Load and Resistance Factor Design Specification for Structural Steel Buildings

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Notes

PREFACE

The AISC *Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings* is the first of a new generation of specifications based on reliability theory. As have all AISC Specifications, this LRFD Specification has been based upon past successful usage, advances in the state of knowledge and changes in design practice. The LRFD Specification has been developed to provide a uniform practice in the design of steel-framed buildings. The intention is to provide design criteria for routine use and not to cover infrequently encountered problems which occur in the full range of structural design. Providing definitive provisions to cover all complex cases would make the LRFD Specification useless for routine designs.

The LRFD Specification is the result of the deliberations of a committee of structural engineers with wide experience and high professional standing, representing a wide geographical distribution throughout the U.S. The committee includes approximately equal numbers of engineers in private practice, engineers involved in research and teaching and engineers employed by steel fabricating companies.

In order to avoid reference to proprietary steels which may have limited availability, only those steels which can be identified by ASTM specifications are approved under this Specification. However, some steels covered by ASTM specifications, but subject to more costly manufacturing and inspection techniques than deemed essential for structures covered by this Specification, are not listed, even though they may provide all of the necessary characteristics of less expensive steels which are listed. Approval of such steels is left to the owner's representative.

The Appendices to this Specification are considered to be an integral part of the Specification.

As used throughout the LRFD Specification, the term structural steel refers exclusively to those items enumerated in Sect. 2 of the AISC *Code of Standard Practice for Steel Buildings and Bridges*; nothing herein contained is intended as a specification for design of items not specifically enumerated in that Code, such as skylights, fire escapes, etc. For the design of cold-formed steel structural members whose profiles contain rounded corners and slender flat elements, the provisions of the American Iron and Steel Institute *Specification for the Design of Cold-Formed Steel Structural Members* are recommended.

A Commentary has been prepared to provide background for the Specification provisions and the user is encouraged to consult it.

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NOMENCLATURE

The section number in parentheses after the definition of a symbol refers to the section where the symbol is first defined.

A Cross-sectional area, in.^{2} (F1.3)
A_B Loaded area of concrete, in.^{2} (I2.4)
A_b Nominal body area of a fastener, in.^{2} (J3.4)
A_b Area of an upset rod based upon the major diameter of its threads (J3.3)
A_c Area of concrete, in.^{2} (I2.2)
A_c Area of concrete slab within effective width (I5.2)
A_e Effective net area, in.^{2} (B3)
A_f Area of flange, in. (Appendix F4)
A_g Gross area, in.^{2} (B1)
A_n Net area, in.^{2} (B2)
A_{ns} Net area subject to shear, in.^{2} (J4)
A_{pb} Projected bearing area, in.^{2} (D3, J8.1)
A_r Area of reinforcing bars, in.^{2} (I2.2)
A_s Area of steel cross section, in.^{2} (I2.2, I5.2)
A_{sc} Cross-sectional area of stud shear connector, in.^{2} (I5.3)
A_{sf} Shear area on the failure path, in.^{2} (D3)
A_w Web area, in.^{2} (F2.1)
A_1 Area of steel bearing on a concrete support, in.^{2} (J9)
A_2 Total cross-sectional area of a concrete support, in.^{2} (J9)
B Factor for bending stress in web-tapered members, in., defined by Formulas A-F4-7 through A-F4-10 (Appendix F4)
B_1, B_2 Factors used in determining M_u for combined bending and axial forces when first order analysis is employed (H1)
C_b Bending coefficient dependent upon moment gradient (F1.3)
C_m Coefficient applied to bending term in interaction formula for prismatic members and dependent upon column curvature caused by applied moments (H1)
C'_m Coefficient applied to bending term in interaction formula for tapered members and dependent upon axial stress at the small end of the member (Appendix F4)
C_p Ponding flexibility coefficient for primary member in a flat roof (K2)
C_{PG} Plate girder coefficient (Appendix G2)
C_s Ponding flexibility coefficient for secondary member in a flat roof (K2)
C_v Ratio of "critical" web stress, according to linear buckling theory, to the shear yield stress of web material (Appendix G4)
C_w Warping constant, in.^{6} (F1.3)
D Outside diameter of circular hollow section, in. (Appendix B5.3)
D Dead load due to the self-weight of the structural and permanent elements on the structure (A4.1)

