of Connected Part, in.	Type of Joint	F_u/F_y Ratio of Connected Part	Resistance Factor ϕ	Nominal Resistance R_n
less than 3/16 inch but greater than or equal to 0.024 in.	Inside sheet of double shear connection	≥ 1.15	0.50	$3.5F_u dt$
		< 1.15	0.65	$3.0F_u dt$
	Single shear and outside sheets of double shear connection	No limit	0.55	$3.0F_u dt$

Table 10.3.4 (B)

Bolted Connections Without Washers Under Both Bolt Head and Nut, or With only One Washer

Thickness of Connected Part, in.	Type of Joint	F_u/F_y Ratio of Connected Part	Resistance Factor ϕ	Nominal Resistance R_n
less than 3/16 inch but greater than or equal to 0.036 in.	Inside Sheet of double shear connection	≥ 1.15	0.65	$3.0F_u dt$
	Single shear and outside sheets of double shear connection	≥ 1.15	0.70	$2.2F_u dt$

10.3.4 Bearing Strength in Bolted Connections

The factored nominal bearing strength, ϕR_n , shall be determined by the values of ϕ and R_n given in Table 10.3.4 for the applicable thickness and F_u/F_y ratio of the connected part and the type of joint used in the connection.

In Table 10.3.4, the symbols ϕ , R_n , d , F_u , and t were previously defined. For conditions not shown in Table 10.3.4, the factored nominal bearing strength of bolted connections shall be determined by tests.

10.3.5 Shear Strength of Bolts

The factored nominal shear strength, ϕR_n , of bolts shall be determined as follows:

$$R_n = 0.6m A_{SA} F_u \quad (10.3.5-1)$$

$$\phi = 0.80, \quad \text{for A307 bolts}$$

$$\phi = 0.80, \quad \text{for A325 and A449 bolts}$$

$$\phi = 0.75, \quad \text{for A490 and A354 Grade BD bolts}$$

where m = the number of shear planes per bolt

A_{SA} = stress area when threading is included in shear planes; gross area when threading is excluded from shear planes, in.²

The symbols ϕ , R_n , and F_u were previously defined.

SECTION 11 - BRACING REQUIREMENTS

Structural members and assemblies of cold-formed steel construction designed on the basis of LRFD criteria shall be adequately braced in accordance with good engineering practice and shall comply with the working stress design provisions of Section 5 of this Specification.

SECTION 12 - TESTS FOR SPECIAL CASES

Special tests shall be conducted and evaluated in accordance with Section 6 of this Specification.

PART II

Commentary On
Tentative Recommendations
LOAD AND RESISTANCE FACTOR DESIGN CRITERIA
FOR
COLD-FORMED STEEL STRUCTURAL MEMBERS

COMMENTARY ON TENTATIVE RECOMMENDATIONS
LOAD AND RESISTANCE FACTOR DESIGN CRITERIA
FOR
COLD-FORMED STEEL STRUCTURAL MEMBERS

INTRODUCTION

In the design of steel buildings, the "Allowable Stress Design Criteria" have long been used for the design of cold-formed steel structural members in the United States, Canada, and other countries. Even though the theoretical concept of reliability analysis has been available for some time and the significance of such a concept in structural safety and design is well recognized, the probabilistic method has not yet been explicitly adopted as a basis for the American design standard for steel structures.

Recently, the load and resistance factor design criteria have been developed for steel buildings using hot-rolled shapes and built-up members fabricated from steel plates. It became evident that the development of a new specification for load and resistance factor design of cold-formed steel is highly desirable because the design criteria for heavy hot-rolled steel construction cannot possibly cover the design features of thin-walled, cold-formed steel construction completely.

Since 1976, a joint project has been conducted at Washington University and the University of Missouri-Rolla to develop the new design criteria for cold-formed steel structural members and connections based on the probabilistic approach.

The Load and Resistance Factor Design criteria developed on the

basis of the 1968 Edition of the AISI Specification for allowable stress design and Addendum Nos. 1, 2, and 3 (ref. 1) are included in Sections 7 through 12 of this Specification.

This Commentary contains a brief presentation of the methodology used for the development of the load and resistance factor design criteria. In addition, it provides a record of the reasoning behind, and the justification for, various provisions of the Specification. For detailed background information, reference is made to the research reports given in the bibliography.

SECTION 7 - GENERAL

Section 7 of the load and resistance factor design criteria is the same as Section 1 of the AISI Specification for allowable stress design. Section 1 of the Commentary on the AISI Specification contains discussions on the scope of the Specification, materials and delivered minimum thickness.

SECTION 8 - DESIGN PROCEDURES

8.1 Procedure

Section 8.1 of the LRFD criteria is essentially the same as Section 2.1 of the AISI Specification for allowable stress design, except that in Section 8.1, reference is made to the load and resistance factor design criteria.

8.2 Definitions

The definitions of various terms used for the LRFD criteria are the same as that used for the allowable stress design criteria.

8.3 Load and Resistance Factor Design

The current method of designing cold-formed steel structural members, as presented in the 1968 AISC Specification (Ref. 1), is based on the AISI Allowable Stress Design method. In this approach, the stresses in structural members are computed by accepted methods of structural analysis for the specified loads. These stresses may not exceed the allowable stresses given in the AISI Specification. The allowable stress is determined by dividing a stress at a limit state by a factor of safety. Usual factors of safety inherent in the AISI Specification for the Design of Cold-Formed Steel Structural Members are $5/3$ for beams and $23/12$ for columns.

A limit state is the condition at which the structural usefulness of a load-carrying element is impaired to such an extent that it becomes unsafe for the occupants of the structure, or the element no longer performs its intended function. Typical limit states for cold-formed steel members are excessive deflection, yielding, buckling and attainment of maximum strength after local buckling (i.e., post-buckling strength). These limit states have been established through experience in practice or in the laboratory, and they have been thoroughly investigated through analytical and experimental research. The background for the establishment of the limit states is extensively documented in the Commentary on the AISI Specification (Ref. 2)(see also Ref. 14), and a continuing research effort provides further improvement in understanding them.

The factors of safety are provided to account for the uncertainties and variabilities inherent in the loads, the analysis, the limit state model, the material properties and the geometry. Through experience it has been established that the present factors of safety provide satisfactory design.

The Allowable Stress Design method employs only one factor of safety for a limit state. The use of multiple load factors provides a refinement in the design which can account for the different degrees of the uncertainties and variabilities of the design parameters. Such a design method is called Load and Resistance Factor Design, and its format is expressed by the following design criterion:

$$\phi R_n \geq \sum \gamma_i Q_i \quad (C8.3-1)$$

where R_n = the nominal resistance

ϕ = resistance factor

γ_i = load factors

Q_i = load effects

The nominal resistance is the strength of the element or member for a given limit state, computed for nominal section properties and for minimum specified material properties according to the appropriate analytical model which defines the strength. For a column, for example, $R_n = AF_{cr}$, where A is the cross-sectional area and F_{cr} is the buckling stress. The resistance factor accounts for the uncertainties and variabilities inherent in R_n , and it is usually less than unity. The load effects Q_i are the forces on the cross section (bending moment, axial force, shear force) determined from the specified minimum loads by structural analysis, and the γ_i 's are the corresponding load factors which account for the uncertainties and variabilities of the loads. The load factors are greater than unity.

