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Load and resistance factor design of cold-formed steel revised tentative recommendations - load and resistance factor design criteria for cold-formed steel structural members with commentary

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LOAD AND RESISTANCE FACTOR DESIGN OF COLD-FORMED STEEL

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

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FOREWORD

This progress report contains the following two parts:

Part I: Revised Tentative Recommendations - Load and Resistance Factor Design Criteria for Cold-Formed Steel Structural Members (pp. i-64).

Part II: Commentary on the Revised Tentative Recommendations - Load and Resistance Factor Design Criteria for Cold-Formed Steel Structural Members (pp. 65-114).

The tentative design recommendations presented herein are the revised version of the design recommendations prepared in March 1980 and submitted to American Iron and Steel Institute as Sixth Progress Report. This document was originally prepared according to the 1980 edition of the AISI Specification for the Design of Cold-Formed Steel Sturctural Members in September 1983 and then finalized in September 1985 following a review of the September 1983 draft by members of Subcommittee 23 of the AISI Advisory Group. 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. Currently, the development of the LRFD recommendations is supervised by Subcommittee 23 of the AISI Asvisory Group on the Specification for the Design of Cold-Formed Steel Structural Members (K. H. Klippstein, Chairman, R. Bjorhovde, D. S. Ellifritt, T. V. Galambos, B. Hall, D. H. Hall, R. B. Heagler, D. L. Johnson, J. Matsen, T. B. Pekoz, C. W. Pinkham, R. M. Schuster, W. W. Yu, S. J. Errera, and A. L. Johnson).

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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.

TABLE OF CONTENTS

NOTATION *

Symbol	Definition	Section
A	Full unreduced cross-sectional area of the member, $in.^2$	3.6.1.1,3.6.1.2, 3.7.2,3.8,9.4.1, 9.4.2,9.5.2,9.6.1
A	Cross-sectional area of the stud, $in.^2$	5.1.1
A_b	$b_1t + A_s$, for transverse stiffeners at interior support under concentrated load, $in.^2$ and $b_2t + A_s$, for transverse stiffeners at end support, $in.^2$	2.3.4
A_c	$18t^2 + A_s$, for transverse stiffeners at interior support and under concentrated load, $in.^2$ and $10t^2 + A_s$, for transverse stiffeners at end support, $in.^2$	2.3.4
A_{ef}	Effective area of the stiffener section, $in.^2$	2.3.1.2,8.4.1.2
A_{eff}	Effective area as determined for the effective design widths, $in.^2$	9.4.1
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_s	Cross-sectional area of transverse stiffeners, $in.^2$	2.3.4.2.1
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_{st}	Full area of stiffener section, $in.^2$	2.3.1.2
A_w	Area of beam web, $in.^2$	9.3.3.1
a	Distance between transverse stiffeners, in.	2.3.4.2.2
a	Shear Panel length of the unreinforced web element, in. For a reinforced web element a is the distance between transverse stiffeners, in.	3.4.1,9.3.3.1
a	Length of bracing interval, in.	5.2.2.2
B	Stud spacing, in.	5.1.1
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.	2.2,2.3.1,8.4.1.1, 8.4.1.2
b_e	Effective design width of sub-element or element, in.	2.3.1.2,8.4.1.2

*This list also includes the symbols used in Sections 2 through 6 of the AISI Specification

NOTATION

Symbol	Definition	Section
b_1	$25t\ [0.0024(L_{st}/t) + 0.72] \leq 25t$, in.	2.3.4
b_2	$12t\ [0.0044(L_{st}/t) + 0.83] \leq 12t$, in.	2.3.4
C	For compression members, ratio of the total corner cross-sectional area to the total cross-sectional area of the full section; for flexural members, ratio of the total corner cross-sectional area of the controlling flange to the full cross-sectional area of the controlling flange	3.1.1.1,9.1.1.1
C_b	Bending coefficient dependent on 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_{mx}	End moment coefficient in interaction formula	3.7.1,9.5.1
C_{my}	End moment coefficient in interaction formula	3.7.1,9.5.1
C_n	Nominal construction load including equipment, workman and formwork but excluding the weight of wet concrete	8.3.4
C_{TF}	End moment coefficient in interaction formula	3.7.2,9.5.2
C_v	$\dfrac{45{,}000k_v}{F_y(h/t)^2}$ when C_v is less than or equal to 0.8 and $\dfrac{190}{h/t}\sqrt{\dfrac{k_v}{F_y}}$ when C_v is more than 0.8	2.3.4.2.2
C_w	Warping constant of torsion of the cross-section, $in.^6$	3.6.1.2,5.1.1, 9.4.2
C_{wn}	Nominal weight of wet concrete during construction	8.3.4
C_y	A term used to compute M_u	3.9,9.7
C_o	Initial column imperfection	5.1.1
C_1	Term used to compute shear strain in wall board	5.1.1
C_1 to C_{11} & C_θ	Terms used to compute allowable reactions and concentrated loads for web crippling	3.5.1.1,9.3.3.4
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

NOTATION

Symbol	Definition	Section
c_f	Amount of curling, in.	2.3.3,8.4.3
D	Mean diameter of cylindrical tube, in.	3.8,9.6
D	Dead load, includes weight of the test specimen	6.2
D	1.0 for stiffeners furnished in pairs; 1.8 for single angle stiffeners; 2.4 for single plate stiffeners	2.3.4.2.2
D_n	Nominal dead load, psf	8.3.4
D_o	Initial column imperfection	5.1.1
d	Depth of section, in.	2.3.3,3.3,3.6.2, 3.7.2,4.3,5.1.1, 8.4.3,9.3.2,9.5.2
d	Width of arc seam weld, in.	4.2.1.2.3,10.2.1.4
d	Visible diameter of outer surface of arc spot weld, in.	4.2.1.2.2,10.2.1.3
d	Diameter of bolt, in.	4.5,10.3.3
d_a	Average diameter of the arc spot weld at mid-thickness of t, in.	4.2.1.2.2,10.2.1.3
d_a	Average width of seam weld, in.	4.2.1.2.3,10.2.1.4
d_e	Effective diameter of fused area, in.	4.2.1.2.2,10.2.1.3
d_e	Effective width of arc seam weld at fused surfaces	4.2.1.2.3,10.2.1.4
d_h	Diameter of standard hole	4.5.4,10.3.2
d_{min}	Overall minimum depth required of simple lip, in.	2.3.2.1,8.4.2.1
d_1	Overall depth of lip, in.	4.3
E	Modulus of elasticity of steel = 29,500 ksi	3.3,3.6.1.2 3.7.2,3.9,5.1.1, 9.3.2,9.4
E_n	Nominal earthquake load	8.3.4
E_o	Initial column imperfection; a measure of the initial twist of the stud from the initial, ideal, unbuckled location	5.1.1
E_1	Term used to compute shear strain in wallboard	5.1.1

NOTATION

Symbol	Definition	Section
E'	$4E\ (QF_y - \sigma)\ /\ (QF_y)^2$, ksi	5.1.1
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
e_{min}	Minimum allowable distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed	4.2.1.2.2
e_{min}	The distance e measured in the line of force from the center of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part toward which the force is directed	4.5.4
e_y	Yield strain = F_y/E	3.9,9.7
F	Basic design stress on the net section of tension members and compression on the extreme fibers of flexural members, ksi	3.1,3.6.1.1, 3.8,3.9
F_a	Maximum average compression stress, ksi	3.7.2
F_{aC}	Average allowable compression stress determined by both requirements (i) and (iv b) of Section 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 Section 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 Section 3.6.1.1 for L = 0, ksi	3.7.2
F_{a1}	Allowable compression stress under concentric loading, ksi	2.3.4.2.1,3.6.1.1, 3.6.1.2,3.7.2,3.8
F_{a2}	Allowable average compression stress under concentric loading, ksi	3.6.1.2,3.7.2
F_{a3}	Allowable compression stress under concentric loading, ksi	5.1.1

NOTATION

Symbol	Definition	Section
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 exits, ksi	3.7.2
F_{bw}	Maximum allowable compression stress in the flat web of a beam due to bending, ksi	3.4.2,3.4.3
F_{bx}	Maximum bending stress in compression that is permitted where bending stress only exists, ksi	5.1.2
F_{b1}	Maximum bending stress in compression permitted by this Specification where bending stress only exists and the possibility of lateral buckling is excluded, ksi	3.7.2
F_c	Maximum allowable compression stress on unstiffened elements, ksi	3.2,3.4.2.2, 3.6.1.1,3.9
F_D	Dead load factor	6.2
F_L	Live load factor	6.2
F_p	Allowable bearing stress, ksi	4.5.6
F_r	Allowable compression stress in cylindrical tubular member, ksi	3.8
F_t	Allowable tension stress on net section, ksi	4.5.5
F_u	Ultimate tensile strength of virgin steel, ksi	3.1.1.1,4.2.1.2, 4.5.4,9.1.1.1
F_u	Nominal tensile strength of the connected part, ksi	10.2.1,10.3
F_v	Maximum allowable average shear stress on the gross area of a flat web, ksi	3.4.1,3.4.3
F_{xx}	Strength level designation in AWS electrode classification, ksi	4.2.1.2,10.2.1
F_y	Yield point, ksi	2.2,2.3,3,4.2.1.2, 4.5.4,5.1.1,8.2, 8.4.2,9.3,9.4,9.5, 9.6,9.7,10.2,10.3

NOTATION

Symbol	Definition	Section
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	8.5,9.1,9.2,9.3, 9.4,9.5,9.6
F_{ya}	Average yield point of section, ksi	3.1,9.1.1
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	3.1.1.1,6.4.2, 9.1.1.1
F_{ys}	Yield point of stiffener steel, ksi	2.3.4.2.1
F'_e	$\dfrac{12\pi^2 E}{23(KL_b/r_b)^2}$, ksi	3.7.1,3.7.2
F'_{ex}	$\dfrac{12\pi^2 E}{23(L/r_x)^2}$, ksi	5.1.2
f	Actual stress in the compression element computed on the basis of the 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	3.7.1,3.7.2, 5.1.2
f_{av}	Average stress in the full, unreduced flange width, ksi	2.3.3,8.4.3
f_{al}	Allowable average compression stress under concentric loading, ksi	3.6.1.1,3.7.2
f_b	Maximum bending stress = bending moment divided by appropriate section modulus of member, ksi	3.7.1,3.7.2, 5.1.2
f_{bw}	Actual compression stress at junction of flange and web, ksi	3.4.3
f_v	Computed maximum average shear stress on the gross area of a flat web, ksi	3.4.1,3.4.3
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	3.6.1.2,5.1.1, 9.4.2
G'	G(E'/E), ksi	5.1.1

NOTATION

Symbol	Definition	Section
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.	2.3.4,3.4,3.5.1, 8.4.4,9.3.3
I_{min}	Minimum allowable moment of inertia of stiffener about its centroidal axis parallel to the stiffener 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_s	Minimum moment of inertia of a pair of attached intermediate stiffeners, or a single intermediate stiffener, with reference to an axis in the plane of the web	2.3.4.2.2
I_x	Moment of inertia of full section about centroidal axis perpendicular to web, $in.^4$	4.3,5.2.2.2
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 of full section about centroidal axes parallel and perpendicular to web, $in.^4$	5.1.1,5.2.2.2
I_y	Moment of inertia of the section about y-axis, $in.^4$	3.7.2,9.5.2
I_{ye}	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
J	St. Venant torsion constant of the cross-section, $in.^4$	3.6.1.2,5.1.1, 9.4.2
j	Section property, torsional-flexural buckling, in.	3.7.2,9.5.2
K	Effective length factor	3.6.1.1,3.6.1.2 3.6.2,3.7.2,9.4,9.5
K'	A constant; for channels = m/d, for Z sections = I_{xy}/I_x	5.2.2.2
k	$F_y/33$	3.5.1,9.3.3.4
k_v	Shear buckling coefficient	3.4.1,9.3.3.1

NOTATION

Symbol	Defintion	Section
L	Full span for simple beams, distance between inflection points for continuous beams, twice the length of cantilever beams, in.	2.3.5,8.4.5
L	Length of seam weld not including the circular ends, in.	4.2.1.2.3, 10.2.1.4
L	Length of fillet weld, in.	4.2.1.2.4, 4.2.1.2.5, 10.2.1.5,10.2.1.6
L	Unbraced length of member, in.	3.3,3.6.1,3.6.2 3.7.2,4.3,9.3.2, 9.4,9.5
L	Length of stud, in.	5.1.1
L	Live load	6.2
L_b	Actual unbraced length in the plane of bending, in.	3.7.2,9.5
L_n	Nominal live load, psf	8.3.4
L_{rn}	Nominal roof live load	8.3.4
L_{st}	Total length of transverse stiffener, in.	2.3.4.2.1
L_i	Length of middle line of segment i, in.	3.6.1.2,5.1.1, 9.4.2
M	Applied bending moment, at or immediately adjacent to the point of application of the concentrated load or reaction P, kip-in.	3.5.2
M_{allow}	Allowable bending moment permitted if bending stress only exists, kip-in	3.5.2
M_c	Elastic critical moment causing compression on the side of the centroid, kip-in.	3.7.2,9.5.2
M_D	Factored bending moment computed on the basis of the factored loads, kip-in.	9.3.3.3,9.5.2
M_D	Bending moment at or immediately adjacent to P_D computed on the basis of factored loads, kip-in.	9.3.3.4
M_T	Elastic critical moment causing tension on the shear 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