D Factor used in Formula A-G4-2, dependent on the type of transverse stiffeners used in a plate girder (Appendix G4)

E Modulus of elasticity of steel (29,000 ksi) (E2)

E Earthquake load (A4.1)

E_c Modulus of elasticity of concrete, ksi (I2.2)

E_m Modified modulus of elasticity, ksi (I2.2)

F_{BM} Nominal strength of the base material to be welded, ksi (J2.4)

F_{EXX} Classification strength of weld metal, ksi (A3.5 and J2.4)

F_a Axial design strength, ksi (A5.1)

$F_{b\gamma}$ Flexural stress for tapered members defined by Formulas A-F4-4 and A-F4-5 (Appendix F4)

F_{cr} Critical stress, ksi (E2)

F_e Elastic buckling stress, ksi (Appendix E3)

F_{ex} Elastic flexural buckling stress about the major axis, ksi (Appendix E3)

F_{ey} Elastic flexural buckling stress about the minor axis, ksi (Appendix E3)

F_{ez} Elastic torsional buckling stress, ksi (Appendix E3)

F_{my} Modified yield stress for composite columns, ksi (I2.2)

F_n Nominal shear rupture strength, ksi (J4)

F_r Compressive residual stress in flange, ksi (F1.3)

$F_{s\gamma}$ Stress for tapered members defined by Formula A-F4-6 (Appendix F4)

F_u Specified minimum tensile strength, ksi (D1)

F_w Nominal strength of the weld electrode material, ksi (J2.4)

$F_{w\gamma}$ Stress for tapered members defined by Formula A-F4-7, ksi (Appendix F4)

F_y Specified minimum yield stress of the type of steel being used, ksi. As used in this Specification, "yield stress" denotes either the specified minimum yield point (for those steels that have a yield point) or specified yield strength (for those steels that do not have a yield point)

F_{yf} Specified minimum yield stress of the flange, ksi (B5.1)

F_{ym} Yield stress obtained from mill test reports or from physical tests, ksi (N2)

F_{yr} Specified minimum yield stress of reinforcing bars, ksi (I2.2)

F_{ys} Static yield stress, ksi (N2)

F_{yst} Specified minimum yield stress of the stiffener material, ksi (Appendix G4)

F_{yw} Specified minimum yield stress of the web, ksi (F2.2)

G Shear modulus of elasticity of steel, ksi ($G = 11{,}200$) (F1.3)

H Horizontal force, kips (H1.2)

H_s Length of stud connector after welding, in. (I3.5)

I Moment of inertia, in.^{4}

I_d Moment of inertia of the steel deck supported on secondary members, in.^{4} (K2)

I_p Moment of inertia of primary members, in.^{4} (K2)

I_s Moment of inertia of secondary members, in.^{4} (K2)

I_{st} Moment of inertia of a transverse stiffener, in.^{4} (Appendix G4)

J Torsional constant for a section, in.^{4} (F1.3)

K Effective length factor for prismatic member (C2)

K_s Slip coefficient (J3)

K_z Effective length factor for torsional buckling (Appendix E3)

K_γ Effective length factor for a tapered member (Appendix F4.3)

L Unbraced length of member measured between the center of gravity of the bracing members, in. (E2)

L Story height, in. (H1.2)

L Distance in line of force from center of a standard or oversized hole or from the center of the end of a slotted hole to an edge of a connected part, in. (J3.6)

L Live load due to occupancy (A4.1)

L_b Laterally unbraced length; length between points which are either braced against lateral displacement of compression flange or braced against twist of the cross section in. (F1.1)