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

Probabilistic Concepts

Factors of safety or load factors are provided against the uncertainties and variabilities which are inherent in the design process. Structural design consists of comparing nominal load effects Q to nominal resistances R , but both Q and R are random parameters (see Fig. C8.3.1). A limit state is violated if $R < Q$. While the possibility of this event ever occurring is never zero, a successful design should, nevertheless, have only an acceptably small probability of exceeding

the limit state. If the exact probability distributions of Q and R were known, then the probability of $R - Q < 1$ could be exactly determined for any design. In general the distributions of Q and R are not known, and only the means, Q_m and R_m , and the standard deviations, σ_Q and σ_R are available. Nevertheless it is possible to determine relative reliabilities of several designs by the scheme illustrated in Fig. C8.3-2. The distribution curve shown is for $\ln(R/Q)$, and a limit state is exceeded when $\ln(R/Q) \leq 0$ is the probability of violating the limit state. The size of this area is dependent on the distance between the origin and the mean of $\ln(R/Q)$. For given statistical data R_m , Q_m , σ_R and σ_Q , the area under $\ln R/Q \leq 0$ can be varied by changing the value of β (Fig. C8.3-2), since $\beta \sigma_{\ln R/Q} = (\ln R/Q)_m$, from which approximately

$$\beta = \frac{\ln(R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}} \quad (C8.3-2)$$

where $V_R = \sigma_R/R_m$ and $V_Q = \sigma_Q/Q_m$, the coefficients of variation of R and Q , respectively. The index β is called the "reliability index", and it is a relative measure of the safety of the design. When two designs are compared, the one with the larger β is more reliable.

The concept of the reliability index can be used in determining the relative reliability inherent in current design, and it can be used in testing out the reliability of new design formats, as illustrated by the following example of simply supported braced beams with stiffened flanges subjected to dead and live loading.

The design requirement of the 1968 AISI Specification for such a beam is

$$\frac{S_{\text{eff}} F_y}{(FS)} = \frac{\ell^2 s}{8} (D_n + L_n) \quad (C8.3-3)$$

where S_{eff} is the section modulus based on the effective cross section,

$FS = 5/3$, is the factor of safety

F_y is the specified yield point

ℓ is the span and s is the beam spacing

D_n and L_n are, respectively, the code specified dead and live load intensities.

The mean resistance is defined as (Ref. 3)

$$R_m = R_n (P_m M_m F_m) \quad (C8.3-4)$$

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

$$R_n = S_{eff} F_y \quad (C8.3-5)$$

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

P_m = the mean ratio of the experimentally determined ultimate moment to the predicted ultimate moment for the actual material and cross-sectional properties of the test specimens;

M_m = mean ratio of the yield point to the minimum specified value;

F_m = mean ratio of the section modulus to the Handbook (nominal) value.

The coefficient of variation of R equals

$$V_R = \sqrt{V_P^2 + V_M^2 + V_F^2} \quad (C8.3-6)$$

The values of these data were obtained from examining all available tests on beams with stiffened compression flanges, and from analyzing data on yield point values from tests and cross-sectional dimensions from many measurements. This information is developed in Refs. 5 and 6, and it given below:

$P_m = 1.08$, $V_P = 0.10$; $M_m = 1.10$, $V_M = 0.10$; $F_m = 1.0$, $V_F = 0.05$
and thus $R_m = 1.19 R_n$ and $V_R = 0.15$.

The mean load effect is equal to

$$Q_m = \frac{\ell^2 s}{8} (D_m + L_m) \quad (C8.3-7)$$

and

$$V_Q = \frac{\sqrt{(D_m V_D)^2 + (L_m V_L)^2}}{D_m + L_m} \quad (C8.3-8)$$

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

Load statistics have been analyzed in Ref. 4, where it was shown that

$$D_m = 1.05 D_n, \quad V_D = 0.1; \quad L_m = L_n, \quad V_L = 0.25.$$

The mean live load intensity equals the code live load intensity if the tributary area is small enough so that no live load reduction is included.

Substitution of the load statistics into Eqs. C8.3-7 and 8 gives

$$Q_m = \frac{\ell^2 s}{8} \left(\frac{1.05 D_n}{L_n} + 1 \right) L_n \quad (C8.3-9)$$

$$V_Q = \frac{\sqrt{\left(\frac{1.05 D_n}{L_n} \right)^2 V_D^2 + V_L^2}}{\left(\frac{1.05 D_n}{L_n} + 1 \right)} \quad (C8.3-10)$$

Q_m and V_Q thus depend on the dead-to-live load ratio. Cold-formed beams typically have small D_n/L_n , and for our purposes it will be assumed that $D_n/L_n = 0.5$, and so $Q_m = 1.53 L_n (\ell^2 s/8)$ and $V_Q = 0.17$.

From Eq. C8.3-3 we obtain the nominal design capacity

for $D_n/L_n = 0.5$ and $FS = 5/3$. Thus

$$\frac{R_m}{Q_m} = \frac{1.19 \times 2.50 \times L_n (\ell^2 s/8)}{1.53 L_n (\ell^2 s/8)} = 1.94$$

and, from Eq. C8.3-2:

$$\beta = \frac{\ln 1.94}{\sqrt{0.15^2 + 0.17^2}} = 2.93$$

Of itself $\beta = 2.93$ for beams with stiffened compression flanges designed by the 1968 AISI Specification means nothing. However, when this is compared to β for other types of cold-formed members, and to β for designs of various types from hot-rolled steel shapes or even for other materials, then it is possible to say that this particular cold-formed steel beam has about an average reliability.

Basis for LRFD of Cold-Formed Structures

A great deal of work has been performed on determining the values of the reliability index β inherent in traditional design as exemplified by the currently structural design specifications such as the AISC Specification for hot-rolled steel, the AISI Specification for cold-formed steel (Ref. 1), the ACI Code for reinforced concrete members, etc. The studies for hot-rolled steel are summarized in Ref. 3, where also many further papers are referenced which contain additional data. The determination of β for cold-formed steel elements or members is presented in Refs. 5 through 9, where both the basic research data as well as the β 's inherent in the AISI Specification are presented in great detail. The β 's computed in the above referenced publications were developed with slightly different load statistics than those of this Commentary, but the essential conclusions remain the same.