NOTATION

Symbol	Definition	Section
M_u	Ultimate moment causing a maximum compression strain of $C_y e_y$, kip-in.	3.9,9.7
M_{ubw}	Nominal maximum bending moment governed by the buckling strength of the beam web, kip-in.	9.3.3
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_y	Moment causing a maximum strain of e_y, kip-in.	3.9,9.7
M_1	Smaller end moment	3.3,3.7.2,9.5.2
M_2	Larger end moment	3.3,3.7.2,9.5.2
m	Term for determining the tensile yield point of corners	3.1.1.1,9.1.1.1
m	t/0.075	3.5.1,9.3.3.4
m	Distance of shear center of channel to mid-plane of the web, in.	4.3,5.2.2.2
m	Number of shear planes per bolt	10.3.5
N	Actual length of bearing, in.	3.5.1,9.3.3.4
P	Concentrated load or reaction in the presence of bending moment, kips	3.5.2
P	Total load on compression member, kips	3.6.1
P	Concentrated load or reaction, kips	4.3,5.2.2.2
P	Force transmitted by bolt, kips	4.5.4
P	Force transmitted by weld, kips	4.2.1.2
P_{allow}	Allowable concentrated load or reaction for one transverse stiffener, kips	2.3.4.2.1
P_{allow}	Allowable concentrated load or reaction, for one solid web sheet connecting top and bottom flanges	3.5.1,3.5.2
P_D	Concentrated load or reaction based on factored load, kips	9.3.3.4

NOTATION

Symbol	Definition	Section
P_{Ex}	$\pi^2 EI_x/(KL)_x^2$, kips	9.5.1
P_{Ey}	$\pi^2 EI_y/(KL)_y^2$, kips	9.5.1
P_L	Force to be resisted by intermediate beam brace, kips	5.2.2.2
P_u	Nominal ultimate concentrated load or reaction, kips	9.3.3.4,9.5
P_{uc}	Axial strength determined by Section 9.4.1, kips	9.5,9.6.2
P_{ur}	AF_{cr}, kips	9.6.2
P_{us}	$A_{eff}\ \bar{F}_y$, kips	9.5
Q	Stress and/or area factor to modify allowable axial stress	2.3.1,3.1.1.1, 3.6.1,3.7.3,3.8, 8.4.1,9.1.1,9.4, 9.5.3,0.6.2
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
$\bar{Q}$	Design shear rigidity for two wallboards, kips	5.1.1
$\bar{Q}_a$	$\bar{Q}/A$, ksi	5.1.1
$\bar{Q}_t$	$\bar{Q}d^2/4Ar_o{}^2$, ksi	5.1.1
q	Intensity of load on beam, kips per lin. in.	4.3
$\bar{q}$	Design shear rigidity for two wallboards per inch of stud spacing, kips per in.	5.1.1
$\bar{q}_o$	Factor used to determine design shear rigidity	5.1.1
R	Inside bend radius, in.	3.1.1.1,3.5, 9.1.1.1,9.3.3.4
R	Minimum load carrying capacity	6.2
R_n	Nominal roof rain load	8.3.4
r	Radius of gyration of full unreduced cross-section, in.	3.6.1.1,3.6.2, 9.4.1,9.4.4

NOTATION

Symbol	Definition	Section
r	Force trnasmitted by the bolt or bolts at the section considered, divided by the tension force in member at that section	4.5.5,10.3.3
r_b	Radius of gyration about axis of bending, in.	3.7.2,9.5.2
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 the shear center, in.	3.6.1.2,3.7.2, 5.1.1,9.4.2
r_x	Radius of gyration of cross-section about centroidal principal axis, in.	3.6.2.1,5.1.1, 9.4.2
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.	3.7.2,5.1.1, 9.5.2
r_y	Radius of gyration of cross-section about centroidal principal axis, in.	3.6.1.2,3.7.2 5.1.1,9.4.2
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
S_{max}	Maximum permissible longitudinal spacing of welds or other connectors joining two channels to form an I-section, in.	4.3
S_c	Elastic section modulus of entire section about axis of bending, I/distance to extreme compression fiber, in.^{3}	9.3.1,9.6.1
S_{eff}	Elastic section modulus of effective section, in.^{3}	9.3.1,9.3.3.2
S_n	Nominal snow load	8.3.4
S_{xc}	Compression section modulus of entire section about major axis, I_x divided by distance to extreme compression fiber, in.^{3}	3.3,9.3.2
S_{yc}	Compression section modulus of entire section about axis normal to axis of symmetry, I_y divided by distance to extreme compression fiber, in.^{3}	3.7.2,9.5.2
s	Fastener spacing, in.	5.1.1
s	Spacing in line of stress, of welds, rivets, or bolts connecting a compression coverplate or sheet to a non-integral stiffener or other element	4.4

NOTATION

Symbol	Definition	Section
s	Weld spacing	4.3
s	Spacing of bolts perpendicular to line of stress, in.	4.5.5,10.3.3
T_s	Strength of connection in tension, kips	4.3
t	Base steel thickness of any element or section, in.	1.3,2.2,2.3, 3.1.1.1,3.2,3.4 3.5,3.8,3.9,8.2, 8.4,8.5,9.1,9.3, 9.4,9.6
t	Total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer, in.	4.2.1.2.2,10.2.1.3
t_e	Effective throat dimension for groove weld, in.	10.2.1.2
t_i	Steel thickness of the member for segment, i, in.	3.6.1.2,5.1.1,9.4.2
t_s	Equivalent thickness of a multiple-stiffened element, in.	2.3.2.2,8.4.2.2
t_w	Effective throat of weld	4.2.1.2.4,4.2.1.2.5, 10.2.1.5,10.2.1.6
V_D	Factored shear force, kips	9.3.3.3
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.	2.2,2.3,3.2,3.9, 8.2,8.4,8.5,9.7
W_n	Nominal wind load	8.3.4
w	Flat width of beam flange which contacts the bearing in.	3.5.2,9.3.3.4.2
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_f	Projection of flanges from inside face of web, in.	4.3
w_s	Whole width between webs or web to edge stiffener, in.	2.3.2.2,8.4.2.2
w_1	Leg on weld	4.2.1.2.4,10.2.1.5
w_2	Leg on weld	4.2.1.2.4,10.2.1.5

NOTATION

Symbol	Definition	Section
x	Distance from concentrated load to brace, in.	5.2.2.2
x_o	Distance from shear center to centroid along the principal x-axis, in.	3.6.1.2,3.7.2,5.1.1, 9.4.2,9.5.2
Y	Yield point of web steel divided by yield point of stiffener steel	2.3.4.2.2
α	Reduction factor for computing effective area of stiffener section	2.3.1.2,8.4.1.2
β	$1 - (x_o/r_o)^2$	3.6.1.2.1,5.1.1, 9.4.2
γ	Shear strain in the wallboard corresponding to σ	5.1.1
$\overline{\gamma}$	Permissible shear strain of the wallboard	5.1.1
θ	Angle between web and bearing surface $\geq 45°$ but no more than 90°	3.5.1,9.3.3.4
σ	Stress related to shear strain in wallboard, ksi	5.1.1
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_{CR}	Theoretical elastic buckling stress under concentric loading, ksi	5.1.1
σ_c	$\pi^2E/(KL_b/r_b)^2$, ksi	3.7.2,9.5.2
σ_{ex}	$\pi^2E/(KL/r_x)^2$, ksi	3.6.1.2,3.7.2,9.4.2, 9.5.2
σ_{ex}	$\pi^2E/(L/r_x)^2$, ksi	5.1.1
σ_{ey}	$\pi^2E/(L/r_y)^2$, ksi	5.1.1
σ_{exy}	π^2EI_{xy}/AL^2, ksi	5.1.1
σ_{TF}	Average elastic torsional-flexural buckling stress, ksi	3.7.2,9.5.2

NOTATION

Symbol	Definition	Section
σ_{TFO}	Elastic torsional-flexural buckling stress under concentric loading, ksi	3.6.1.2, 9.4.2
σ_t	Torsional buckling stress, ksi	3.6.1.2,3.7.2, 9.4.2,9.4.3,9.5.2
σ_{tQ}	$\sigma_t + \bar{Q}_t$, ksi	5.1.1
λ	Stress reduction factor	9.3.3.2

Tentative Recommendations

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, Coils and Cut Lengths, ASTM A446-76 (Grades A,B,C,D, & F)

Hot-Rolled Carbon Steel Sheet and Strip, Structural Quality, ASTM A570-79

Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High Strength, Low Alloy Columbium and/or Vanadium, ASTM A607-75$^{\varepsilon}$

Steel, Cold-Rolled Sheet, Carbon, Structural, ASTM A611-72(1979) (Grades A,B,C & D)

Sheet Steel and Strip, Hot-Rolled, High-Strength, Low-Alloy, With Improved Formability, ASTM A715-75(Grades 50 and 60)

Structural Steel, ASTM A36-77a$^{\varepsilon}$

High-Strength Low-Alloy Structural Steel, ASTM A242-79

High-Strength Low-Alloy Structural Manganese Vanadium Steel, ASTM A441-79

High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality, ASTM A572-79

High-Strength Low-Alloy Structural Steel with 50,000 psi Minimum Yield Point to 4 in. Thick, ASTM A588-80

Structural Steel with 42,000 psi Minimum Yield Point (½ in. Maximum Thickness), ASTM A529-75

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 and Section 7.2.3.

7.2.3 Ductility

Steels not listed in Section 7.2.1 and used for structural members and connections shall comply with one of the following ductility requirements:

7.2.3.1 The ratio of tensile strength to yield point shall not be less than 1.08, and the total elongation shall not be less than 10 percent for a two-inch gage length or 7 percent for an 8 inch gage length standard specimen tested in accordance with ASTM A370-77$^{\varepsilon}$. Sections 8 through 11 of this Specification shall be limited to steels conforming to these requirements.

7.2.3.2 Steels conforming to ASTM A446-76 (Grade E) and A611-72 (1979) (Grade E) and other steels which do not meet provisions of Section 7.2.3.1 may be used for particular configurations provided that the design shall comply with the working stress design provisions of Section 1.2.3.2 of this Specification.

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 Section 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 kips per square inch, abbreviated throughout as ksi.

(k) Confirmatory Test. A confirmatory test is a test made, when desired, on members, connections, and assemblies designed according to the provisions of Sections 7 through 11 of this Specification or its specific references, in order to compare actual versus calculated performance.

(l) Performance Test. A performance test is a test made on structural members, connections, and assemblies whose performance cannot be determined by the provisions of Sections 7 through 11 of this Specification or its specific references.

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

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

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 formulas 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 or equal to 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 A58.1-1982, "American National Standard: Minimum Design Loads for 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:

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* For roof and floor construction, the load combination for dead load, weight of wet concrete, and construction load including equipment, workmen and formwork is suggested in Section 8.3.(2)(a) of the Commentary.

1. $1.4\ D_n$

2. $1.4\ D_n + L_n$

3. $1.2\ D_n + 1.6\ L_n + 0.5(L_{rn}$ or S_n or $R_n)$

4. $1.2\ D_n + 1.6(L_{rn}$ or S_n or $R_n) + (0.5\ L_n$ or $0.8\ W_n)$

5. $1.2\ D_n + 1.3\ W_n + 0.5\ L_n + 0.5(L_{rn}$ or S_n or $R_n)$

6. $1.2\ D_n + 1.5\ E_n + (0.5\ L_n$ or $0.2\ S_n)$

7. $0.9\ D_n - (1.3\ W_n$ or $1.5\ E_n)$

where D_n = nominal dead load

E_n = nominal earthquake load

L_n = nominal live load

L_{rn} = nominal roof live load

R_n = nominal roof rain load

S_n = nominal snow load

W_n = nominal wind load (Exception: For wind load on individual purlins, girts, wall panels and roof decks, multiply W_n by 0.9)

Exception: The load factor on L_n in combinations (4), (5), and (6) shall equal to 1.0 for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 psf.