L_c Length of channel shear connector, in. (I5.4)

L_p Limiting laterally unbraced length for full plastic bending capacity, uniform moment case ($C_b = 1.0$) in. (F1.3)

L_p Column spacing in direction of girder, in. (K2)

L_{pd} Limiting laterally unbraced length for plastic analysis, in. (F1.1)

L_r Limiting laterally unbraced length for inelastic lateral-torsional buckling, in. (F1.3)

L_r Roof live load (A4.1)

L_s Column spacing perpendicular to direction of girder, in. (K2)

M_1 Smaller moment at end of unbraced length of beam or beam-column, kip-in. (F1.1)

M_2 Larger moment at end of unbraced length of beam-column, kip-in. (F1.3)

M_{cr} Elastic buckling moment, kip-in. (F1.4)

$M_{\ell t}$ Required flexural strength in member due to lateral frame translation including second-order effects, kip-in. (H1)

M_n Nominal flexural strength, kip-in. (F1.2)

M'_{nx}, M_{ny} Flexural strength defined in Formulas A-H3-7 and A-H-3-8 for use in alternate interaction equations for combined bending and axial force, kip-in. (Appendix H3)

M_{nt} Required flexural strength in member assuming there is no lateral translation of the frame, kip-in. (H1)

M_p Plastic bending moment, kip-in. (F1.1)

M'_p Moment defined in Formulas A-H3-5 and A-H3-6, for use in alternate interaction equations for combined bending and axial force, kip-in. (Appendix H3)

M_r Limiting buckling moment, M_{cr}, when $\lambda = \lambda_r$ and $C_b = 1.0$, kip-in. (F1.3)

M_u Required flexural strength, kip-in. (H1)

N Length of bearing, in. (K1.3)

N_r Number of stud connectors in one rib at a beam intersection (I3.5)

P Force transmitted by one fastener to the critical connected part, kips (J3.9)

P_e Euler buckling strength, kips (H1)

P_e Elastic buckling load, kips (I4)

P_n Nominal axial strength (tension or compression), kips (D1)

P_p Bearing load on concrete, kips (J9)

P_u Required axial strength (tension or compression), kips (H1)

P_y Yield strength, kips (B5.1)

Q Full reduction factor for slender compression elements (Appendix E3)

Q_a Reduction factor for slender stiffened compression elements (Appendix B5)

Q_n Nominal strength of one stud shear connector, kips (I5)

Q_s Reduction factor for slender unstiffened compression elements (Appendix B5.3)

R Nominal load due to initial rainwater or ice exclusive of the ponding contribution (A4.1)

R_{PG} Plate girder bending strength reduction factor (Appendix G)

R_e Hybrid girder factor (Appendix F1.7)

R_n Nominal resistance (A5.3)

R_v Web shear strength, kips (K1.7)

S Elastic section modulus, in.^{3}

S Spacing of secondary members, in. (K2)

S Snow load (A4.1)

S_x Elastic section modulus about major axis, in.^{3} (F1.3)

S'_x Elastic section modulus of larger end of tapered member about its major axis, in.^{3} (Appendix F4)

$(S_x)_{eff}$ Effective section modulus about major axis, in.^{3} (Appendix F1.7)

S_{xt}, S_{xc} Elastic section modulus referred to tension and compression flanges, respectively, in.^{3} (Appendix G2)

T Required tension force, kips (J3.5)

T_b Specified pretension load in high-strength bolt, kips (J3.5)

U Reduction coefficient, used in calculating effective net area (B3)

U_p, U_s Ponding stress index for primary and secondary members (Appendix K2)

V Shear force, kips (J10.2)

V_n Nominal shear strength, kips (F2.2)

V_u Required shear strength, kips (Appendix G4)

W Wind load (A4.1)

X_1 Beam buckling factor defined by Formula F1-8 (F1.3)