The entire set of data for hot-rolled steel and cold-formed steel design, as well as data for reinforced concrete, aluminum, laminated

timber, and masonry walls was re-analyzed in Ref. 4, by using a) updated load statistics and b) a more advanced level of probability analysis which was able to incorporate probability distributions which describe the true distributions more realistically. The details of this extensive reanalysis are presented in Ref. 4, and so only the final conclusions from the analysis are summarized here:

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

Basic case: Gravity loading, $\beta_o = 3.0$

For connections: $\beta_o = 4.5$

For wind loading: $\beta_o = 2.5$

2) The following load factors and load combinations were developed in Ref. 4 to give essentially the same β 's as the target β_o 's, and are recommended for use with the 1980 ANSI Load Code (Ref. 10) for all materials, including cold-formed steel:

$$1.4D_n$$

$$1.2D_n + 1.6(L_n \text{ or } S_n)$$

$$1.2D_n + 0.5(L_n \text{ or } S_n) + 1.3W_n$$

$$1.2D_n + 0.5(L_n \text{ or } S_n) + 1.5E_n$$

$$1.2D_n + 0.5L_n + 1.6S_n$$

$$1.2D_n + 1.6S_n + 0.8W_n$$

$$0.9D_n - 1.3W_n$$

$$0.9D_n - 1.5E_n$$

$$1.2D_n + 1.2P_n$$

where D_n = nominal dead load

L_n = nominal live load due to occupancy

S_n = nominal snow load

W_n = nominal wind load

E_n = nominal earthquake load

P_n = nominal ponding load, including the increase due to ponded liquid.

Deflection calculations for serviceability criteria are to be made with the appropriate unfactored loads.

The load factors and load combinations given above are recommended for use with the LRFD criteria for cold-formed steel.* The following portions of this Commentary present the background for the resistance factors ϕ which are recommended in Sec. 8.3.5 for the various members in Sections 9 and 10. These ϕ factors are determined in conformance with the load factors given above to provide a target β_o of 3.0 for members and 4.0 for connections, respectively, for the load combination $1.2D_n + 1.6L_n$. This means that

$$\phi R_n = c(1.2D_n + 1.6L_n) = (1.2D_n/L_n + 1.6)cL_n \quad (\text{C8.3-11})$$

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

By assuming $D_n/L_n = 0.5$, Eqs. C8.3-11 and C8.3-9 can be rewritten as follows:

* It is intended that the load factors and load combinations recommended in Ref. 4 will become part of ANSI A58.1-1980. (Ref. 10). These recommendations, as well as the load code itself, are at present (Jan. 1980) under review, thus the load factors may yet be subject to some change.

$$R_n = 2.2(cL_n/\phi) \quad (C8.3-12)$$

$$Q_m = (1.05D_n/L_n + 1)cL_n = 1.525cL_n \quad (C8.3-13)$$

Therefore,

$$\frac{R_m}{Q_m} = \left(\frac{1.443}{\phi}\right)\left(\frac{R_m}{R_n}\right) \quad (C8.3-14)$$

The ϕ factors can be computed from Eq. C8.3-14 and the following equation by using $V_Q = 0.17$:

$$\text{Target } \beta_o = \frac{\ln(R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}} \quad (C8.3-15)$$

In the above calculation, the values of (R_m/Q_m) and V_R can be obtained from Refs. 5 through 9.

The resistance factors ϕ can also be determined for the desired β_o and the resistance statistics R_m/R_n and V_R from charts provided in Ref. 4. For example, for the cold-formed beams with stiffened compression flanges, for which $R_m/R_n = 1.19$ and $V_R = 0.16$, for a D/L ratio of 0.5 and $\beta_o = 3.0$, $\phi = 0.86$ from the charts in Ref. 4.

8.4 Properties of Sections

Section 8.4 of the LRFD criteria is primarily the same as Section 2.3 of the AISI Specification for allowable stress design.

It should be noted that Eqs. 8.4.1.1-1 and 8.4.1.1-2 are to be used for load determination and deflection determination. Because the safety factor is not involved in the derivation of these two design equations, the actual stress in the compression element, f_{\max} , should be based on the factored load for load determination and on the nominal load for deflection determination.

8.5 Critical Stress for Unstiffened Compression Elements

In the design of cold-formed steel flexural members and compression

members, the use of an effective design width is required for stiffened compression elements as specified in Section 8.4.1. For members having unstiffened compression elements, the load and resistance factor design of such members is based on the critical local buckling stress or the yield point of steel, whichever is smaller.

Equations 8.5-1 through 8.5-5 are derived from the allowable stress formulas given in Section 3.2 of the AISI Specification and a safety factor of 1.67. The value of \bar{F}_y is the specified minimum yield point, F_y , or the average stress, F_{ya} , when the increase in steel strength resulting from cold-forming is utilized.

SECTION 9 - DESIGN OF MEMBERS

9.1 Yield Point

This section is the same as Section 3.1 of the 1968 AISI Specification.

The following statistical data (mean values and coefficients of variation) on material and cross-sectional properties were developed in Refs. 5 and 6 for use in the derivation of the resistance factors ϕ :

$$\begin{aligned} (F_y)_m &= 1.10 F_y; & M_m &= 1.10; & V_{F_y} &= V_M = 0.10 \\ (F_{ya})_m &= 1.10 F_{ya}; & M_m &= 1.10; & V_{F_{ya}} &= V_M = 0.11 \\ (F_u)_m &= 1.10 F_u; & M_m &= 1.10; & V_{F_u} &= V_M = 0.08 \\ F_m &= 1.00; & V_F &= 0.05 \end{aligned}$$

The subscript m refers to mean values. The symbol V stands for coefficient of variation. The symbols M and F are, respectively, the ratio of the mean-to-the nominal material property or cross-sectional property; and F_y , F_{ya} , and F_u are, respectively, the specified minimum yield point, the average yield point including the effect of cold forming, and the

specified minimum tensile strength.

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

9.2 Tension Members

Section 9.2 of the LRFD criteria was developed on the basis of Section 3.1 of the AISI Specification for allowable stress design, in which the design of tension members is based only on the yield point of steel.

The resistance factor of $\phi = 0.85$ used for tension member design was derived from the procedure described in Section 8.3 of this Commentary and a selected β_o value of 3.0. In the determination of the resistance factor, the following formulas were used for R_m and R_n :

$$R_m = A_n (F_y)_m \quad (C9.2-1)$$

$$R_n = A_n F_y \quad (C9.2-2)$$

$$\text{i.e. } R_m/R_n = (F_y)_m/F_y \quad (C9.2-3)$$

in which A_n is the net area of the cross section, $(F_y)_m$ is equal to $1.10 F_y$ as discussed in Section 9.1 of the Commentary. By using $V_M = 0.10$, $V_F = 0.05$ and $V_P = 0$, the coefficient of variation V_R is:

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} = 0.11 \quad (C9.2-4)$$

Based on $V_Q = 0.17$, the resistance factor is approximately 0.86 for $\beta = 3.0$.

In Section 9.2, this is rounded off to $\phi = 0.85$.

9.3 Flexural Members

Flexural members are differentiated according to whether or not the member is laterally braced. If such members are laterally supported, then they are proportioned according to the strength of the cross section (Sec. 9.3.1). If they are laterally unbraced, then the limit state is lateral-torsional buckling (Sec. 9.3.2). Cross section strength depends on whether or not the compression flange is composed of stiffened or unstiffened elements.