When the structure effects of F,H,P, or T are significant, they shall be considered in design as the following factored loads: 1.3F, 1.6H, 1.2P, and 1.2T, where

F = loads due to fluids with well-defined pressures and maximum heights

H = loads due to the weight and lateral pressure of soil and water in soil

P = loads, forces, and effects due to ponding

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

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:

Type of Strength	Resistance Factor, ϕ
(a) Tension members	0.95
(b) Flexural members	
Bending strength	0.95
Laterally unbraced beams	0.90
Web design	
Shear strength*	0.90
Bending strength	0.90
Web crippling	0.85
(c) Axially loaded compression members	0.85
(d) Beam - columns	
ϕ_c	0.85
ϕ_s	0.95
ϕ	
For using Section 9.3.1 to compute M_{uc}	0.95
For using Section 9.3.2 to compute M_{uc}	0.90
(e) Cylindrical tubular members	
Bending strength	0.95
Axial compression	0.85

*When $h/t \leq 171\sqrt{k_v/F_y}$, $\phi = 1.0$

(f) Welded Connections

Type of Strength	Resistance Factor, ϕ
Groove welds	
Tension or compression	0.90
Shear (welds)	0.80
Shear (base metal)	0.90
Arc spot welds	
Welds	0.70
Connected part	0.50-0.60
Arc seam welds	
Welds	0.70
Connected part	0.60
Fillet welds	
Longitudinal loading (connected part)	0.60
Transverse loading (connected part)	0.60
Welds	0.70
Flare groove welds	
Transverse loading (connected part)	0.55
Longitudinal loading (connected part)	0.55
Welds	0.70
Resistance welds	0.65

(g) Bolted Connections

Minimum spacing and edge distance	0.70
Tension strength on net section	
With washers	
Double shear connection	0.65
Single shear connection	0.60

Without washers. 0.65

Bearing strength

See Tables 10.3.4 (A) and 10.3.4 (B) 0.60-0.70

Type of Strength	Resistance Factor, ϕ
Shear strength of bolts	
A307 bolts.	0.65
A325 and A449 bolts	0.65
A490 and A354 Grade BD bolts.	0.65

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:*

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*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

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

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

$$\frac{b}{t} = \frac{326}{\sqrt{f_{max}}}\left[1 - \frac{71.3}{(w/t)\sqrt{f_{max}}}\right] \qquad \text{(Eq. 8.4.1-1)}$$

Exception: Flanges of closed square and rectangular tubes are fully effective up to $(w/t)_{lim} = 237 / \sqrt{f_{max}}$. For flanges with w/t larger than $(w/t)_{lim}$

$$\frac{b}{t} = \frac{326}{\sqrt{f_{max}}}\left[1 - \frac{64.9}{(w/t)\sqrt{f_{max}}}\right] \qquad \text{(Eq. 8.4.1-2)}$$

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:*

$$\frac{b_e}{t} = \frac{b}{t} - 0.10\left(\frac{w}{t} - 60\right) \qquad \text{(Eq. 8.4.1-3)}$$

* 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 $60 < w/t < 90$:

$$A_{ef} = \alpha A_{st} \qquad \text{(Eq. 8.4.1-4)}$$

where

$$\alpha = (3 - 2b_e/w) - \frac{1}{30}\left(1 - \frac{b_e}{w}\right)\left(\frac{w}{t}\right) \qquad \text{(Eq. 8.4.1-5)}$$

For $w/t > 90$:

$$A_{ef} = (b_e/w)\, A_{st} \qquad \text{(Eq. 8.4.1-6)}$$

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

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

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* 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

A flat compression element may be considered a stiffened compression element if it is 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:

$$I_{min} = 1.83t^4 \sqrt{(w/t)^2 - 4{,}000/F_y} \quad \text{(Eq. 8.4.2-1)}$$

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:

$$d_{min} = 2.8t \sqrt[6]{(w/t)^2 - 4{,}000/F_y}, \text{ but not less than } 4.8t \quad \text{(Eq. 8.4.2-2)}$$

A simple lip shall not be used as an edge stiffener for any element having a flat-width ratio larger 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:

$$t_s = \sqrt[3]{\frac{12I_s}{w_s}} \qquad \text{(Eq. 8.4.2.2)}$$

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 statisfying Section 8.4.2.1	90

(b) Stiffened compression element with <u>both</u> 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:

$$w_f = \sqrt{\frac{1{,}800\ td}{f_{av}}} \times \sqrt[4]{\frac{100\ c_f}{d}} \qquad \text{(Eq. 8.4.3-1)}$$

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} = 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 and Web Stiffener Requirements

8.4.4.1 Maximum h/t Ratio

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

(a) For unreinforced webs:

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

(b) For webs which are provided with transverse stiffeners satisfying the requirements of Section 8.4.4.2:

(1) When using bearing stiffeners only,

$(h/t)_{max} = 260$

(2) When using bearing stiffeners and intermediate stiffeners,

$(h/t)_{max} = 300$

In the above,

h = 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.

8.4.4.2 Transverse Stiffeners for Beam Webs

Bearing and intermediate stiffeners used for flexural members which are designed on the basis of the LRFD criteria shall comply with the allowable stress design requirements of Section 2.3.4.2 of this Specification.

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 bending 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 $w/t \leq 63.3/\sqrt{F_y}$:

$$F_{cr} = F_y \qquad \text{(Eq. 8.5-1)}$$

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

$$F_{cr} = F_y \left[1.28 - 0.0044(w/t)\sqrt{F_y}\right] \qquad \text{(Eq. 8.5-2)}$$

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

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

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

$$F_{cr} = 33.0 - 0.467\ (w/t) \qquad \text{(Eq. 8.5-4)}$$

except that for angle struts

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

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

F_y = yield point of steel specified in Section 9.1, ksi

SECTION 9 - DESIGN OF MEMBERS

9.1 Yield Point

The stress F_y to be used herein is the minimum specified yield point 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 the strength increase of steel from the 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 propertions 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 6.4.1); (ii) stub column test (See paragraph (b) of Section 6.4.1); or (iii) computed as follows:

$$F_{ya} = CF_{yc} + (1-C)F_{yf} \qquad \text{(Eq. 9.1.1-1)}$$

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

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

F_{yc} = $B_c F_y/(R/t)^m$, tensile yield point of corners,ksi. 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° (Eq. 9.1.1-2)

F_{yf} = weighted average tensile yield point of the flat portions established in accordance with Section 6.4.2 or virgin yield point if tests are not made

$$B_c = 3.69(F_u/F_y) - 0.819(F_u/F_y)^2 - 1.79 \qquad \text{(Eq. 9.1.1-3)}$$

$$m = 0.192(F_u/F_y) - 0.068 \qquad \text{(Eq. 9.1.1-4)}$$

R = inside bend radius, in.

F_y = tensile yield point of virgin steel* specified by Section 7.2 or established in accordance with Section 6.4.3, ksi

F_u = ultimate tensile strength of virgin steel specified by Section 7.2 or established in accordance with Section 6.4.3, 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 quanity 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 6.4.3 or (iii) the weighted average tensile yield point of flats established in accordance with Section 6.4.2.

(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

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

9.5 Axially Loaded Compression Members

9.6 Beam - Columns

9.7 Cylindrical Tubular Members in Compression or Bending

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 withing the guage 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:

$$\phi = 0.95$$

$$R_{nt} = A_n F_y \qquad \text{(Eq. 9.2-1)}$$

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}

F_y = specified minimum yield point of steel 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) bending 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 Bending Strength

The factored nominal bending strength, ϕM_u, shall be determined with $\phi = 0.95$ and

(a) For members with stiffened compression flange

$$M_u = S_{eff} F_y \qquad \text{(Eq. 9.3.1-1)}$$

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

F_y = specified minimum yield point or F_{ya}, as appropriate (Section 9.1), ksi

(b) For members with unstiffened compression flange

$$M_u = S_c F_{cr} \text{ or } M_u = S_t F_y \text{ whichever is smaller} \qquad \text{(Eq. 9.3.1-2)}$$

where S_c = compression 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

S_t = tension elastic section modulus of entire section about axis of bending; I divided by distance to extreme tension fiber, in.^{3}

9.3.2 Laterally Unbraced Beams*

The factored nominal strength of laterally unbraced I, channel, or Z-shaped members with equal compression and tension flanges, ϕM_u, shall be determined with $\phi = 0.90$ and

(a) For $M_y/M_e \leq 0.36$, $\quad M_u = M_y$ (Eq. 9.3.2-1)

(b) For $0.36 \leq M_y/M_e \leq 1.8$, $\quad M_u = M_y(\frac{10}{9})(1 - \frac{5}{18}(M_y/M_e))$ (Eq. 9.3.2-2)

(c) For $M_y/M_e \geq 1.8$, $\quad M_u = M_e$ (Eq. 9.3.2-3)

where $M_y = S_{xc}F_y$

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,

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

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

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

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*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.

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

(a) For $h/t \leq 171\sqrt{k_v/F_y}$

$\phi_v = 1.0$

$$V_u = A_w F_y/\sqrt{3} \quad \text{(Eq. 9.3.3-1)}$$

(b) For $171\sqrt{k_v/F_y} < h/t \leq 243\sqrt{k_v/F_y}$

$\phi_v = 0.90$

$$V_u = 110A_w\sqrt{k_v F_y}/(h/t) \quad \text{(Eq. 9.3.3-2)}$$

(c) For $h/t > 243\sqrt{k_v/F_y}$

$\phi_v = 0.90$

$$V_u = 26{,}700\ k_v A_w/(h/t)^2 \quad \text{(Eq. 9.3.3-3)}$$

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 = yield point of the beam web, ksi

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

t = base steel thickness of the web element, in.

k_v = shear buckling coefficient determined as follows:

1. For unreinforced webs, $k_v = 5.34$
2. For beam webs with transverse stiffeners satisfying the requirements of Section 8.4.4.2,

$$k_v = 4.00 + \frac{5.34}{(a/h)^2}, \text{ when } a/h < 1.0 \qquad \text{(Eq. 9.3.3-4)}$$

$$k_v = 5.34 + \frac{4.00}{(a/h)^2}, \text{ when } a/h > 1.0 \qquad \text{(Eq. 9.3.3-5)}$$

In the above expressions, a = shear panel length of the unreinforced web element, in. For a reinforced web element, a is the distance between transverse stiffeners, in.

9.3.3.2 Bending Strength of Beams Governed by Webs

The bending strength of beams shall also be limited by the factored strength governed by webs, $\phi_{bw} M_{ubw}$, determined from $\phi_{bw} = 0.90$

and $M_{ubw} = S_{eff} (\lambda F_y)$ (Eq. 9.3.3-6)

where ϕ_{bw} = resistance factor for bending governed by beam webs

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

F_y = yield point of the beam web, ksi

S_{eff} = compression elastic section modulus of the effective section determined by using full areas of the web and the tension flange and the effective compression flange area, $in.^3$. For beams having stiffened compression flanges, the effective compression area shall be determined according to Section 8.4.1. For beams having unstiffened compression flanges, the effective compression flange area is equal to the gross flange area times the stress ratio F_{cr}/F_y, where F_{cr} is the critical stress computed according to Section 8.5.

λ = $1.21-0.00034(h/t)\sqrt{F_y} \leq 1.0$ for beams having stiffened compression flanges

λ = $1.26-0.0005(h/t)\sqrt{F_y} \leq 1.0$ for beams having unstiffened compression flanges

9.3.3.3 Combined Bending and Shear in Webs

For unreinforced 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:

$$\left(\frac{V_D}{\phi_v V_u}\right)^2 + \left(\frac{M_D}{\phi_{bw} M_{ubw}}\right)^2 \leq 1.0 \qquad \text{(Eq. 9.3.3-7)}$$

For beam webs with transverse stiffeners satisfying the requirements of Section 8.4.4.2, the member may be proportioned so that the factored shear force and the factored bending moment do not exceed the values specified in Sections 9.3.3.1 and 9.3.3.2 and that

$$\left(\frac{V_D}{\phi_v V_u}\right) + 0.6\left(\frac{M_D}{\phi_{bw} M_{ubw}}\right) \leq 1.3 \qquad \text{(Eq. 9.3.3-8)}$$

when $M_D/(\phi_{bw}M_{ubw}) > 0.5$ and $V_D/(\phi_v V_u) > 0.7$,

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_{bw} = resistance factor for bending = 0.90

V_u = nominal maximum shear strength determined according to Section 9.3.3.1 except that the equation $V_u = 110\ A_w\sqrt{k_v F_y}\ /\ (h/t)$ shall be used for $h/t \leq 171\sqrt{k_v/F_y}$, kips

M_{ubw} = nominal maximum bending moment determined according to Section 9.3.3.2 except that for the computation of λ the limit of 1.0 shall not apply, kip-in.

9.3.3.4 Web Crippling of Flexural Members

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

9.3.3.4.1 Reactions and Concentrated Loads

To avoid crippling of unreinforced flat webs of flexural members having a flat width ratio, h/t, equal to or less than 200, neither concentrated loads nor reactions determined according to the factored design loads shall exceed the values of $\phi_w P_u$ with $\phi_w = 0.85$ and P_u given in Tables 9.3.3-1 and 9.3.3-2. Webs of flexural members for which the ratio, h/t, is greater than 200 shall be provided with adequate means of transmitting concentrated loads and/or reactions directly into the webs.

The following formulas apply to beams when $R/t \leq 6$ and to deck when $R/t \leq 7$, $N/t \leq 210$ and $N/h \leq 3.5$.