X_2 Beam buckling factor defined by Formula F1-9 (F1.3)

Z Plastic section modulus, in.^{3}

a Clear distance between transverse stiffeners, in. (F2.2)

a Distance between connectors in a built-up member, in. (E4)

a Shortest distance from edge of pin hole to edge of member measured parallel to direction of force, in. (D3)

a_r Ratio of web area to compression flange area (Appendix G2)

b Compression element width (Table B5.1)

b_e Reduced effective width for slender compression elements, in. (Appendix B5.3)

b_{eff} Effective edge distance, in. (D3)

b_f Flange width, in. (K1.5)

c_1, c_2, c_3 Numerical coefficients (I2.2)

d	Nominal fastener diameter, in. (J3.6)
d	Overall depth of member, in. (F2.1)
d	Pin diameter, in. (D3)
d	Roller diameter, in. (J8.2)
d_L	Depth at larger end of unbraced tapered segment, in. (Appendix F4)
d_c	Web depth clear of fillets, in. (K1.5)
d_h	Diameter of a standard size hole, in. (J3.9)
d_o	Depth at smaller end of unbraced tapered segment, in. (Appendix F4)
e	Base of natural logarithm = 2.71828 . . .
f	Computed compressive stress in the stiffened element, ksi (Appendix B5.3)
f_a	Computed axial stress in column, ksi (A5.1)
f_{b1}	Smallest computed bending stress at one end of a tapered segment, ksi (Appendix F4)
f_{b2}	Largest computed bending stress at one end of a tapered segment, ksi (Appendix F4)
f'_c	Specified compressive strength of concrete, ksi (I2.2)
f_o	Stress due to $1.2D + 1.2R$, ksi (Appendix K2)
f_t	Computed tension stress in bolts or rivets, ksi (J3.4)
f_{un}	Required normal stress, ksi (H2)
f_{uv}	Required shear stress, ksi (H2)
f_v	Computed shear stress in bolts or rivets, ksi (J3.4)
g	Transverse c.-to-c. spacing (gage) between fastener gage lines, in. (B2)
g	Acceleration due to gravity = 32.2 ft/sec.^{2} (A4.3)
h	Clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, in. (B5.1)
h_c	Assumed web depth for stability, in. (B5.1)
h_r	Nominal rib height, in. (I3.5)
h_s	Factor used in Formula A-F4-6 for web-tapered members (Appendix F4.3)
h_w	Factor used in Formula A-F4-7 for web-tapered members (Appendix F4.3)
j	Factor defined by Formula F3-1 for minimum moment of inertia for a transverse stiffener (F3)
k	Web plate buckling coefficient (F2.2)
k	Distance from outer face of flange to web toe of fillet, in. (K1.3)
ℓ	Largest laterally unbraced length along either flange at the point of load, in. (K1.5)
ℓ	Length of bearing, in. (J8.2)
m	Ratio of web to flange yield stress or critical stress in hybrid beams (Appendix G2)
r	Governing radius of gyration, in. (E2)
r_i	Minimum radius of gyration of individual component in a built-up member, in. (E4)
r_T	Radius of gyration of compression flange plus one third of the compression portion of the web taken about an axis in the plane of the web, in. (Appendix F1.7)

r_{To} Radius of gyration, r_T, for the smaller end of a tapered member, in. (Appendix F4.3)

r_m Radius of gyration of the steel shape, pipe or tubing in composite columns. For steel shapes it may not be less than 0.3 times the overall thickness of the composite section, in. (I2)