9.3.1 Section Strength

a) Flexural Members with Stiffened Compression Flange

The strength of beams with a compression flange having stiffened elements is based on the post-buckling strength of the member, and use is made in LRFD of the effective width concept in the same way as in the 1968 AISI Specification (see Ref. 1, where the Commentary provides an extensive treatment of the background research).

The experimental basis for the post-buckling strength of cold-formed beams is examined in Ref. 5, where Table 3 gives the calculation of the predicted strength according to Winter's effective width formulas. A total of 43 tests are examined, and the statistics are summarized as follows:

$$P_m = 1.08, \quad V_p = 0.10$$

The symbol P is the ratio of the experimental strength to the strength predicted by the effective width theory for the material and cross-sectional properties of the test specimens. According to Eqs. C8.3-4 and C8.3-6, the mean and coefficient of variation of the resistance are equal to:

$$R_m = R_n (P_m M_m F_m) = 1.08 \times 1.10 \times 1.0 R_n = 1.19 R_n$$

and

$$V_R = \sqrt{V_P^2 + V_M^2 + V_F^2} = \sqrt{0.10^2 + 0.11^2 + 0.05^2} = 0.16$$

The values of M_m , V_m , F_m and V_F are the values presented in Sec. 9.1 of this Commentary for the material strength (using the data for sections where the increase in yield strength due to cold-forming is utilized). The nominal strength R_n is based on the nominal effective cross section and on the specified minimum yield point, i.e., $R_n = S_{eff} F_y$.

The required value of ϕ , as determined from the charts in Ref. 4 for $\beta_o = 3.0$ and for a dead-to-live load ratio of 0.5 is approximately 0.86. Accordingly, $\phi = 0.9$ was selected for use in LRFD.

b) Flexural Members with Unstiffened Compression Flanges

The basis for the prediction of the strength of beams with unstiffened compression flanges in these LRFD criteria and in the 1968 AISI Specification is the plate buckling theory. The data of the tests are given in Table 3 of Ref. 6, and they are summarized as follows:

for $63.3/\sqrt{F_y} < w/t < 25$; $P_m = 1.24$, $V_p = 0.13$ (for 24 tests)

for $25 < w/t < 60$; $P_m = 1.76$, $V_p = 0.21$ (for 26 tests)

where w/t is the width/thickness ratio of the unstiffened flange element. If all 50 tests are averaged, $P_m = 1.51$ and $V_p = 0.26$. It is evident from these data that the theory underestimates the capacity considerably. This has long been noted, and a generalized effective-width theory, including both stiffened and unstiffened compression flanges, has been proposed (Ref. 11). The same 50 test results with this improved theory give $P_m = 1.04$ and $V_p = 0.14$. Since the intent of this draft of the LRFD criteria is to provide only a translation from the 1968 Allowable Stress Design criteria

into a LRFD format, no change in the basic treatment of the underlying theory will be made. The ϕ -factor is derived as follows:

for $63.3/\sqrt{F_y} < w/t < 25$

$$R_m/R_n = P_m M_m F_m = 1.24 \times 1.10 \times 1.0 = 1.36$$

$$V_R = \sqrt{V_P^2 + V_M^2 + V_F^2} = \sqrt{0.13^2 + 0.10^2 + 0.05^2} = 0.17$$

for $25 < w/t < 60$

$$R_m/R_n = 1.76 \times 1.0 \times 1.0 = 1.76$$

$$V_R = \sqrt{0.21^2 + 0.06^2 + 0.05^2} = 0.22$$

In the latter case the limit state is elastic buckling. $M_m = 1.0$ and $V_m = 0.06$ have been used to account for the basic material variable, the elastic modulus, E .

The ranges of R_m/R_n and V_R in both instances are beyond the charts provided in Ref. 4 and they are outside the range where Eq. C8.3-11 is valid (see Ref 8). The procedure will thus be to select a value of ϕ and then to determine the resulting reliability index using Eq. C8.3-2.

For a dead-to-live load ratio of $D_n/L_n = 0.5$, the load effect data is $Q_m = \left(\frac{1.05D_n}{L_n} + 1 \right) cL_n = 1.53 cL_n$ and $V_Q = 0.17$ (see Sec. C8.3 of this Commentary). According to the LRFD load factors

$$\phi R_n = c (1.2 D_n + 1.6 L_n) = cL_n (1.2 D_n/L_n + 1.6) = 2.2 cL_n$$

or
$$R_n = \frac{2.2 cL_n}{\phi}$$

Thus
$$R_m/Q_m = (R_m/R_n) \left(\frac{1}{\phi} \right) \left(\frac{2.20}{1.525} \right)$$

and
$$\beta = \frac{\ln (R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}} \quad (\text{Eq. C8.3-2})$$

The following table summarizes the results:

R_m/R_n	V_R	ϕ	β
1.36	0.17	0.9	3.2
1.36	0.17	0.95	3.0
1.76	0.22	0.9	3.7
1.76	0.22	0.95	3.5
1.76	0.22	1.0	3.4

The selected value of $\phi = 0.9$ in the LRFD criteria thus furnishes a greater reliability than the target value of $\beta_o = 3.0$.

9.3.2 Laterally Unbraced Beams

There are not many test data on laterally unsupported cold-formed beams. The available test results are summarized in Ref. 8, and they are compared with predictions from elastic buckling theory which states that for a simply supported I- or Channel-shaped beam bent about the major axis by a uniform moment, the buckling moment is equal to:

$$M_{cr} = \frac{\pi^2 E}{L^2} \sqrt{I_y C_w} \sqrt{1 + \frac{GJL^2}{\pi^2 E C_w}} \quad (C9.3.2-1)$$

where L = unbraced length

I_y = minor axis moment of inertia

J = torsion constant

C_w = warping constant

G = shear modulus

E = elastic modulus

The statistical data from Ref. 8 are the following:

$$P_m = 1.15 \quad \text{and} \quad V_p = 0.15$$

$$M_m = 1.0 \quad \text{and} \quad V_M = 0.06$$

$$F_m = 1.0 \quad \text{and} \quad V_F = 0.05$$

and thus

$$R_m/R_n = 1.15 \times 1.0 \times 1.0 = 1.15 \quad \text{and} \quad V_R = \sqrt{0.15^2 + 0.06^2 + 0.05^2} = 0.17$$

The symbol P is the ratio of the test capacity to the lateral-torsional buckling strength predicted by Eq. C9.3.2-1, M is the ratio of the actual to the specified value of the modulus of elasticity, and F is the ratio of the actual to the nominal sectional properties.

Using the values of $R_m/R_n = 1.15$ and $V_R = 0.17$, the resistance factor ϕ as determined using the charts in Ref. 8 gives $\phi = 0.81$. A value of $\phi = 0.80$ is recommended for use in these LRFD criteria. A simplified and conservative form of Eq. C9.3.2-1 is the basis of the design criteria (same as in the Allowable Stress design rules in the AISI Specification of 1968: the second square root in Eq. C8.3-11 is taken to be unity).