TABLE 9.3.3-1

P_u

Shapes Having Single Webs

At locations of one concentrated load or reaction acting either on the top or bottom flange, when the clear distance between the bearing edges of this and adjacent opposite concentrated loads or reactions is greater than 1.5h	For end reactions of beams or concentrated loads on the end of cantilevers when the distance from the edge of bearing to the end of the beam is less than 1.5h	Stiffened flanges $t^2kC_3C_4C_\Theta[331 - 0.61(h/t)][1 + 0.01(N/t)]$ Unstiffened flanges $t^2kC_3C_4C_\Theta[217 - 0.28(h/t)][1 + 0.01(N/t)]$*	(Eq. 9.3.3-9) (Eq. 9.3.3-10)
	For reactions and concentrated loads when the distance from the edge of bearing to the end of the beam is equal to or larger than 1.5h	Stiffened and unstiffened flanges $t^2kC_1C_2C_\Theta[538 - 0.74(h/t)][1 + 0.007(N/t)]$**	(Eq. 9.3.3-11)
At locations of two opposite concentrated loads or of a concentrated load and an opposite reaction acting simultaneously on the top and bottom flanges, when the clear distance between their adjacent bearing edges is equal to or less than 1.5h	For end reactions of beams or concentrated loads on the end of cantilevers when the distance from the edge of bearing to the end of the beam is less than 1.5h	Stiffened and unstiffened flanges $t^2kC_3C_4C_\Theta[244 - 0.57(h/t)][1 + 0.01(N/t)]$	(Eq. 9.3.3-12)
	For reactions and concentrated loads when the distance from the edge of bearing to the end of the beam is equal to or larger than 1.5h	Stiffened and unstiffened flanges $t^2kC_1C_2C_\Theta[771 - 2.26(h/t)][1 + 0.0013(N/t)]$	(Eq. 9.3.3-13)

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

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

TABLE 9.3.3-2

P_u

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

At locations of one concentrated load or reaction acting either on the top or bottom flange, when the clear distance between the bearing edges of this and adjacent opposite concentrated loads or reactions is greater than 1.5h	For end reactions of beams or concentrated loads on the end of cantilevers when the distance from the edge of bearing to the end of the beam is less than 1.5h	Stiffened and unstiffened flanges $t^2F_yC_7(10 + 1.25\sqrt{N/t})$	(Eq. 9.3.3-14
	For reactions and concentrated loads when the edge of bearing to the end of the beam is equal to or larger than 1.5h	Stiffened and unstiffened flanges $t^2F_yC_5C_6(15 + 3.25\sqrt{N/t})$	(Eq. 9.3.3-15
At locations of two opposite concentrated loads or of a concentrated load and an opposite reaction acting simultaneously on the top and bottom flanges, when the clear distance between their adjacent bearing edges is equal to or less than 1.5h	For end reactions of beams or concentrated loads on the end of cantilevers when the distance from the edge of bearing to the end of the beam is less than 1.5h	Stiffened and unstiffened flanges $t^2F_yC_{10}C_{11}(10 + 1.25\sqrt{N/t})$	(Eq. 9.3.3-16
	For reactions and concentrated loads when the distance from the edge of bearing to the end of the beam is equal to or larger than 1.5h	Stiffened and unstiffened flanges $t^2F_yC_8C_9(15 + 3.25\sqrt{N/t})$	(Eq. 9.3.3-17

In all of the above, P_u represents the nominal ultimate load or reaction for one solid web connecting top and bottom flanges. For shapes consisting of two or more such adjacent webs, P_u shall be computed for each individual sheet and the results added to obtain the load or reaction for the multiple web.

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

In the above formulas,

ϕ_w = resistance factor for web crippling = 0.85

P_u = nominal ultimate concentrated load or reaction, kips, per web

$C_1 = (1.22 - 0.22k)$ (Eq. 9.3.3-18)

$C_2 = (1.06 - 0.06R/t) \leq 1.0$ (Eq. 9.3.3-19)

$C_3 = (1.33 - 0.33k)$ (Eq. 9.3.3-20)

$C_4 = (1.15 - 0.15R/t) \leq 1.0$ but not less than 0.50 (Eq. 9.3.3-21)

$C_5 = (1.49 - 0.53k) \geq 0.6$ (Eq. 9.3.3-22)

$C_6 = (0.88 + 0.12m)$ (Eq. 9.3.3-23)

$C_7 = 1 + \frac{h/t}{750}$ when $h/t \leq 150$;

$C_7 = 1.20$ when $h/t > 150$ (Eq. 9.3.3-24)

$C_8 = 1/k$, when $h/t \leq 66.5$; $C_8 = \left[1.10 - \frac{h/t}{665}\right]/k$ when $h/t > 66.5$ (Eq. 9.3.3-25)

$C_9 = (0.82 + 0.15m)$ (Eq. 9.3.3-26)

$C_{10} = \left[0.98 - \frac{h/t}{865}\right]/k$ (Eq. 9.3.3-27)

$C_{11} = (0.64 + 0.31m)$ (Eq. 9.3.3-28)

$C_\theta = 0.7 + 0.3(\theta/90)^2$ (Eq. 9.3.3-29)

F_y = yield point of the web, ksi

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

$k = F_y/33$ (Eq. 9.3.3-30)

$m = t/0.075$ (Eq. 9.3.3-31)

t = web thickness, in.

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

R = inside bend radius, in.

θ = angle between plane of web and plane of bearing surface $\geq 45°$ but no more than $90°$

9.3.3.4.2 Combined Bending and Web Crippling

Unreinforced flat webs of shapes subjected to a combination of bending and reaction or concentrated load shall be designed to meet the following requirements:

(a) Shapes having single webs

$$1.07 \frac{P_D}{\phi_w P_u} + \frac{M_D}{\phi_b M_u} \leq 1.42 \quad \text{(Eq. 9.3.3-32)}$$

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

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

$$0.82\ \frac{P_D}{\phi_w P_u} + \frac{M_D}{\phi_b M_u} \leq 1.32 \qquad \text{(Eq. 9.3.3-33)}$$

Exception: When $h/t \leq 400/\sqrt{F_y}$ and $w/t \leq (w/t)_{lim}$, the reaction or concentrated load may be determined by Section 9.3.3.4.1 without considering the effect of bending moment on the reduction of the web crippling load.

In the above formulas,

ϕ_b = resistance factor for bending = 0.90

ϕ_w = resistance factor for web crippling = 0.85

P_D = concentrated load or reaction in the presence of bending moment computed on the basis of factored loads, kips

P_u = nominal ultimate concentrated load or reaction in the absence of bending moment determined in accordance with Section 9.3.3.4.1, kips

M_D = applied bending moment, at or immediately adjacent to the point of application of the concentrated load or reaction P_D computed on the basis of factored loads, kip-in.

M_u = nominal ultimate bending moment permitted if bending stress only exists. 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.

w = flat width of the beam flange which contacts the bearing plate, in.

t = thickness of web or flange, in.

$(w/t)_{lim}$ = limiting w/t ratio for the beam flange. Use Sections 8.4.1.1 and 8.5(a) for stiffened flanges and unstiffened flanges, respectively.

9.3.4 Serviceability

Deflection of beams shall be computed 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 or 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 or torsional-flexural buckling, and for members braced against twisting, the factored axial strength, $\phi_c P_u$, shall be determined from $\phi_c = 0.85$ and

(a) For $KL/r \leq C_c/\sqrt{Q}$, $P_u = AQF_y[1- \frac{QF_y}{4\pi^2 E}(\frac{KL}{r})^2]$ (Eq. 9.4.1-1)

(b) For $KL/r > C_c/\sqrt{Q}$, $P_u = \frac{\pi^2 EA}{(KL/r)^2}$ (Eq. 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.

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*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 the compression members shall be taken as unity, unless analysis shows that a smaller value may be used. In a frame which depends upon its own bending stiffness for lateral stability, the effective length KL of the compression members shall be determined by a rational method and shall not be less than the actual unbraced length.

$F_y = F_y$ or F_{ya}, as appropriate, ksi

Q = factor determined as follows:

(a) For members composed entirely of stiffened elements

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

where A_{eff} is the effective area as determined for the effective design widths from Section 8.4 for $f_{max} = F_y$.

(b) For members composed entirely of unstiffened elements

$$Q = Q_s = \frac{F_{cr}}{F_y} \qquad \text{(Eq. 9.4.1-4)}$$

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

$$Q = Q_a Q_s \qquad \text{(Eq. 9.4.1-5)}$$

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, the factored axial strength, $\phi_c P_u$, shall be determined from $\phi_c = 0.85$ and the load P_u which is the smaller of the values determined from Section 9.4.1 and the following formulas:

For $\sigma_{TFO} > 0.5QF_y$, $$P_u = AQF_y\left(1 - \frac{QF_y}{4\sigma_{TFO}}\right) \qquad \text{(Eq. 9.4.2-1)}$$

For $\sigma_{TFO} \leq 0.5QF_y$, $$P_u = A\sigma_{TFO} \qquad \text{(Eq. 9.4.2-2)}$$

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:

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

where

$$\sigma_{ex} = \frac{\pi^2 E}{(KL/r_x)^2}, \text{ ksi} \qquad \text{(Eq. 9.4.2-4)}$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(KL)^2}\right], \text{ ksi} \qquad \text{(Eq. 9.4.2-5)}$$

$$\beta = 1 - (x_o/r_o)^2 \qquad \text{(Eq. 9.4.2-6)}$$

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. (Eq. 9.4.2-7)

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,

$$J = \frac{1}{3}(l_1 t_1^3 + l_2 t_2^3 + \ldots + l_i t_i^3 \ldots + l_n t_n^3) \qquad \text{(Eq. 9.4.2-8)}$$

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 Point-Symmetric Sections Which May Be Subject to Torsional Buckling

For point-symmetric open shapes such as cruciform sections or such built-up shapes which may be subject to torsional buckling and which are not braced against twisting, the factored axial strength, $\phi_c P_u$, shall be determined from $\phi_c = 0.85$ and the load P_u which is the smaller of the values determined from Section 9.4.1 and the following formulas:

For $\sigma_t > 0.5\ QF_y$, $\quad P_u = AQF_y(1 - \frac{QF_y}{4\sigma_t})$ (Eq. 9.4.3-1)

For $\sigma_t \leq 0.5\ QF_y$, $\quad P_u = A\sigma_t$ (Eq. 9.4.3-2)

where σ_t is defined in Section 9.4.2.

If the section consists entirely of unstiffened elements Q shall be taken as 1.0, otherwise Q shall be determined in accordance with Section 9.4.1.

9.4.4 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 or Torsional Flexural Buckling

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

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

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

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:

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

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), $C_m = 0.85$ (Eq. 9.5.1-4)

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,

$$C_m = 0.6 - 0.4\frac{M_1}{M_2}, \text{ but not less than } 0.4 \qquad \text{(Eq. 9.5.1-5)}$$

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, $C_m = 1.0$. (Eq. 9.5.1-6) (Eq. 9.5.1-7)

E = modulus of elasticity = 29,500 ksi

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.

M_{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

$$P_{Ex} = \frac{\pi^2 EI_x}{(KL)_x^2}\text{, kips} \qquad \text{(Eq. 9.5.1-8)}$$

$$P_{Ey} = \frac{\pi^2 EI_y}{(KL)_y^2}\ , \quad \text{kips} \qquad \text{(Eq. 9.5.1-9)}$$

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

$$P_{us} = AQ_sQ_aF_y, \text{ kips} \qquad \text{(Eq. 9.5.1-10)}$$

ϕ = 0.95 for bending strength (Sec. 9.3.1) or 0.90 for laterally unbraced beams (Sec. 9.3.2)

ϕ_c = 0.85

ϕ_s = 0.95

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)

$$\frac{P_D}{\phi_c P_{uc}} + \frac{C_m M_D}{\phi_s M_{us}\left(1 - \frac{P_D}{\phi_c P_{Ey}}\right)} \leq 1.0 \qquad \text{(Eq. 9.5.2-1)}$$

and

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

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:

$$\frac{P_D}{\phi_c P_{uc}} + \frac{M_D}{\phi_s M_{us}} \leq 1.0 \qquad \text{(Eq. 9.5.2-3)}$$

(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

$$P_D \leq \phi_c P_u \qquad \text{(Eq. 9.5.2-4)}$$

where for $\sigma_{TF} > 0.5QF_y$, $\quad P_u = AQF_y(1-\frac{QF_y}{4\sigma_{TF}})$ (Eq. 9.5.2-5)

and for $\sigma_{TF} \leq 0.5QF_y$, $\quad P_u = A\sigma_{TF}$ (Eq. 9.5.2-6)

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

$$\frac{\sigma_{TF}}{\sigma_{TFO}} + \frac{C_{TF}\sigma_{b1}}{\sigma_{bT}(1 - \frac{\sigma_{TF}}{\sigma_e})} = 1.0 \qquad \text{(Eq. 9.5.2-7)}$$

(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,

$$P_D \leq \phi_c P_{uc2} + \frac{e}{x_o}(P_{DE} - \phi_c P_{uc2}) \qquad \text{(Eq. 9.5.2-8)}$$

(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},

$$P_D \leq \phi_c P_{uc2} + \frac{e}{x_o}(P_{DC} - \phi_c P_{uc2}) \qquad \text{(Eq. 9.5.2-9)}$$

(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 $\sigma_{TF} > 0.5QF_y$, $P_u = AQF_y(1 - \frac{QF_y}{4\sigma_{TF}})$ (Eq. 9.5.2-10)

and for $\sigma_{TF} \leq 0.5QF_y$, $P_u = A\sigma_{TF}$ (Eq. 9.5.2-11)

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

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

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), $C_{TF} = 0.85$ (Eq. 9.5.2-13)
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

$$C_{TF} = 0.6 - 0.4\frac{M_1}{M_2} \qquad \text{(Eq. 9.5.2-14)}$$

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 maximun 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 (Eq. 9.5.2-15)

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. (Eq. 9.5.2-16)

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. (Eq. 9.5.2-17)

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

P_{DC} = 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 $\phi_c P_u$ in requirement (iv b) for $e = x_o$, kips

P_{DE} = 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 papallel 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.