$\bar{r}_o$ Polar radius of gyration about the shear center, in. (Appendix E3)

r_{ox}, r_{oy} Radius of gyration about x and y axes at the smaller end of a tapered member, respectively, in. (Appendix F4.3)

r_x, r_y Radius of gyration about x and y axes, respectively, in. (F1.1, E3)

s Longitudinal c.-to-c. spacing (pitch) of any two consecutive holes, in. (B2)

t Thickness of connected part, in. (J3.6)

t Thickness of the critical part, in. (J3.9)

t_f Flange thickness, in. (B5.1)

t_f Flange thickness of channel shear connector, in. (I5.4)

t_w Web thickness of channel shear connector, in. (I5.4)

t_w Web thickness, in. (F2.1)

w Plate width; distance between welds, in. (B3)

w Unit weight of concrete, lbs./cu. ft. (I2)

w_r Average width of concrete rib or haunch, in. (I3.5)

x Subscript relating symbol to strong axis bending

x_o, y_o Coordinates of the shear center with respect to the centroid, in. (Appendix E3)

y Subscript relating symbol to weak axis bending

z Distance from the smaller end of tapered member used in Formula A-F4-1 for the variation in depth, in. (Appendix F4)

Δ_{oh} Translation deflection of the story under consideration, in. (H1)

γ Depth tapering ratio (Appendix F4)
Subscript for tapered members (Appendix F4)

ζ Exponent for alternate beam-column interaction equation (Appendix H3)

η Exponent for alternate beam-column interaction equation (Appendix H3)

λ_c Column slenderness parameter (E2)

λ_e Equivalent slenderness parameter (Appendix E3)

λ_{eff} Effective slenderness ratio defined by Formula A-F4-2 (Appendix F4)

λ_p Limiting slenderness parameter for compact element (B5.1)

λ_r Limiting slenderness parameter for noncompact element (B5.1)

μ Coefficient of friction (J10.2)

ϕ Resistance factor (A5.3)

ϕ_b Resistance factor for flexure (B1)

ϕ_c Resistance factor for compression (E2)

ϕ_c Resistance factor for axially loaded composite columns (I2.2)

ϕ_{sf} Resistance factor for shear on the failure path (D3)

ϕ_t Resistance factor for tension (D1)

ϕ_v Resistance factor for shear (F2.2)

CHAPTER A.
GENERAL PROVISIONS

A1. SCOPE

This *Load and Resistance Factor Design Specification for Structural Steel Buildings* is intended as an alternate to the currently approved *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings* of the American Institute of Steel Construction.

A2. LIMITS OF APPLICABILITY

1. Structural Steel Defined

As used in this Specification, the term *structural steel* refers to the steel elements of the structural steel frame essential to the support of the design loads. Such elements are generally enumerated in Sect. 2.1 of the AISC *Code of Standard Practice for Steel Buildings and Bridges*. For the design of cold-formed steel structural members, whose profiles contain rounded corners and slender flat elements, the provisions of the American Iron and Steel Institute *Specification for the Design of Cold-Formed Steel Structural Members* are recommended.

2. Types of Construction

Two basic types of construction and associated design assumptions are permissible under the conditions stated herein, and each will govern in a specific manner the size of members and the types and strength of their connections. Both types must comply with the stability requirements of Sect. B4.

Type FR (fully restrained), commonly designated as "rigid-frame" (continuous frame), assumes that beam-to-column connections have sufficient rigidity to hold the original angles between intersecting members virtually unchanged.

Type PR (partially restrained) assumes that the connections of beams and girders possess an insufficient rigidity to hold the original angles between intersecting members virtually unchanged.

The design of all connections shall be consistent with assumptions as to type of construction called for on the design drawings.

Type FR construction is unconditionally permitted under the LRFD Specification.

The use of Type PR construction under this Specification depends on the evidence of predictable proportion of full end restraint. Where the connection restraint is ignored, commonly designated "simple framing," it is assumed that under gravity loads the ends of the beams and girders are connected for shear only and are free to rotate. For "simple framing" the following requirements apply:

a. The connections and connected members must be adequate to carry the factored gravity loads as "simple beams."
b. The connections and connected members must be adequate to resist the factored lateral loads.
c. The connections must have sufficient inelastic rotation capacity to avoid overload of fasteners or welds under combined factored gravity and lateral loading.