9.3.3 Web Strength

For the design of beam webs, consideration should be given to the shear strength, bending strength, combined bending and shear, and web crippling. The design requirements given in Sections 9.3.3.1 through 9.3.3.4 are based on Sections 3.4 and 3.5 of the AISI Specification for allowable stress design.

9.3.3.1 Shear Strength of Beam Webs

The shear strength of beam webs is governed by either yielding or buckling, depending on the h/t ratio and the mechanical properties of steel. For beam webs having small h/t ratios, the shear strength is governed by shear yielding, i.e.:

$$V_u = A_w \tau_y = A_w F_y / \sqrt{3} \quad (\text{C9.3.3.1-1})$$

in which A_w is the area of the beam web computed by $(h \times t)$, and τ_y is

the yield point of steel in shear, which can be computed by $F_y/\sqrt{3}$.

For beam webs having large h/t ratios, the shear strength is governed by elastic shear buckling, i.e.:

$$V_u = A_w \tau_{cr} = \frac{k\pi^2 EA_w}{12(1-\mu^2)(h/t)^2} \quad (C9.3.3.1-2)$$

in which τ_{cr} is the critical shear buckling stress in the elastic range, k is the buckling coefficient, E is the modulus of elasticity, μ is the Poisson's ratio, h is the web depth, and t is the web thickness. By using $k = 5.35$, $E = 29,500$ ksi, and $\mu = 0.3$, the shear strength, V_u , can be determined as follows:

$$V_u = \frac{143000 A_w}{(h/t)^2} \quad (C9.3.3.1-3)$$

For beam webs having moderate h/t ratios, the shear strength is based on the inelastic buckling, i.e.:

$$V_u = \left(\frac{5}{3}\right) \left[\frac{152\sqrt{F_y}}{(h/t)}\right] A_w = \frac{254\sqrt{F_y}}{(h/t)} A_w \quad (C9.3.3.1-4)$$

In the above equation, the maximum shear stress is based on the allowable shear stress specified in Section 3.4.1 of the AISI Specification and a safety factor of 5/3.

In view of the fact that the appropriate test data on shear are not available, the ϕ factors used in Section 9.3.3.1 were derived from the condition that the nominal resistance for the LRFD method is the same as the nominal resistance for the allowable stress design method. Thus,

$$(R_n)_{LRFD} = (R_n)_{ASD} \quad (C9.3.3.1-5)$$

$$\text{Since } (R_n)_{LRFD} \geq c(1.2D_n + 1.6L_n)/\phi \quad (C9.3.3.1-6)$$

$$(R_n)_{ASD} \geq c(\text{F.S.})(D_n + L_n) \quad (C9.3.3.1-7)$$

the resistance factors can be computed from the following formula:

$$\phi = \frac{1.2D_n + 1.6L_n}{(\text{F.S.})(D_n + L_n)}$$

$$= \frac{1.2(D_n/L_n) + 1.6}{(\text{F.S.})(D_n/L_n + 1)} \quad (\text{C9.3.3.1-8})$$

By using a dead-to-live load ratio of $D_n/L_n = 0.5$, the ϕ factors computed from the above equation are listed in Table C9.3.3.1 for three different ranges of h/t ratios. The factors of safety are adopted from the AISI Specification for allowable stress design. It should be noted that the use of a small safety factor of 1.44 for yielding in shear is justified by long standing use and by the minor consequences of incipient yielding in shear compared with those associated with yielding in tension and compression.

Table C9.3.3.1

Range of h/t Ratio	F.S. for Allowable Stress Design	ϕ Factor Computed by Eq.C9.3.3.1-8	Recommended ϕ Factor
$h/t \leq 374/\sqrt{F_y}$	1.44	1.02	1.00
$374/\sqrt{F_y} \leq h/t \leq 561/\sqrt{F_y}$	1.67	0.88	0.85
$h/t > 561/\sqrt{F_y}$	1.71	0.86	0.85

9.3.3.2 Flexural Strength of Beams Governed by Webs

In Section 9.3.1, the factored nominal section strength is based on either yielding or local buckling of the beam flange. In Section 9.3.3.2, the flexural strength of beams is governed by either yielding or buckling of beam webs.

Equations 9.3.3.2-1 and 9.3.3.2-2 are similar to Eqs. 9.3.1-1 and

9.3.1-2, except that the equations for F_{cr} are different. In Section 9.3.3.2, the critical stress equation was derived from the allowable stress equation given in Section 3.4.2 of the AISI Specification and a safety factor of 5/3. Because the AISI allowable design stress is based on a safety factor of 1.23, the critical stress specified in Section 9.3.3.2 recognizes a postbuckling strength factor of $1.67/1.23 = 1.36$.

The bending strength of beams governed by webs was studied in Ref. 9 by comparing the experimental data and the predicted results. Based on a study made on beams having stiffened and unstiffened flanges, the statistical data are as follows (Ref. 9):

(a) Beams having stiffened flanges

$$P_m = 0.98; \quad V_P = 0.11$$

$$M_m = 1.00; \quad V_M = 0.06$$

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

$$R_m/R_n = 0.98; \quad V_R = 0.14$$

(b) Beams having unstiffened flanges

$$P_m = 1.18; \quad V_P = 0.20$$

$$M_m = 1.00; \quad V_M = 0.06$$

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

$$R_m/R_n = 1.18; \quad V_R = 0.22$$

For $\beta_o = 3.0$, the computed ϕ factors are 0.74 and 0.75 for beams having stiffened flanges and beams having unstiffened flanges, respectively. A value of $\phi = 0.75$ is recommended in the design criteria.

9.3.3.3 Combined Bending and Shear in Beams

This section is based on the interaction formula included in Section 3.4.3 of the AISI Specification for allowable stress design.

9.3.3.4 Web Crippling

The nominal ultimate concentrated load or reaction, P_u , is determined by the allowable load given in Section 3.5 of the AISI Specification times the appropriate factor of safety. In this regard, a factor of safety of 1.85 is used for Eqs. 9.3.3.4-1 and 9.3.3.4-2, and a factor of safety of 2.2 is used for Eqs. 9.3.3.4-4 and 9.3.3.4-5.

On the basis of the statistical analysis of the available test data on web crippling, the values of P_m , M_m , F_m , V_p , V_M , and V_F were computed and selected. These values are presented in Table C9.3.3.4. By using $\beta_o = 3.0$ and the computed values of V_R for different conditions, the resistance factors, ϕ , were calculated by using Eq. C8.3-11 as listed in Table C9.3.3.4.

9.4 Axially Loaded Compression Members

The available experimental data on cold-formed steel axially loaded compression members were evaluated in Ref. 7. The test results were compared to the predictions based on the same mathematical models on which the AISI Specification (Ref. 1) was based. The design provisions in these LRFD criteria are also based on the same mathematical models.