$\sigma_{bC} = \dfrac{M_c c}{I_y}$ = maximum compression bending stress caused by M_c, ksi. For I-sections with unequal flanges σ_{bC} may be approximated by $\dfrac{\pi^2 EdI_{xc}}{L^2 S_{yc}}$ (Eq. 9.5.2-18)

$\sigma_{bT} = \dfrac{M_T c}{I_y}$ = maximum compression bending stress caused by M_T, ksi. For I-sections with unequal flanges σ_{bT} may be approximated by $\dfrac{\pi^2 EdI_{xc}}{L^2 S_{yc}}$ (Eq. 9.5.2-19)

$\sigma_{b1} = \sigma_{TF}\dfrac{ec}{r_y^2}$ = maximun compressive bending stress in the section caused by σ_{TF}, ksi (Eq. 9.5.2-20)

$$\sigma_{b2} = \sigma_{TF}\frac{x_o c}{r_y^2}, \text{ ksi} \qquad \text{(Eq. 9.5.2-21)}$$

$$\sigma_e = \frac{\pi^2 E}{(KL_b/r_b)^2}, \text{ ksi} \qquad \text{(Eq. 9.5.2-22)}$$

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, σ_{ex}, σ_t, σ_{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 $\phi = 0.95$ and M_{us} is determined below:

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

where F_{cr} is computer as follows:

(a) For $D/t \leq \dfrac{3300}{F_y}$

$$F_{cr} = F_y \qquad \text{(Eq.9.6.1-1)}$$

(b) For $\dfrac{3300}{F_y} < D/t \leq \dfrac{13000}{F_y}$

$$F_{cr} = \frac{1104}{(D/t)} + 0.666F_y \qquad \text{(Eq.9.6.1-2)}$$

in which

D = mean diameter of the cylindrical tube, in.

t = wall thickness of the tube, in.

F_y = F_y or F_{ya} 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 $\phi = 0.85$

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

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

A = cross-sectional area, $in.^2$

F_{cr} = stress computed according to Section 9.6.1

9.7 INELASTIC RESERVE CAPACITY OF FLEXURAL MEMBERS

The inelastic flexural reserve capacity may be used when the following conditions are met:

(a) The member is not subject to twisting, lateral, torsional, or torsional-flexural buckling

(b) The effect of cold-forming is not included in determining the yield point F_y

(c) The ratio of the depth of the compressed portion of the web to its thickness does not exceed $190/\sqrt{F_y}$

(d) The depth to thickness ratio of the entire web does not exceed $640/\sqrt{F_y}$

(e) The shear force does not exceed $0.58F_y$ times the web area

(f) The angle between any web and the vertical does not exceed 20 degrees

The factored nominal bending strength, ϕM_{u1}, shall be determined with $\phi = 0.95$ and M_{u1} is either 1.25 M_y or M_u, whichever is smaller; where

M_y = moment causing a maximum strain of e_y, kip-in.

e_y = yield strain = F_y/E

E = modulus of elasticity = 29,500 ksi

M_u = ultimate moment causing a maximum compression strain of $C_y e_y$ (no limit is placed on the maximum tensile strain), kip-in.

C_y = a factor determined as follows:

(1) Stiffened compression elements without intermediate stiffeners

$C_y = 3$ for w/t less than or equal to $190/\sqrt{F_y}$ (Eq. 9.7-1)

$C_y = 3 - [(w/t)\sqrt{F_y} - 190]/15.5$ (Eq. 9.7-2)

for w/t greater than $190/\sqrt{F_y}$ but not greater than $221/\sqrt{F_y}$

$C_y = 1$ for w/t greater than $221/\sqrt{F_y}$ (Eq. 9.7-3)

(2) Unstiffened compression elements

$C_y = F_{cr}/F_y$ (Eq. 9.7-4)

F_{cr} is defined in Section 8.5 and

F_y is the minimum specified yield point

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

$C_y = 1$ (Eq. 9.7-5)

When applicable effective design widths shall be used in calculating section properties. M_u shall be calculated considering equilibrium of stresses, assuming an ideally elastic-plastic stress-strain curve which is the same in tension as in compression, assuming small deformations and assuming that plane sections before bending remain plane during flexure.

Combined bending and web crippling shall be checked by the provisions of Section 9.3.3.4.2.

SECTION 10 - CONNECTIONS

10.1 General

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

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 Arc-Welds

10.2.1.1 Thickness Limitations

The following LRFD design criteria govern welded connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is 0.18 in. or less. For welded connections in which the thickness of the thinnest connected part is greater than 0.18 in., refer to the AISC's "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings," November 1, 1978.

The allowable stress design provisions of Section 4.2.1.2 shall also apply to the welded connections designed in accordance with the LRFD criteria specified in this section.

10.2.1.2 Groove Welds in Butt Joints

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

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

$\phi = 0.90$

$$R_n = Lt_w F_y \qquad \text{(Eq. 10.2.1-1)}$$

(b) shear on the effective area

$$\phi = 0.80, \quad R_n = Lt_e(0.6F_{xx}) \qquad \text{(Eq. 10.2.1-2)}$$

$$\text{and } \phi = 0.90, \quad R_n = Lt_e(F_y/\sqrt{3}) \qquad \text{(Eq. 10.2.1-3)}$$

where

ϕ = resistance factor for welded connections

R_n = nominal ultimate strength of a groove weld, kips

F_{xx} = strength level designation in AWS electrode classification, ksi

F_y = specified minimum yield point of steel, ksi

L = length of weld, in.

t_e = effective throat dimension for groove weld, in.

10.2.1.3 Arc Spot Welds

The factored nominal strength, ϕR_n, of each arc spot weld between sheet or sheets and supporting member shall be determined by using the smaller of either

$$\text{(a) } \phi = 0.70, \quad R_n = \frac{\pi d_e^2}{4}(0.6F_{xx}) \qquad \text{(Eq. 10.2.1-4)}$$

or (b) For $\frac{d_a}{t} \leq \frac{114}{\sqrt{F_u}}$

$$\phi = 0.60, \quad R_n = 2.2\, t\, d_a\, F_u \qquad \text{(Eq. 10.2.1-5)}$$

For $\frac{114}{\sqrt{F_u}} < \frac{d_a}{t} < \frac{240}{\sqrt{F_u}}$

$$\phi = 0.50, \quad R_n = 0.28\left[1+\frac{960t}{d_a\sqrt{F_u}}\right]t\, d_a\, F_u \qquad \text{(Eq. 10.2.1-6)}$$

For $\frac{d_a}{t} \geq \frac{240}{\sqrt{F_u}}$:

$$\phi = 0.50 \quad R_n = 1.4\, t\, d_a\, F_u \qquad \text{(Eq. 10.2.1-7)}$$

where

ϕ = resistance factor for welded connections

R_n = nominal ultimate strength of an arc spot weld, kips

d = visible diameter of outer surface of arc spot weld, in.

d_a = average diameter of the arc spot weld at mid-thickness of t, in. (where d_a = (d-t) for a single sheet, and (d-2t) for multiple sheets (not more than four lapped sheets over a supporting member)), in.

d_e = effective diameter of fused area, in.

d_e = 0.7d - 1.5t but $\leq$ 0.55d (Eq. 10.2.1-8)

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

F_{xx} = strength level designation in AWS electrode classification, ksi

F_y = specified minimum yield point of steel, ksi

F_u = specified minimum tensile strength of steel, ksi

Note: See Figure 4.2 for diameter definitions.

The allowable design provisions of Section 4.2.1.2.2 for thickness limitations, weld washers, minimum effective diameter of welds, minimum spacing, and edge distance shall also apply to the arc spot welds designed in accordance with this section.

10.2.1.4 Arc Seam Welds

The factored nominal strength, ϕR_n, of arc seam welds shall be determined by using the smaller of either

(a) $\phi = 0.70$, $R_n = [\frac{\pi d_e^2}{4} + L d_e](0.6 F_{xx})$ (Eq. 10.2.1-9)

or (b) $\phi = 0.60$, $R_n = (0.63\ L + 2.4\ d_a)\ t F_u$ (Eq. 10.2.1-10)

where

ϕ = resistance factor for welded connections

R_n = nominal ultimate strength of an arc seam weld, kips

d = width of arc seam weld, in.

L = length of seam weld not including the circular ends, in. (For computation purposes, L shall not exceed 3d.)

d_a = average width of seam weld, in. (where

d_a = (d-t) for a single sheet, and (Eq. 10.2.1-11)

(d-2t) for a double sheet) (Eq. 10.2.1-12)

d_e = effective width of arc seam weld at fused surfaces

d_e = 0.7d - 1.5t, in. (Eq. 10.2.1-13)

F_u and F_{xx} are defined in Section 10.2.1.3.

The minimum edge distance shall be as determined for the arc spot weld, Section 4.2.1.2.2 (See Figure 4.5).

10.2.1.5 Fillet Welds

The factored nominal strength, ϕR_n, of a fillet weld shall be determined as follows:

(a) For longitudinal loading:

For $L/t < 25$:

$\phi = 0.60$, $R_n = (1-0.01 \frac{L}{t})\ tLF_u$ (Eq. 10.2.1-14)

For $L/t \geq 25$:

$\phi = 0.60$, $R_n = 0.75t\ L\ F_u$ (Eq. 10.2.1-15)

(b) For transverse loading:

$\phi = 0.60$, $R_n = t\ L\ F_u$ (Eq. 10.2.1-16)

In addition, for $t > 0.15$ in., the factored nominal strength determined above shall not exceed the following value of ϕR_n:

$\phi = 0.70$, $R_n = 0.6\ t_w LF_{xx}$ (Eq. 10.2.1-17)

where

ϕ = resistance factor for welded connections

R_n = nominal ultimate strength of a fillet weld, kips

L = length of fillet weld, in.

t_w = effective throat = 0.707 w_1 or 0.707 w_2, whichever is smaller. A larger effective throat may be taken if it can be shown by measurement that a given welding procedure will consistently give a larger value providing the particular welding procedure used for making the welds that are measured is followed.

w_1 and w_2 = leg on weld (See Figure 4.6)

F_u and F_{xx} are defined in Section 10.2.1.3.

10.2.1.6 Flare Groove Welds

The factored nominal strength, ϕR_n, of a flare groove weld shall be determined as follows:

(a) For flare-bevel groove welds, transverse loading (Fig.4.7):

$\phi = 0.55$, $R_n = 0.8\ tLF_u$ (Eq. 10.2.1-18)

(b) For flare groove welds, longitudinal loading (Fig. 4.8):

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

$$\phi = 0.55, \quad R_n = 0.75\, t\, L\, F_u \qquad \text{(Eq. 10.2.1-19)}$$

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

$$\phi = 0.55, \quad R_n = 1.5\, t\, L\, F_u \qquad \text{(Eq. 10.2.1-20)}$$

In addition, if $t > 0.15$ in., the factored nominal strength determined above shall not exceed the following value of ϕR_n:

$$\phi = 0.70, \quad R_n = 0.6\, t_w L F_{xx} \qquad \text{(Eq. 10.2.1-21)}$$

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 AISC's "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings," November 1, 1978.

The allowable stress design provisions of Sections 4.5.2 and 4.5.3 for materials and bolt installation shall also apply to the bolted connections designed in accordance with the LRFD criteria specified in this section.

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:

(a) when $F_u/F_y \geq 1.15$:

$$\phi = 0.70, \quad R_n = teF_u \qquad \text{(Eq. 10.3.1-1)}$$

(b) when $F_u/F_y < 1.15$:

$$\phi = 0.70, \quad R_n = (0.9)teF_u \qquad \text{(Eq. 10.3.1-2)}$$

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*

*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.

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

t = thickness of the connected part, in.

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

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

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

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

ϕ = 0.65 for double shear connection

ϕ = 0.60 for single shear connection

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

$$\phi = 0.65$$

$$R_n = (1.0 - r + 2.5rd/s)\ F_u A_n \leq F_u A_n \qquad \text{(Eq. 10.3.3-2)}$$

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

$$\phi = 0.90$$

$$R_n = F_y A_n \qquad \text{(Eq. 10.3.3-3)}$$

where A_n = net area of the connected part, in.^{2}

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 (Inches)	Type of Joint	F_u/F_y Ratio of Connected Part	Resistance Factor ϕ	Nominal Resistance R_n (kips)
$\geq$ 0.024 but < 3/16	Inside sheet of double shear connection	$\geq$ 1.15	0.60	$3.5F_u dt$
		< 1.15	0.70	$3.0F_u dt$
	Single shear and outside sheets of double shear connection	No limit	0.65	$3.0F_u dt$
> 3/16	See Section 10.3.1			

Table 10.3.4 (B)

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

Thickness of Connected Part (Inches)	Type of Joint	F_u/F_y Ratio of Connected	Resistance Factor	Nominal Resistance R_n (kips)
$\geq$ 0.036 but < 3/16	Inside Sheet of double shear connection	$\geq$ 1.15	0.70	$3.0F_u dt$
	Single shear and outside sheets of double shear connection	$\geq$ 1.15	0.70	$2.2F_u dt$
> 3/16	See Section 10.3.1			

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:

$$R_n = 0.6mA_{sA}F_u \qquad \text{(Eq. 10.3.5-1)}$$

$\phi = 0.65$ for A307 bolts

$\phi = 0.65$ for A325 and A449 bolts

$\phi = 0.65$ 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.