When the rotational restraint of the connections is used in the design of the connected members or for the stability of the structure as a whole, the capacity of the connection for such restraint must be established by analytical or empirical means.

Type PR construction may necessitate some inelastic, but self-limiting, deformation of a structural steel part.

A3. MATERIAL

1. Structural Steel

Material conforming to one of the following standard specifications is approved for use under this Specification:

Structural Steel, ASTM A36
Welded and Seamless Steel Pipe, ASTM A53, Gr. B
High-strength Low-alloy Structural Steel, ASTM A242
High-strength Low-alloy Structural Manganese-vanadium Steel, ASTM A441
Cold-formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes, ASTM A500
Hot-formed Welded and Seamless Carbon Steel Structural Tubing, ASTM A501
High Yield-strength Quenched and Tempered Alloy Steel Plate, Suitable for Welding, ASTM A514
Structural Steel with 42,000 psi Minimum Yield Point, ASTM A529
Hot-rolled Carbon Steel Sheets and Strip, Structural Quality, ASTM A570, Gr. 40, 45 and 50
High-strength Low-alloy Columbium-vanadium Steels of Structural Quality, ASTM A572
High-strength Low-alloy Structural Steel with 50,000 psi Minimum Yield Point to 4 in. Thick, ASTM A588

Steel Sheet and Strip, Hot-rolled and Cold-rolled, High-strength, Low-alloy, with Improved Corrosion Resistance, ASTM A606
Steel Sheet and Strip, Hot-rolled and Cold-rolled, High-strength, Low-alloy, Columbium and/or Vanadium, ASTM A607
Hot-formed Welded and Seamless High-strength Low-alloy Structural Tubing, ASTM A618

Certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6 or A568, as applicable, shall constitute sufficient evidence of conformity with one of the above ASTM standards. If requested, the fabricator shall provide an affidavit stating that the structural steel furnished meets the requirements of the grade specified.

Unidentified steel, if surface conditions are acceptable according to criteria contained in ASTM A6, may be used for unimportant members or details, where the precise physical properties and weldability of the steel would not affect the strength of the structure.

2. Steel Castings and Forgings

Cast steel shall conform to one of the following standard specifications:

Mild-to-medium-strength Carbon-steel Castings for General Applications, ASTM A27, Gr. 65-35
High-strength Steel Castings for Structural Purposes, ASTM A148, Gr. 80-50

Steel forgings shall conform to the following standard specification:

Steel Forgings Carbon and Alloy for General Industrial Use, ASTM A668

Certified test reports shall constitute sufficient evidence of conformity with the standards.

3. Bolts

Steel bolts shall conform to one of the following standard specifications:

Low-carbon Steel Externally and Internally Threaded Standard Fasteners, ASTM A307
High-strength Bolts for Structural Steel Joints, ASTM A325
Quenched and Tempered Steel Bolts and Studs, ASTM A449
Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints, ASTM A490
Carbon and Alloy Steel Nuts, ASTM A563
Hardened Steel Washers, ASTM F436

A449 bolts may be used only in connections requiring bolt diameters greater than 1½ in. and shall not be used in slip-critical connections. A449 material is acceptable for high-strength anchor bolts and threaded rods of any diameter.

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

4. Anchor Bolts and Threaded Rods

Anchor bolt and threaded rod steel shall conform to one of the following standard specifications:

Structural Steel, ASTM A36
Quenched and Tempered Alloy Steel Bolts, Studs and Other Externally Threaded Fasteners, ASTM A354
High-strength Low-alloy Structural Steel with 50,000 psi Minimum Yield Point to 4 in. Thick, ASTM A588.
High-strength Nonheaded Steel Bolts and Studs, ASTM A687.
Threads on bolts and rods shall conform to Unified Standard Series of latest edition of ANSI B18.1 and shall have Class 2A tolerances.

Steel bolts conforming to other provisions of Sect. A3 may be used as anchor bolts.