Cross-Sectional Strength

Axially loaded columns are designed against overall instability and local instability. This latter effect is included through the use of the Q-factor in the column equations where this is appropriate. For columns the resistance factor ϕ thus includes both types of instability. Beam-columns are designed both against an overall stability limit state and against a member strength limit state (see Sec. 9.5) separately. Therefore it is necessary to derive a value of ϕ for member strength to be used in beam-column design. The basis for the determination of

Table C9.3.3.4
 Computed ϕ -Factors for
 Web Crippling

Case	P_m	M_m	F_m	$\frac{R_m}{R_n}$	V_P	V_M	V_F	V_R	ϕ Value for $\beta_o \approx 3.0$
(a) Single Unreinforced Webs									
- End One-Flange Loading	1.14	1.10	1.00	1.25	0.18	0.10	0.05	0.21	0.80
- Interior One-Flange Loading	1.09	1.10	1.00	1.20	0.13	0.10	0.05	0.17	0.85
- End Two-Flange Loading	0.85	1.10	1.00	0.94	0.13	0.10	0.05	0.17	0.65
- Interior Two-Flange Loading	0.99	1.10	1.00	1.09	0.10	0.10	0.05	0.15	0.80
(b) I-Sections									
- End One-Flange Loading	1.26	1.10	1.00	1.39	0.22	0.10	0.05	0.27	0.78
- Interior One-Flange Loading	0.82	1.10	1.00	0.90	0.20	0.10	0.05	0.23	0.55
- End Two-Flange Loading	0.77	1.10	1.00	0.85	0.30	0.10	0.05	0.32	0.42
- Interior Two-Flange Loading	0.88	1.10	1.00	0.97	0.25	0.10	0.05	0.27	0.55

ϕ for the limit state of member strength is the capacity of a compressed short member. Stub column strength is predicted from the effective-width concept for members with stiffened elements, and the theory of plate buckling is used for the prediction of the capacity of members with unstiffened elements. This latter theory is overly conservative, and a generalized effective-width formula has been developed for use with both stiffened and unstiffened elements (Ref. 11). However, the new recommendations have not yet been incorporated into the AISI Specification as of this date (1980), and so the buckling limit state is retained here for unstiffened elements. It should be noted that the statistical evaluation of the test results in Refs. 5, 6, and 7 also includes the comparisons with the generalized effective-width approach. Thus the necessary information to develop new ϕ -factors when the specification is changed is already developed.

Stiffened Elements

Stub-column strength was analyzed in Ref. 5 by comparing the experimental strength to the prediction from the effective-width (post-buckling strength) theory. A total of 44 tests were reported, and the statistical data are as follows (Table 4 in Ref. 5):

$$\begin{array}{ll} P_m = 1.10; & V_P = 0.10 \\ M_m = 1.10; & V_F = 0.10 \\ F_m = 1.0; & V_F = 0.05 \\ R_m/R_n = 1.21; & V_R = 0.15 \end{array}$$

The reliability index β , as determined from the charts in Ref. 4 for a D_n/L_n of 0.5 is 2.9 for the selected value of $\phi_s = 0.9$.

Unstiffened Elements

The strength of stub-columns with unstiffened elements was

analyzed in Ref. 6 according to the plate buckling theory, and the statistical data from Table 4 (Ref. 6) are as follows:

a) width-thickness ratios < 25

Number of data: 22*

$$P_m = 1.08; \quad V_P = 0.11$$

$$M_m = 1.10^{**}; \quad V_M = 0.10$$

$$F_m = 1.0; \quad V_F = 0.05$$

$$R_m/R_n = 1.19; \quad V_R = 0.16$$

For the recommended $\phi_s = 0.9$ the reliability index is $\beta = 2.8$ when $D_n/L_n = 0.5$ (Ref. 4).

b) width-thickness ratios ≥ 25

Number of data: 32***

$$P_m = 1.69; \quad V_P = 0.18$$

$$M_m^{**} = 1.0; \quad V_M = 0.06$$

$$F_m = 1.0; \quad V_F = 0.05$$

$$R_m/R_n = 1.69; \quad V_R = 0.20$$

For the recommended $\phi_s = 0.9$, the reliability index $\beta = 3.8$ when $D_n/L_n = 0.5$. This is considerably above the target of $\beta_o = 3$ and a value of $\phi_s = 1.0$ could have been justified. However, $\phi_s = 0.9$ is recommended for the sake of consistency.

Column Strength

Column capacity in these LRFD criteria is based on the same prediction

* Last test from Table 4b in Ref. 6 is included in the data for $w/t \geq 25$.

** Limit state is inelastic buckling, and so the statistics of the yield stress are used here.

*** This includes the last test point from Table 4b and all data from Table 4c of Ref. 6, except that the last two tests were omitted. These stub columns had w/t of about 60 and their inclusion would have biased the results unduly.

models as were employed in the formulation of the AISI Specification: elastic buckling theory for the case of slender columns, and the tangent modulus theory for columns of intermediate and short length. Two types of limit states are considered: flexural buckling in the plane perpendicular to the minor principal axis (FB) and torsional-flexural buckling (TFB). In the latter case it is required that the cross section is compact, i.e., $Q = 1.0$, while in the case of FB the cross-sectional strength of noncompact shapes is accommodated through the use of $Q < 1.0$, same as in the 1968 AISI Specification.

The resistance factor $\phi_c = 0.8$ was established from the statistical data given in Ref. 7. The summary of the information is given in Table C9.4-1.

The reliability index β was determined from the charts in Ref. 4 for a D_n/L_n ratio of 0.5. The target of $\beta_o = 3.0$ is not entirely satisfied, and different ϕ -factors could have been used for the different cases. However, $\phi_c = 0.8$ for all cases provides consistency and simplicity

9.5 Beam-Columns

With the exception of one set of beam-column tests (see Ref. 8) for hat shapes for which the limit state was torsional-flexural buckling, there are no tests of cold-formed steel beam-columns. The LRFD design criteria provides the same interaction equations as the 1968 Edition of the AISI Specification (Ref. 1), with $\phi_c = 0.80$ (i.e., as recommended for columns) when the limit state is overall member instability, and $\phi_s = 0.90$ (i.e., as recommended for laterally braced beams) when the limit state is section strength. In the calculation of the factored nominal beam strength, ϕM_u , in Eqs. 9.5.1-1, 9.5.1-3, 9.5.2-1, and 9.5.2-3 of the Specification, the ϕ -factor is taken as 0.90 when the

Table C9.4-1 Column Statistics from Ref. 7

Table No. in Ref. 7	Number of Tests	Limit State	P_m	M_m	F_m	V_P	V_M	V_F	$\frac{R_m}{R_n}$	V_R	β
13	9	Elastic FB	0.97	1.0	1.0	0.04	0.06	0.05	0.97	0.09	2.8
14	10	Inelastic FB, compact	1.09	1.1	1.0	0.05	0.11	0.05	1.20	0.13	3.6
15	18	Inelastic FB, stiffened*	0.97	1.1	1.0	0.16	0.10	0.05	1.07	0.20	2.5
16	31	Inelastic FB, unstiffened	1.53	1.1	1.0	0.24	0.10	0.05	1.68	0.26	3.6
17	12	Inelastic FB, stiffened*	1.10	1.1	1.0	0.08	0.11	0.05	1.21	0.14	3.5
20	8	Elastic TFB	1.11	1.0	1.0	0.11	0.06	0.05	1.11	0.13	3.2
21	30	Inelastic TFB	1.20	1.1	1.0	0.14	0.10	0.05	1.32	0.18	3.5

* These two data sets differ in that the predictions for the tests from Table 17 include the effect of cold forming on the average yield stress.

limit state is section strength (Section 9.3.1). For laterally unbraced members, the ϕ -factor is 0.80 as recommended in Section 9.3.2.