F_u = nominal tensile strength of bolt, ksi

The symbols ϕ and R_n 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 cold-formed 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 1980 Edition of the AISI Specification for allowable stress design 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 1980 AISI Specification (Ref. 1), is based on the 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 (Refs. 2 and 17)(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:

$$\phi R_n \geq \sum \gamma_i Q_i \qquad \text{(C8.3-1)}$$

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

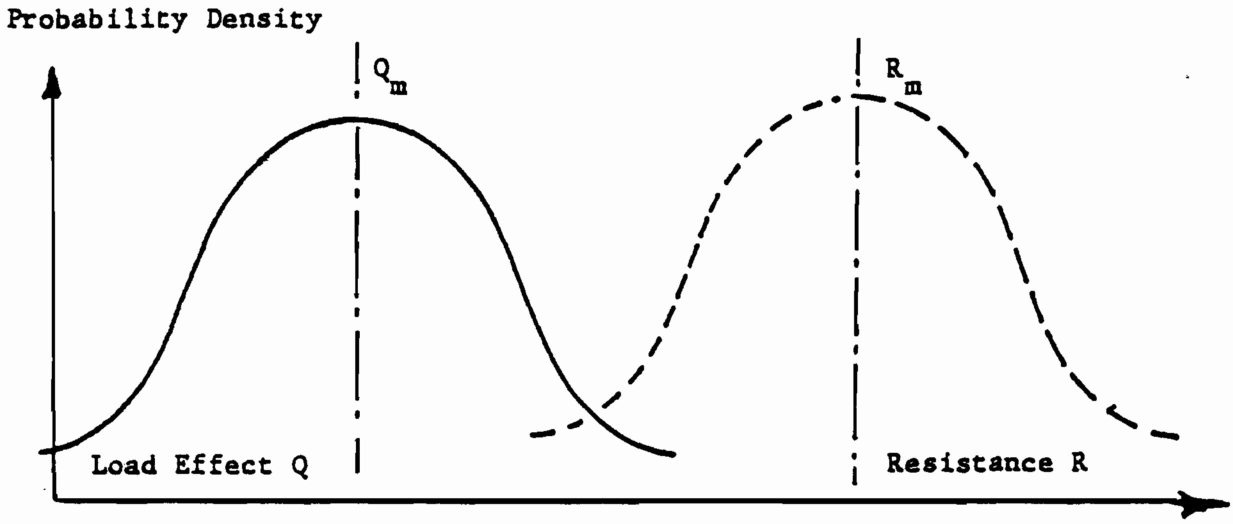

Fig. C8.3-1 Definition of the Randomness of Q and R

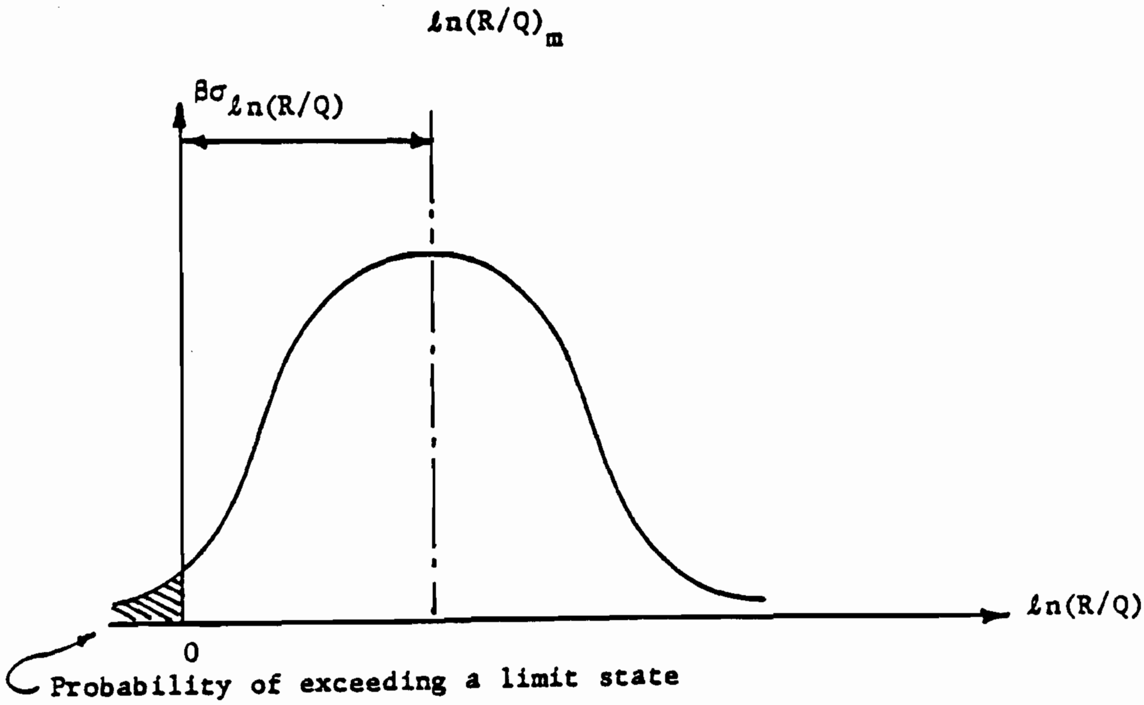

Fig. C8.3-2 Definition of the Reliability Index β the limit state. If the exact probability distributions of Q and R were known, then the probability of R - Q < 1 could be exactly determined for any design. In general the distributions of Q and R are not known, and only the means, Q_m and R_m, and the 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 $\ell n(R/Q)$, and a limit state is exceeded when $\ell n(R/Q) \leq 0$ 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 $\ell n(R/Q)$. For given statistical data R_m, Q_m, σ_R and σ_Q, the area under $\ell n(R/Q) \leq 0$ can be varied by changing the value of β (Fig. C8.3-2), since $\beta\, \sigma_{\ell n(R/Q)} = \ell n(R/Q)_m$, from which approximately

$$\beta = \frac{\ell n(R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}} \qquad \text{(C8.3-2)}$$

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 1980 AISI Specification for such a beam is

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

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

FS = 5/3 = the factor of safety

F_y = the specified yield point

ℓ = the span length and s = 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)

$$R_m = R_n(P_m M_m F_m) \tag{C8.3-4}$$

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

$$R_n = S_{eff} F_y \tag{C8.3-5}$$

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

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

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

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

and

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

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 C8.3-8 gives

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

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

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 the purposes of checking the reliability of these LRFD criteria it will be assumed that $D_n/L_n = 1/5$, and so $Q_m = 1.21 L_n (\ell^2 s/8)$ and $V_Q = 0.21$.

From Eq. C8.3-3 we obtain the nominal design capacity for $D_n/L_n = 1/5$ and FS = 5/3. Thus

$$\frac{R_m}{Q_m} = \frac{1.19 \times 2.00 \times L_n(\ell^2 s/8)}{0.21\, L_n(\ell^2 s/8)} = 1.97$$

and, from Eq. C8.3-2:

$$\beta = \frac{\ln 1.96}{\sqrt{0.15^2 + 0.21^2}} = 2.63$$

Of itself $\beta = 2.63$ for beams with stiffened compression flanges designed by the 1980 AISI Specification means nothing. However, when this is compared to β for other types of cold-formed members, and to β for designs of various types from hot-rolled steel shapes or even for other materials, then it is possible to say that this particular cold-formed steel beam has about an average reliability (Ref. 15).

Basis for LRFD of Cold-Formed Structures

A great deal of work has been performed for determining the values of the reliability index β inherent in traditional design as exemplified by the current structural design specifications such as the AISC Specification for hot-rolled steel, the AISI Specification for cold-formed steel (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 Refs. 4, 15 and 16 by using a) updated load statistics and b) a more advanced level of probability analysis which was able to incorporate probability distributions which describe the true distributions more realistically. The details of this extensive reanalysis are presented in Refs. 4, 15 and 16 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 in Ref. 16 that the following values of β would provide this improved consistency while at the same time give, on the average, essentially the same design by the new LRFD method as is obtained by current design for all materials of construction. These target reliabilities β_o for use in LRFD are:

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

For connections: $\beta_o = 4.5$

For wind loading: $\beta_o = 2.5$

These target reliability indices are the ones inherent in the load factor recommended in the ANSI A58.1-82 Load Code (Ref. 10). For cold-formed steel members it was shown previously that for the representative dead-to-live-load ratio of 1/5 the reliability index is 2.6. Since design according to the 1980 edition of the AISI Allowable Stress Design specification has shown itself to result in members with satisfactory performance, the basic reliability index of $\beta_o = 2.5$ will be used herein for the development of the resistance factors ϕ for the LRFD criteria for cold-formed steel structures.

2) The following load factors and load combinations were developed in Refs.4 and 16 to give essentially the same β's as the target β_o's, and are recommended for use with the 1982 ANSI Load Code (Ref. 10) for all materials, including cold-formed steel:

$$1.4D_n$$

$$1.2D_n + 1.6L_n + 0.5(L_{rn} \text{ or } S_n \text{ or } R_n)$$

$$1.2D_n + 1.6(L_{rn} \text{ or } S_n \text{ or } R_n) + (0.5L_n \text{ or } 0.8W_n)$$

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

$$1.2D_n + 1.5E_n + (0.5L_n \text{ or } 0.2S_n)$$

$$0.9D_n - (1.3W_n \text{ or } 1.5E_n)$$

where D_n = nominal dead load

E_n = nominal earthquake load

L_n = nominal live load due to occupancy; weight of wet concrete for composite construction

L_{rn} = nominal roof live load

R_n = nominal roof rain load

S_n = nominal snow load

W_n = nominal wind load

Because of special circumstances inherent in cold-formed steel structures, the following additional LRFD criteria apply for roof, floor and wall construction using cold-formed steel:

a) For roof and floor construction

$$1.2D_n + 1.6C_{wn} + 1.4C_n$$

where C_{wn} = nominal weight of wet concrete during construction

C_n = nominal construction load, including equipment, workmen and formwork, but excluding the weight of the wet concrete.

This criterion has been added to provide safe construction practices for cold-formed steel decks and panels which otherwise could be damaged during erection.

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

Deflection calculations for serviceability criteria 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 approximately provide a target β_o of 2.5 for members and 4.0 for connections, respectively, for the load combination $1.2D_n + 1.6L_n$. For practical reasons it is desirable to have relatively few different resistence factors, and so the actual values of β will differ from the derived targets. This means that

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

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

By assuming $D_n/L_n = 1/5$, Eqs. C8.3-11 and C8.3-9 can be rewritten as follows:

$$R_n = 1.84(cL_n/\phi) \tag{C8.3-12}$$

$$Q_m = (1.05D_n/L_n + 1)cL_n = 1.21cL_n \tag{C8.3-13}$$

Therefore,

$$\frac{R_m}{Q_m} = \left(\frac{1.521}{\phi}\right)\left(\frac{R_m}{R_n}\right) \tag{C8.3-14}$$

The ϕ factors can be computed from Eq. C8.3-14 and the following equation by using $V_Q = 0.21$:

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

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

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 and 8.4.1-2 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 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 1980 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:

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

The subscript m refers to mean values. The symbol V stands for coefficient of variation. The symbols M and F are, respectively, the 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 $\phi = 0.95$ 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 approximately 2.5. In the determination of the resistance factor, the following formulas were used for R_m and R_n:

$$R_m = A_n (F_y)_m \tag{C9.2-1}$$

$$R_n = A_n F_y \tag{C9.2-2}$$

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

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:

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

Based on $V_Q = 0.21$ and the resistance factor of 0.95, the value of β is 2.4, which is close to the stated target value of $\beta \simeq 2.5$.

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 1980 AISI Specification (Ref. 1). References 2 and 17 provide 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 value of β, as determined from Eq. C8.3-2 for the selected value of $\phi = 0.95$ for a dead-to-live load ratio of 1/5 is 2.44.

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 1980 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 these LRFD criteria is to provide only a translation from the 1980 Allowable Stress Design criteria (Ref. 1) 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.

For a dead-to-live load ratio of $D_n/L_n = 1/5$, the load effect data is $Q_m = \left(\frac{1.05D_n}{L_n} + 1\right) cL_n = 1.21\ cL_n$ and $V_Q = 0.21$ (see Sec. C8.3 of this Commentary). According to the LRFD load factors

$$\phi\ R_n = c\ (1.2\ D_n + 1.6\ L_n) = cL_n\ (1.2\ D_n/L_n + 1.6) = 1.84cL_n$$

or $$R_n = \frac{1.84\ cL_n}{\phi}$$

Thus $$R_m/Q_m = (R_m/R_n)\ (\frac{1}{\phi})\ (\frac{1.84}{1.21})$$

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

The following table summarizes the results:

R_m/R_n	V_R	ϕ	β
1.36	0.17	0.95	2.9
1.76	0.22	0.95	3.4

The selected value of $\phi = 0.95$ in the LRFD criteria thus furnishes a greater reliability than the target value of $\beta_o = 2.5$

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:

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

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:

$P_m = 1.15$ and $V_P = 0.15$

$M_m = 1.0$ and $V_M = 0.06$

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

and thus

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

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 recommended resistance factor $\phi = 0.90$ gives $\beta = 2.5$. It should also be noted that a simplified and conservative form of Eq. C9.3.2-1 is the basis of the recommended design criteria, which is the same as the Allowable Stress design rules in the 1980 AISI Specification, i.e., 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.:

$$V_u = A_w \tau_y = A_w F_y/\sqrt{3} \qquad \text{(C9.3.3-1)}$$

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.:

$$V_u = A_w \tau_{cr} = \frac{k_v \pi^2 E A_w}{12(1-\mu^2)(h/t)^2} \qquad \text{(C9.3.3-2)}$$

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

$$V_u = \frac{26{,}700\, k_v\, A_w}{(h/t)^2} \qquad \text{(C9.3.3-3)}$$

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

$$V_u = \left(\frac{5}{3}\right)\left[\frac{65.7\sqrt{k_v F_y}}{(h/t)}\right]A_w = \frac{110\sqrt{k_v F_y}}{(h/t)} A_w \qquad \text{(C9.3.3-4)}$$

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,

$$(R_n)_{LRFD} = (R_n)_{ASD} \qquad \text{(C9.3.3-5)}$$

Since

$$(R_n)_{LRFD} \geq c(1.2D_n + 1.6L_n)/\phi \qquad \text{(C9.3.3-6)}$$

$$(R_n)_{ASD} \geq c(F.S.)(D_n + L_n) \qquad \text{(C9.3.3-7)}$$

the resistance factors can be computed from the following formula:

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

$$= \frac{1.2(D_n/L_n) + 1.6}{(F.S.)(D_n/L_n + 1)} \qquad \text{(C9.3.3-8)}$$

By using a dead-to-live load ratio of $D_n/L_n = 1/5$, 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.C9.3.3-8	Recommended ϕ Factor
$h/t \leq 171\sqrt{k_v/F_y}$	1.44	1.06	1.00
$171\sqrt{k_v/F_y} \leq h/t \leq 243\sqrt{k_v/F_y}$	1.67	0.92	0.90
$h/t > 243\sqrt{k_v/F_y}$	1.71	0.90	0.90

9.3.3.2 Bending Strength of Beams Governed by Webs

In Section 9.3.1, the factored nominal bending strength is based on either yielding or local buckling of the beam flange. In Section 9.3.3.2, the bending strength of beams is governed by either yielding or buckling of beam webs. Equation 9.3.3-6 was derived from the allowable stress equation given in Section 3.4.2 of the AISI Specification and a safety factor of 5/3.