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

5. Filler Metal and Flux for Welding

Welding electrodes and fluxes shall conform to one of the following specifications of the American Welding Society.*

Specification for Covered Carbon Steel Arc Welding Electrodes, AWS A5.1
Specification for Low-alloy Steel Covered Arc Welding Electrodes, AWS A5.5
Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding, AWS A5.17
Specification for Carbon Steel Filler Metals for Gas Shielded Arc Welding, AWS A5.18
Specification for Carbon Steel Electrodes for Flux Cored Arc Welding, AWS A5.20
Specification for Low-alloy Steel Electrodes and Fluxes for Submerged-arc Welding, AWS A5.23
Specification for Low-alloy Steel Filler Metals for Gas-shielded Arc Welding, AWS A5.28
Specification for Low-alloy Steel Electrodes for Flux-cored Arc Welding, AWS A5.29

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

6. Stud Shear Connectors

Steel stud shear connectors shall conform to the requirements of *Structural Welding Code*—Steel, AWS D1.1.

Manufacturer's certification shall constitute sufficient evidence of conformity with the code.

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*Approval of these welding electrode specifications is given without regard to weld metal notch toughness requirements, which are generally not critical for building construction.

A4. LOADS AND LOAD COMBINATIONS

The nominal loads shall be the minimum design loads stipulated by the applicable code under which the structure is designed or dictated by the conditions involved. In the absence of a code, the loads and load combinations shall be those stipulated in the American National Standard *Minimum Design Loads for Buildings and Other Structures*, ANSI A58.1. For design purposes, the loads stipulated by the applicable code shall be taken as nominal loads. For ease of reference, the more common ANSI load combinations are listed below.

1. Loads, Load Factors and Load Combinations

The following nominal loads are to be considered:

- D : dead load due to the weight of the structural elements and the permanent features on the structure
- L : live load due to occupancy and moveable equipment
- L_r : roof live load
- W : wind load
- S : snow load
- E : earthquake load
- R : load due to initial rainwater or ice exclusive of the ponding contribution

The required strength of the structure and its elements must be determined from the appropriate critical combination of factored loads. The most critical effect may occur when one or more loads are not acting. The following load combinations and the corresponding load factors shall be investigated:

$$1.4\,D \tag{A4-1}$$
$$1.2\,D + 1.6\,L + 0.5\,(L_r \text{ or } S \text{ or } R) \tag{A4-2}$$
$$1.2\,D + 1.6\,(L_r \text{ or } S \text{ or } R) + (0.5\,L \text{ or } 0.8\,W) \tag{A4-3}$$
$$1.2\,D + 1.3\,W + 0.5\,L + 0.5\,(L_r \text{ or } S \text{ or } R) \tag{A4-4}$$
$$1.2\,D + 1.5\,E + (0.5\,L \text{ or } 0.2\,S) \tag{A4-5}$$
$$0.9\,D - (1.3\,W \text{ or } 1.5\,E) \tag{A4-6}$$

Exception: The load factor on L in combinations A4-3, A4-4 and A4-5 shall equal 1.0 for garages, areas occupied as places of public assembly and all areas where the live load is greater than 100 psf.

2. Impact

For structures carrying live loads which induce impact, the assumed nominal live load shall be increased to provide for this impact in Formulas A4-2 and A4-3.

If not otherwise specified, the increase shall be:

For supports of elevators and elevator machinery 100%
For supports of light machinery, shaft or motor driven, not less than 20%
For supports of reciprocating machinery or power driven units, not less than 50%
For hangers supporting floors and balconies 33%
For cab-operated traveling crane support girders and their connections 25%
For pendant-operated traveling crane support girders and their connections 10%

3. Crane Runway Horizontal Forces

The nominal lateral force on crane runways to provide for the effect of moving crane trolleys shall be a minimum of 20% of the sum of weights of the lifted load and of the crane trolley, but exclusive of other parts of the crane. The force shall be assumed to be applied at the top of the rails, acting in either direction normal to the runway rails, and shall be distributed with due regard for lateral stiffness of the structure supporting the rails.