9.6 Cylindrical Tubular Members

Section 9.6 of the LRFD criteria is based on Section 3.8 of the AISI Specification for allowable stress design and the applicable factor of safety. A safety factor of 5/3 is used for determining F_{cr} .

The ϕ factor of 0.90 used in Section 9.6.1 is the same as that used in Section 9.3.1 for bending, while the ϕ factor of 0.80 used in Section 9.6.2 is the same as that used in Section 9.4.1 for axially loaded compression members.

SECTION 10 - CONNECTIONS

Section 10 of the LRFD criteria is based on Section 4 of the AISI Specification for allowable stress design. This section contains only the design provisions for welded connections and bolted connections. The allowable stress design provisions of Sections 4.3 and 4.4 for the spacing of connectors can also be used for load and resistance factor design.

The resistance factors to be used for the welded and bolted connections were derived from $\beta_o = 4.0$ and the statistical data summarized in the subsequent discussions. The use of 4.0 instead of 4.5 for β_o is justified by considering that $D_n/L_n = 0.5$ is conservative for most of the applications of cold-formed steel sections.

10.2 Welded Connections

Section 10.2 contains the design provisions for fusion welds (groove welds and fillet welds) and resistance welds. The design equations for the nominal ultimate strength and the ϕ -factor for the complete penetration

groove welds are the same as that used in the AISC LRFD criteria. (Ref. 12).

For fillet welds, the equation used for the nominal ultimate shear strength is the same as that used in the AISC LRFD criteria. The resistance factor was derived from the statistical data presented in Refs. 6 and 7. Based on the available statistical data, the average values of the shear strength-to-specified tensile strength ratio, $(\tau_u/F_{EXX})_m$, and the corresponding coefficients of variation, V_R , have been computed for four different cases as given in Table 10.2.1. Since the nominal value of the (τ_u/F_{EXX}) ratio is considered to be 0.60, then

$$\frac{R_m}{R_n} = \frac{(\tau_u/F_{EXX})_m}{(\tau_u/F_{EXX})_n} = \frac{(\tau_u/F_{EXX})_m}{0.60}$$

All computed values of R_m/R_n are listed in Table 10.2.1. Also given in this table are the ϕ -factors determined for $\beta = 4.0$. For simplicity, a single ϕ -factor of 0.50 is used in Section 10.2.1.2 for fillet welds.

Table 10.2.1

Statistical Data for Fillet Welds

Source	$(\frac{\tau_u}{F_{EXX}})_m$	$\frac{R_m}{R_n}$	V_R	ϕ value for $\beta_o = 4.0$
1. Ref. 6 (page 29)	0.56	0.93	0.16	0.53
2. Ref. 7 (Table 2)				
a. Longitudinal Welds	0.63	1.05	0.25	0.45
b. Transverse Welds	0.61	1.02	0.21	0.50
c. Puddle Welds	0.81	1.33	0.27	0.54

For resistance welds, the nominal ultimate shear strength is based on the following equation:

$$R_n = (2.5) \times (\text{allowable shear per spot specified in Section 4.2.2 of the AISI Specification for allowable stress design})$$

In the above equation, the safety factor is 2.5.

The ϕ -factor used in Section 10.2.2 for the design of resistance welds was determined on the basis of the following statistical data reported in Ref. 6:

$$\begin{array}{ll} P_m = 1.10; & V_P = 0.05 \\ M_m = 1.10; & V_M = 0.10 \\ F_m = 1.00; & V_F = 0.10 \\ R_m/R_n = 1.11; & V_R = 0.15 \end{array}$$

By using the procedure described in Section 8.3 of this Commentary, the computed ϕ -factor for $\beta_o \simeq 4.0$ is 0.65.

10.3 Bolted Connections

Section 10.3 of the LRFD criteria is based on the newly revised Section 4.5 of the AISI Specification for allowable stress design. It deals only with the design of bolted connections used for connected parts thinner than 3/16 inch in thickness. For the design of bolted connections using materials equal to or greater than 3/16 inch in thickness, the AISC Specification should be used.

The equations used for the nominal resistance, R_n , in Sections 10.3.2, 10.3.3, and 10.3.4 are based on Section 4.5 of the AISI Specification and the applicable factors of safety. All ϕ -factors were computed from the statistical data given in Ref. 7 and $\beta_o = 4.0$. Tables C10.3(a), (b), and (c) give a cross reference on the statistical data presented in Tables C10.3(d) and (e).

In Eq. 10.3.5-1, the shear strength of bolts is assumed to be 60%

of the tensile strength. The ϕ -factors used for the high strength bolts are adopted from Ref. 13.

Table C10.3(a)
 Cross Reference on
 Statistical Data for Bolted Connections

Section No. and Title of the LRFD Criteria	Statistical Data for Computing ϕ -factor
10.3.2 - Minimum Spacing and Edge Distance in Line of Stress	
(a) When $F_u/F_y \geq 1.15$	Cases 1, 2, and 5 in Table C10.3(d)
(b) When $F_u/F_y < 1.15$	Cases 3, 4, and 6 in Table C10.3(d)
10.3.3 - Tensile Strength on Net Sections	
(a) With Washers	
Double Shear Condition	Case 8 in Table C10.3(d)
Single Shear Condition	Case 9 in Table C10.3(d)
(b) Without Washers	Case 11 in Table C10.3(d)
10.3.4 - Bearing Strength in Bolted Connections	Tables C10.3(b) and (c)
10.3.5 - Shear Strength of Bolts	
A307 Bolts	All cases in Table C10.3(e)
A325, A449, A490, and A354 Bolts	Adopted from Ref. 13

Table C10.3(b)
 Cross Reference on Statistical Data
 Bolted Connections With Washers Under Both Bolt Head and Nut

Thickness of Connected Part, in.	Type of Joint	F_u/F_y Ratio of Connected Part	Resistance Factor ϕ	Nominal Resistance R_n	Statistical Data for Computing ϕ -factor
less than 3/16 inch but greater than or equal to 0.024 in.	Inside sheet of double shear connection	≥ 1.15	0.50	$3.5F_u dt$	Case 13 in Table C10.3(d)
		< 1.15	0.65	$3.0F_u dt$	Case 14 in Table C10.3(d)
	Single shear and outside sheets of double shear connection	No limit	0.55	$3.0F_u dt$	Cases 15 & 16 in Table C10.3(d)

Table C10.3(c)
 Cross Reference on Statistical Data
 Bolted Connections Without Washers Under Both Bolt Head
 and Nut, or With Only One Washer