The bending strength of beams governed by webs was studied in Refs. 9 and 18 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 (Table III of Ref. 18):

(a) Beams having stiffened flanges

$P_m = 1.00$; $V_P = 0.08$

$M_m = 1.10$; $V_M = 0.10$

$F_m = 1.00$; $V_F = 0.05$

$R_m/R_n = 1.10$; $V_R = 0.14$

(b) Beams having unstiffened flanges

$P_m = 0.99$; $V_P = 0.09$

$M_m = 1.10$; $V_M = 0.10$

$F_m = 1.00$; $V_F = 0.05$

$R_m/R_n = 1.09$; $V_R = 0.14$

For $\phi = 0.90$, the computed β's are 2.46 and 2.42 for beams having stiffened flanges and beams having unstiffened flanges, respectively.

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 of Flexural Members

The nominal ultimate concentrated load or reaction, P_u, is determined by the allowable load given in Section 3.5.1 of the AISI Specification times the appropriate factor of safety. In this regard, a factor of safety of 1.85 is used for Eqs. 9.3.3-9 through 9.3.3-13, and a factor of safety of 2.0 is used for Eqs. 9.3.3-14 through 9.3.3-17.

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 (See Table III of Ref. 18). By using $\beta_o = 2.5$ and the computed values of V_R for different conditions, the resistance factors, ϕ, were calculated for various conditions as listed in Table C9.3.3.4. For the purpose of simplicity, the value of $\phi = 0.85$ is used in Sections 8.3.5 and 9.3.3.4.1. The values of β corresponding to this value of ϕ are also given 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 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

TABLE C9.3.3.4

Computed ϕ-Factors for Web Crippling

Case	$R_n = P_u$	M_m	V_M	F_m	V_F	P_m	V_P	$\frac{R_m}{R_n}$	V_R	ϕ value for $\beta_o \simeq 2.5$	β for $\phi = 0.85$
(a) Single, Unreinforced Webs											
- End One-Flange Loading (Stiffened Flanges)	Eq. (9.3.3-9)	1.10	0.10	1.00	0.05	1.00	0.12	1.10	0.16	0.86	2.57
- End One-Flange Loading (Unstiffened Flanges)	Eq. (9.3.3-10)	1.10	0.10	1.00	0.05	1.00	0.16	1.10	0.20	0.81	2.33
- Interior One-Flange Loading	Eq. (9.3.3-11)	1.10	0.10	1.00	0.05	0.99	0.11	1.09	0.16	0.86	2.53
- End Two-Flange Loading	Eq. (9.3.3-12)	1.10	0.10	1.00	0.05	0.99	0.09	1.09	0.14	0.88	2.65
- Interior Two-Flange Loading	Eq. (9.3.3-13)	1.10	0.10	1.00	0.05	0.98	0.10	1.08	0.15	0.86	2.55
(b) I-Sections											
- End One-Flange Loading	Eq. (9.3.3-14)	1.10	0.10	1.00	0.05	1.10	0.19	1.21	0.22	0.86	2.54
- Interior One-Flange Loading	Eq. (9.3.3-15)	1.10	0.10	1.00	0.05	0.96	0.13	1.06	0.17	0.82	2.37
- End Two-Flange Loading	Eq. (9.3.3-16)	1.10	0.10	1.00	0.05	1.01	0.13	1.11	0.17	0.86	2.54
- Interior Two-Flange Loading	Eq. (9.3.3-17)	1.10	0.10	1.00	0.05	1.02	0.11	1.12	0.16	0.88	2.63

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 (1985), 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-puckling strength) theory. A total of 44 tests were reported, and the statistical data are as follows (Table 4 in Ref. 5):

$P_m = 1.10$; $V_P = 0.10$

$M_m = 1.10$; $V_F = 0.10$

$F_m = 1.0$; $V_F = 0.05$

$R_m/R_n = 1.21$; $V_R = 0.15$

The reliability index β, as determined from the charts in Ref. 4 for a D_n/L_n of 1/5 is 2.56 for the selected value of $\phi_s = 0.95$.

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*

$P_m = 1.08$; $V_P = 0.11$

$M_m = 1.10$**; $V_M = 0.10$

$F_m = 1.0$; $V_F = 0.05$

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

For the recommended $\phi_s = 0.95$ the reliability index is $\beta = 2.44$ when $D_n/L_n = 1/5$.

b) width-thickness ratios ≥ 25

Number of data: 32***

$P_m = 1.69$; $V_P = 0.18$

M_m** $= 1.0$; $V_M = 0.06$

$F_m = 1.0$; $V_F = 0.05$

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

For the recommended $\phi_s = 0.95$, the reliability index $\beta = 3.43$ when $D_n/L_n = 1/5$. This is considerably above the target of $\beta_o = 2.5$ and a value of $\phi_s = 1.0$ could have been justified. However, $\phi_s = 0.95$ is recommended for the sake of consistency.

Column Strength

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

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* 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 1980 AISI Specification.

The resistance factor $\phi_c = 0.85$ was selected on the basis of 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 Eq. C8.3-2 for a D_n/L_n ratio of 1/5. The target of $\beta_0 = 2.5$ is not entirely satisfied, and different ϕ-factors could have been used for the different cases.

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 1980 Edition of the AISI Specification (Ref. 1), with $\phi_c = 0.85$ (i.e., as recommended for columns) when the limit state is overall member instability, and $\phi_s = 0.95$ (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 0.95 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 for $\phi_c = 0.85$
13	9	Elastic FB	0.97	1.0	1.0	0.04	0.06	0.05	0.97	0.09	2.41
14	10	Inelastic FB, compact	1.09	1.1	1.0	0.05	0.11	0.05	1.20	0.13	3.09
15	18	Inelastic FB, stiffened*	0.97	1.1	1.0	0.16	0.10	0.05	1.07	0.20	2.24
16	31	Inelastic FB, unstiffened	1.53	1.1	1.0	0.24	0.10	0.05	1.68	0.26	3.29
17	12	Inelastic FB, stiffened*	1.10	1.1	1.0	0.08	0.11	0.05	1.21	0.14	3.06
20	8	Elastic TFB	1.11	1.0	1.0	0.11	0.06	0.05	1.11	0.13	2.78
21	30	Inelastic TFB	1.20	1.1	1.0	0.14	0.10	0.05	1.32	0.18	3.11

* 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 bending strength (Section 9.3.1). For laterally unbraced members, the ϕ-factor is 0.90 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 0.95 used in Section 9.6.1 is the same as that used in Section 9.3.1 for bending, while the ϕ factor of 0.85 used in Section 9.6.2 is the same as that used in Section 9.4.1 for axially loaded compression members.

9.7 Inelastic Reserve Capacity of Flexural Members

Section 9.7 of the LRFD criteria is based on Section 3.9 of the 1980 AISI Specification for allowable stress design and the applicable factor of safety. The ϕ factor of 0.95 used in this section is the same as that used in Section 9.3.1 for determining the factored nominal bending strength of flexural 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 for a target reliability index of $\beta_o = 4.0$ and the statistical data summarized in the subsequent discussions.

10.2 Welded Connections

Section 10.2 contains the design provisions for arc-welds (groove welds in butt joints, arc spot welds, arc seam welds, fillet welds, and flare groove welds) and resistance welds. The design equations for the nominal ultimate strength and the ϕ-factor for groove welds in butt joints are the same as that used in the AISC LRFD criteria. (Ref. 12).

For arc spot welds, the ϕ-factor of 0.70 used for determining the shear strength of welds is based on the test data reported in Ref. 19, giving $\beta = 3.76$. The statistical data used for deriving the ϕ-factor are given in the Appendix as follows:

$P_m = 1.47;$ $\quad V_P = 0.22$

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

$F_m = 1.0;$ $\quad V_F = 0.10$

With regard to the type of plate failure governed by Eq. 10.2.1-4 through Eq. 10.2.1-7 in the design criteria, ϕ-factors were derived from the statistical data presented in Table C10.2 (Ref. 18).

For arc seam welds, the shear strength of welds is determined from the same ϕ-factor used for arc spot welds. The derivation of the ϕ-factor for plate tearing is given in Item (B) of the Appendix which is based on the following statistical data:

$P_m = 1.00;$ $\quad V_P = 0.10$

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

$F_m = 1.0;$ $\quad V_F = 0.10$

For the selected value of $\phi = 0.60$, the value of $\beta = 3.81$.

Table C10.2

ϕ-Factors for Plate Failure in Weld Connections

(Tables VII and VIII of Ref. 18)

Case	R_n	M_m	V_M	F_m	V_F	P_m	V_P	$\frac{R_m}{R_n}$	V_R	selected ϕ	β
(a) Arc-Spot Welds											
For $d_a/t \leq 114/\sqrt{F_u}$	Eq. (10.2.1-5)	1.10	0.08	1.00	0.15	1.10	0.17	1.21	0.24	0.60	3.52
For $d_a/t > 240/\sqrt{F_u}$	Eq. (10.2.1-7)	1.10	0.08	1.00	0.15	0.98	0.18	1.08	0.25	0.50	3.64
(b) Fillet Welds											
Longitudinal Loading											
$L/t < 25$	Eq. (10.2.1-14)	1.10	0.08	1.00	0.15	1.01	0.08	1.11	0.19	0.60	3.65
$L/t \geq 25$	Eq. (10.2.1-15)	1.10	0.08	1.00	0.15	0.89	0.09	0.98	0.19	0.60	3.21
Transverse Loading	Eq. (10.2.1-16)	1.10	0.08	1.00	0.15	1.05	0.11	1.16	0.20	0.60	3.72

For fillet welds, the ϕ-factors used for longitudinal loading (Eqs.10.2.1-14 and 10.2.1-15) and transverse loading (Eq. 10.2.1-16) are based on the statistical data presented in Table C 10.2 (Ref. 18).

Similar to the arc spot welds, a ϕ-factor of 0.70 is used for the shear strength of welds.

For flare groove welds, the following statistical data were used to determine the ϕ-factors:

(a) Transverse Flare Bevel Welds (ϕ = 0.55, β = 3.81)

$P_m = 1.04$; $V_P = 0.17$

$M_m = 1.10$; $V_M = 0.10$

$F_m = 1.0$; $V_F = 0.10$

(b) Longitudinal Flare Bevel Welds (ϕ = 0.55, β = 3.56)

$P_m = 0.97$; $V_P = 0.17$

$M_m = 1.10$; $V_M = 0.10$

$F_m = 1.0$; $V_F = 0.10$

See Items (C) and (D) in the Appendix for detailed information.

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

R_n = (2.5) x (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 of 0.65 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, giving β = 3.70.