Thickness of Connected Part, in.	Type of Joint	F_u/F_y Ratio of Connected Part	Resistance Factor ϕ	Nominal Resistance R_n	Statistical Data for Computing ϕ -factor
less than 3/16 inch but greater than or equal to 0.0036 in.	Inside sheet of double shear connection	≥ 1.15	0.65	$3.0F_u dt$	Case 17(b) in Table C10.3(d)
	Single shear and outside sheets of double shear connection	≥ 1.15	0.70	$2.2F_u dt$	Case 17(a) in Table C10.3(d)

Table C10.3(d)
Statistical Data for Bolted Connections

Type of Design Criteria	Case No. in Table 6a of Ref. 7	$\frac{R_m}{R_n}$	V_R	ϕ value for $\beta_o = 4.0$
Minimum Spacing and Edge Distance	1	1.24	0.16	0.70
	2	1.30	0.17	0.72
	3	1.03	0.12	0.64
	4	1.14	0.13	0.70
	5	1.17	0.15	0.68
	6	1.39	0.21	0.68
Tension Stress on Net Section	8	1.25	0.22	0.59
	9	1.05	0.23	0.48
	11	1.14	0.17	0.63
Bearing Stress on Bolted Connections	13	1.13	0.24	0.50
	14	1.07	0.12	0.67
	15	1.12	0.22	0.53
	16	1.16	0.16	0.66
	17(a)	1.11	0.11	0.66
	17(b)	1.02	0.11	0.71

Table C10.3(e)
Statistical Data for Shear on A307 Bolts

Case No. in Table 6b of Ref. 7	$\left(\frac{\tau_f}{\sigma_f}\right)_m$	$\left(\frac{\sigma_f}{F_u}\right)_m$	$\frac{R_m^*}{R_n}$	V_R	ϕ value for $\beta_o = 4.0$
12a	0.68	1.28	1.45	0.15	0.85
12b	0.60	1.13	1.13	0.14	0.68
13a	0.75	1.28	1.60	0.14	0.96
13b	0.63	1.18	1.24	0.11	0.80
13c	0.76	1.13	1.43	0.11	0.92

$$* \frac{R_m}{R_n} = \left(\frac{\tau_f}{\sigma_f}\right)_m \left(\frac{\sigma_f}{F_u}\right)_m \frac{1}{0.6}$$

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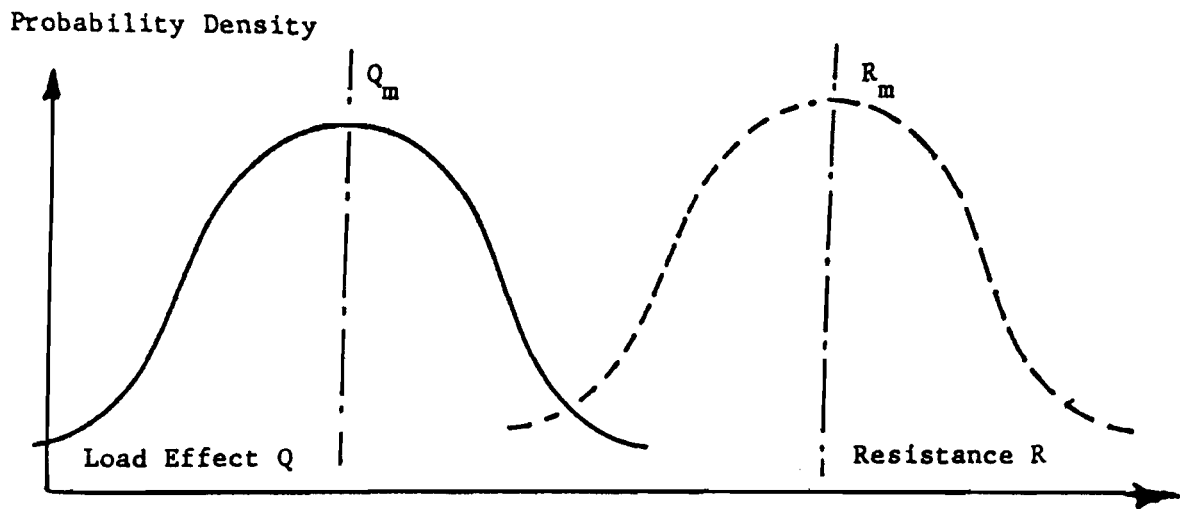


Fig. C8.3-1 Definition of the Randomness of Q and R

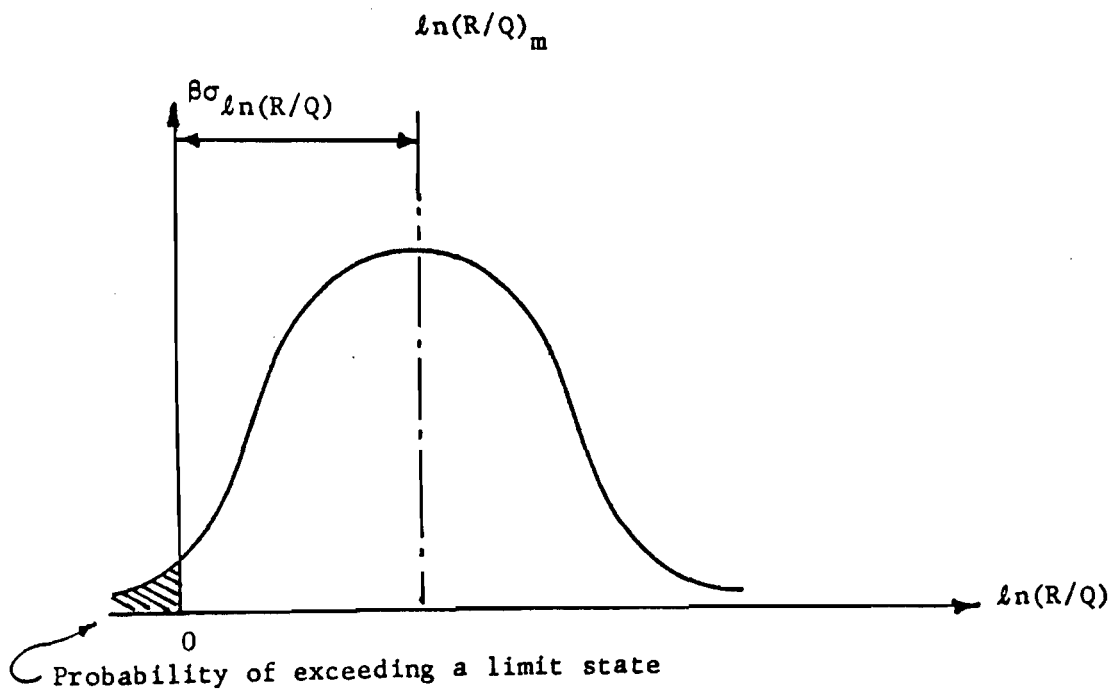


Fig. C8.3-2 Definition of the Reliability Index β