$P_m = 1.10$; $V_P = 0.05$

$M_m = 1.10$; $V_M = 0.10$

$F_m = 1.00$; $V_F = 0.10$

$R_m/R_n = 1.11$; $V_R = 0.15$

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 Section	
(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(kips)	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	0.60	$3.5F_u dt$	Case 13 in Table C10.3(d)
		< 1.15	0.70	$3.0F_u dt$	Case 14 in Table C10.3(d)
	Single shear and outside sheets of double shear connection	No limit	0.65	$3.0F_u dt$	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(kips)	Statistical Data for Computing ϕ-factor
less than 3/16 inch but greater than or equal to 0.036 in.	Inside sheet of double shear connection	≥ 1.15	0.70	$3.0F_u dt$	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 $\beta_o = 4.0$	Recomm. ϕ	β
Minimum Spacing and Edge Distance	1	1.24	0.16	0.66	0.70	3.75
	2	1.30	0.17	0.67	0.70	3.84
	3	1.03	0.12	0.60	0.70	3.33
	4	1.14	0.13	0.65	0.70	3.67
	5	1.17	0.15	0.63	0.70	3.62
	6	1.39	0.21	0.64	0.70	3.72
Tension Stress on Net Section	8	1.25	0.22	0.56	0.65	3.53
	9	1.05	0.23	0.46	0.60	3.14
	11	1.14	0.17	0.59	0.65	3.63
Bearing Stress on Bolted Connections	13	1.13	0.24	0.48	0.60	3.30
	14	1.07	0.12	0.62	0.70	3.49
	15	1.12	0.22	0.50	0.65	3.17
	16	1.16	0.16	0.61	0.65	3.78
	17(a)	1.11	0.11	0.65	0.70	3.71
	17(b)	1.02	0.11	0.60	0.70	3.36

Table C10.3(e)

Statistical Data for Shear on A307 Bolts

Case No. in Table 6b of Ref. 7	$(\frac{\tau_f}{\sigma_f})_m$	$(\frac{\sigma_f}{F_u})_m$	$\frac{R_m}{R_n}$*	V_R	ϕ value for $\beta_o = 4.0$	Recomm. ϕ	β
12a	0.68	1.28	1.45	0.15	0.78	0.65	4.73
12b	0.60	1.13	1.13	0.14	0.63	0.65	3.85
13a	0.75	1.28	1.60	0.14	0.89	0.65	5.23
13b	0.63	1.18	1.24	0.11	0.73	0.65	4.49
13c	0.76	1.13	1.43	0.11	0.84	0.65	5.09

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

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APPENDIX

Resistance factors, ϕ, are determined to provide a target value of β_o for the load combination ($1.2\ D_n + 1.6\ L_n$) and assuming a $D_n/L_n = 1/5$. For connections, the target value of β_o is 4.0. Equations C8.3-14 and C8.3-15 in the Commentary can be used for this case. Solving for R_m/Q_m in Equation C8.3-15 and setting it equal to Equation C8.3-14 results in the following equality and an equation for ϕ:

$$\frac{R_m}{Q_m} = \left(\frac{1.521}{\phi}\right)\left(\frac{R_m}{R_n}\right) = e^{\beta_o\sqrt{V_R^2 + V_Q^2}}$$

$$\phi = 1.521\ (R_m/R_n)\ e^{-\beta_o\sqrt{V_R^2 + V_Q^2}}$$

where

$$R_m/R_n = M_m F_m P_m$$

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2}$$

$$V_Q = 0.21\ (\text{for } D_n/L_n = 1/5)$$

(A) <u>Resistance Factor for Shear Failure of Arc-Spot Welds</u>

For arc-spot welds, the mean value of the material factor, M_m, is taken as 1.10. The mean value of the fabrication factor, F_m, is assumed to be equal to unity. P_m, the mean value of the professional factor, is found from Table A to be 1.467. Therefore,

$$R_m/R_n = (1.10)(1.0)(1.467) = 1.614$$

The variation of the material properties, V_M, may be taken as 0.10 and the variation of the fabrication factor of the weld spot, V_F, is assumed to be 0.10. V_P is the coefficient of variation of the professional factor and is found from Table A to be 0.217.

Therefore,

$$V_R = \sqrt{0.10^2 + 0.10^2 + 0.217^2} = 0.259$$

These assumed values are the same values used for the spot resistance welds in Article II. 5.3 of Ref. 6.

Therefore,

$$\phi = (1.521)(1.614)\ e^{-4\sqrt{0.259^2 + 0.21^2}} = 0.65, \text{ use } \phi = 0.70.$$

(B) Resistance Factor for Plate Tearing of Arc Seam Welds

For plate tearing failure of arc seam welds the following values were assumed:

$M_m = 1.10 \qquad V_M = 0.10$

$F_m = 1.0 \qquad V_F = 0.10$

The following values were obtained from Table B:

$P_m = 1.004 \qquad V_P = 0.095$

Therefore,

$$R_m/R_n = (1.10)(1.0)(1.004) = 1.104$$

$$V_R = \sqrt{0.10^2 + 0.10^2 + 0.095^2} = 0.170$$

$$\phi = 0.57, \text{ use } \phi = 0.60.$$

(C) Resistance Factor for Plate Tearing of Transverse Flare Bevel Welds

For plate tearing failure of transverse flare bevel welds, the following values were assumed:

$M_m = 1.10 \qquad V_M = 0.10$

$F_m = 1.0 \qquad V_F = 0.10$

The following valves were obtained from Table C:

$P_m = 1.04 \qquad V_P = 0.165$

Therefore,

$$R_m/R_n = (1.10)(1.0)(1.04) = 1.144$$

$$V_R = \sqrt{0.10^2 + 0.10^2 + 0.165^2} = 0.217$$

$$\phi = 0.52, \text{ use } \phi = 0.55.$$

(D) <u>Resistance Factor for Plate Tearing of Longitudinal Flare Bevel Welds</u>

For plate tearing failure of longitudinal flare bevel welds, the following values were assumed:

$M_m = 1.10$ $\quad\quad$ $V_M = 0.10$

$F_m = 1.0$ $\quad\quad$ $V_F = 0.10$

The following values were obtained from Table D:

$P_m = 0.969$ $\quad\quad$ $V_P = 0.169$

Therefore,

$$R_m/R_n = (1.10)(1.0)(0.969) = 1.066$$

$$V_R = \sqrt{0.10^2 + 0.10^2 + 0.169^2} = 0.220$$

$$\phi = 0.48, \text{ use } \phi = 0.55 \text{ (i.e., } \beta = 3.56).$$

Table A

Comparison of Tested and Predicted Loads of Arc Spot Welds Failing in Shearing of the Weld

Specimen	$(P_u)_t$ (kips)	$(P_u)_p$ (kips)	$(P_u)_t/(P_u)_p$
A A/B 12/7 D(B-C)1	20.60	12.94	1.592
A A/B 12/7 D(B-C)2	24.80	13.72	1.808
A A/B 12/7 D(B-C)3	20.30	13.63	1.489
A A/B 12/7 D(F-C)1	24.10	13.72	1.757
A A/B 12/7 D(F-C)2	24.90	14.91	1.670
A A/B 12/7 D(E-C)1	24.10	14.91	1.616
A A/B 12/7 D(E-C)2	24.10	16.06	1.501
A A/B 12/7 D(AA-C)3	14.00	12.94	1.082
B A/B 14/7 D(A-C)1	17.20	12.82	1.342
B A/B 14/7 D(A-C)2	20.90	15.50	1.348
B A/B 14/7 D(D-C)1	16.10	13.97	1.152
B A/B 14/7 D(D-C)2	11.80	10.65	1.108
B A/B 14/7 D(F-C)1	14.80	12.44	1.190
B A/B 14/7 D(F-C)2	16.50	11.07	1.491
B A/B 14/7 D(D-E)1	38.90	19.18	2.028
B A/B 14/7 D(D-E)2	39.40	19.65	2.005
B A/B 18/7 C(D-AA)1	18.90	8.38	2.255
B A/B 18/7 C(D-AA)2	12.60	8.07	1.561
B A/B 14/7 D(E-D)1	18.80	16.10	1.168
B A/B 14/7 D(E-D)2	22.50	15.58	1.444
A A/B 12/7 C(E-AA)2	10.70	11.12	0.962
A A/B 10/7 D(E-CC)1	26.10	15.28	1.708
A A/B 10/7 D(E-CC)2	20.90	15.66	1.335
A A/B 10/7 D(E-E)1	34.50	19.63	1.758
A A/B 10/7 D(E-E)2	28.30	24.54	1.153
B A/B 18/7 D1S	12.70	9.37	1.355
B A/B 18/7 D2S	16.20	11.05	1.466
B A/B 18/7 D3S	15.40	10.70	1.439
B A/B 18/7 D4S	11.70	7.82	1.496
B A/B 18/7 D5S	8.60	11.05	0.778
B A/B 12/7 D2	6.00	3.74	1.604
B A/B 12/7 D3	5.00	3.94	1.269
Mean		P_m =	1.467
Coefficient of variation		V_p =	0.217

Note: The data selected were predicted in weld shear and are listed in Table 5 of Ref. 19.

Table B

Comparison of Tested and Predicted Loads of Arc Seam Welds

(Table 6 of Ref. 19)

Specimen	$(P_u)_t$ (kips)	$(P_u)_p$ (kips)	$(P_u)_t/(P_u)_p$
Single Sheet Oblong Puddle Welds			
A A/B 18/7 X1	15.60	14.83	1.05
A A/B 18/7 X2	15.50	14.90	1.04
A A/B 18/7 X3	15.00	13.89	1.08
A A/B 18/7 Y1	13.10	11.90	1.10
A A/B 18/7 Y3	10.90	12.07	0.90
A A/B 22/7 X1	7.61	7.41	1.03
A A/B 22/7 X2	7.50	7.52	1.00
A A/B 22/7 X3	7.06	6.67	1.06
A A/B 22/7 Y1	3.90	5.57	0.70
A A/B 22/7 Y2	6.04	5.14	1.18
A A/B 22/7 Y3	4.76	5.49	0.87
Double Sheet Oblong Puddle Welds			
B A/B 18/7 X1	30.20	30.55	0.99
B A/B 18/7 X2	31.10	31.61	0.98
B A/B 18/7 X3	31.00	30.97	1.00
B A/B 18/7 Y1	23.90	24.99	0.95
B A/B 18/7 Y2	25.70	24.81	1.04
B A/B 18/7 Y3	24.90	24.88	1.00
B A/B 22/7 X1	15.60	14.34	1.09
B A/B 22/7 X2	15.10	14.52	1.04
B A/B 22/7 X3	15.40	14.63	1.05
B A/B 22/7 Y1	12.10	12.52	0.97
B A/B 22/7 Y2	12.40	11.70	1.06
B A/B 22/7 Y3	11.20	12.18	0.92
Mean			$P_m = 1.004$
Coefficient of Variation			$V_P = 0.095$

Table C

Comparison of the Tested and Predicted Loads of Flare Groove Welds

(Table 3 of Ref. 19)

Specimen	$(P_u)_t$ (kips)	$(P_u)_p$ (kips)	$(P_u)_t/(P_u)_p$
Shop Welded Specimens			
E A/B 18/7 F1	7.04	7.88	0.89
E A/B 18/7 F2	9.58	7.82	1.22
E A/B 18/7 F3	7.82	7.98	0.98
E A/B 18/7 C1	2.66	4.94	0.53
E A/B 18/7 C2	3.70	4.58	0.80
E A/B 18/7 C3	4.70	4.94	0.95
E A/B 18/7 C4	2.84	4.48	0.63
E A/B 18/7 L1	16.20	16.11	1.01
E A/B 18/7 L2	16.60	15.81	1.05
E A/B 18/7 L3	16.60	15.55	1.06
E A/B 18/7 P1	20.80	20.28	1.02
E A/B 18/7 P2	21.50	21.33	1.01
E A/B 18/7 P3	20.50	20.96	0.98
E A/B 12/7 C1	9.50	8.36	1.14
E A/B 12/7 C2	8.94	7.48	1.19
E A/B 12/7 C3	9.44	10 23	0.92
E A/B 12/7 F1	13.00	13.29	0.98
E A/B 12/7 F2	10.80	13.51	0.80
E A/B 12/7 F3	14.56	13.69	1.06
E A/B 12/7 F4	14.30	13.29	1.07
E A/B 12/7 L1	27.50	26.49	1.04
E A/B 12/7 L2	27.50	26.40	1.04
E A/B 12/7 L3	27.50	26.58	1.03
E A/B 12/7 P1	33.10	35.20	0.94
E A/B 12/7 P2	32.90	35.11	0.93
E A/B 12/7 P3	33.30	34.78	0.96
Field Welded Specimens			
E A/B 12/7 P1	35.20	34.63	1.02
E A/B 12/7 P2	35.70	34.81	1.02
E A/B 12/7 P3	35.80	34.90	1.03
E A/B 12/7 C1	13.96	9.71	1.42
E A/B 12/7 C2	12.62	9.89	1.28
E A/B 12/7 C3	13.06	10.34	1.26
E A/B 12/7 C4	12.58	10.97	1.15
E A/B 12/7 C5	12.62	10.88	1.16

Table C (continued)

Specimen	$(P_u)_t$ (kips)	$(P_u)_p$ (kips)	$(P_u)_t/(P_u)_p$
E A/B 18/7 P1	19.25	19.88	0.97
E A/B 18/7 P2	20.00	20.24	0.99
E A/B 18/7 P3	19.90	19.98	1.00
E A/B 18/7 C1	6.84	5.38	1.26
E A/B 18/7 C2	6.82	5.49	1.25
E A/B 18/7 C3	6.40	5.64	1.14
E A/B 18/7 C4	7.08	5.28	1.33
E A/B 18/7 C5	7.26	5.95	1.22
Mean			$P_m = 1.040$
Coefficient of Variation			$V_P = 0.165$

Table D

Comparison of Tested and Predicted Loads of Longitudinal Flare Bevel Welds Failing in Tearing along Weld Contour

Specimen	$(P_u)_t$ (kips)	$(P_u)_p$ (kips)	$(P_u)_t/(P_u)_p$
Shop Welded Specimens			
D A/B 18/7 C1	17.60	15.88	1.108
D A/B 18/7 C2	19.40	15.88	1.222
D A/B 18/7 C3	21.20	17.33	1.223
D A/B 12/7 C1	28.50	30.03	0.949
D A/B 12/7 C2	25.20	28.71	0.878
D A/B 12/7 C3	29.00	29.37	0.987
D A/B 12/7 F6	35.50	46.86	0.758
Field Welded Specimens			
D A/B 12/7 C1	30.10	35.75	0.842
D A/B 12/7 C2	31.00	36.92	0.840
D A/B 12/7 C3	31.60	35.75	0.884
Mean			$P_m = 0.969$
Coefficient of Variation			$V_P = 0.169$

Note: The data selected were predicted in tearing along the weld contour and are listed in Table 4 of Ref. 19.