The longitudinal force shall be a minimum of 10% of the maximum wheel loads of the crane applied at the top of the rail, unless otherwise specified.

A5. DESIGN BASIS

1. Required Strength at Factored Loads

The required strength of structural members and connections shall be determined by structural analysis for the appropriate factored load combinations given in Sect. A4.

Design by either elastic or plastic analysis is permitted, except that plastic analysis is permitted only for steels with yield stress not exceeding 65 ksi and is subject to provisions of Sects. B5.2, C2, E1.2, F1.1, H1, and I1.

Except for hybrid girders and members of A514 steel, beams and girders (including members designed on the basis of composite action) which meet the requirements of the above, and are continuous over supports or are rigidly framed to columns by means of rivets, high-strength bolts, or welds, may be proportioned for 9/10 of the negative moments produced by gravity loading which are maximum at points of support, provided that, for such members, the maximum positive moment shall be increased by 1/10 of the average negative moments. This reduction shall not apply to moments produced by loading on cantilevers. If the negative moment is resisted by a column rigidly framed to the beam or girder, the 1/10 reduction may be used in proportioning the column for the combined axial and bending loading, provided that the stress, f_a, due to any concurrent axial load on the member, does not exceed $0.15F_a$.

2. Limit States

LRFD is a method of proportioning structures so that no applicable limit state is exceeded when the structure is subjected to all appropriate factored load combinations.

Strength limit states are related to safety and concern maximum load carrying capacity. Serviceability limit states are related to performance under normal service conditions. The term "resistance" includes both strength limit states and serviceability limit states.

3. Design for Strength

The design strength of each structural component or assemblage must equal or exceed the required strength based on the factored nominal loads. The design strength ϕR_n is calculated for each applicable limit state as the nominal strength R_n multiplied by a resistance factor ϕ. The required strength is determined for each applicable load combination as stipulated in Section A4.

Nominal strength R_n and resistance factors ϕ are given in the appropriate chapters. Additional strength considerations are given in Chap. K.

4. Design for Serviceability and Other Considerations

The overall structure and the individual members, connections and connectors should be checked for serviceability. Provisions for design for serviceability are given in Chap. L.

A6. REFERENCED CODES AND STANDARDS

The following documents are referenced in this Specification.

American National Standards Institute
- ANSI B18.1
- ANSI A58.1-1982

American Society of Testing and Materials

ASTM A6-84c	ASTM A27-84	ASTM A36-84a
ASTM A53-84a	ASTM A148-84	ASTM A242-84
ASTM A307-84	ASTM A325-84	ASTM A354-84b
ASTM A441-84	ASTM A449-84a	ASTM A490-84
ASTM A500-84	ASTM A501-84	ASTM A514-84a
ASTM A529-84	ASTM A563-84	ASTM A570-84a
ASTM A572-84	ASTM A588-84a	ASTM A606-84
ASTM A607-84a	ASTM A618-84	ASTM A668-83
ASTM A687-84	ASTM C33-85	ASTM C330-85
ASTM F436-84		

American Welding Society

AWS D1.1-85	AWS A5.1-81	AWS A5.5-81
AWS A5.17-80	AWS A5.18-79	AWS A5.20-79
AWS A5.23-80	AWS A5.28-79	AWS A5.29-80

Research Council on Structural Connections
- *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, 1985

A7. DESIGN DOCUMENTS

1. Plans

The design plans shall show a complete design with sizes, sections and relative locations of the various members. Floor levels, column centers and offsets shall be dimensioned. Drawings shall be drawn to a scale large enough to show the information clearly.

Design documents shall indicate the type or types of construction as defined in Sect. A2.2 and include the nominal loads and design strengths if necessary for preparation of shop drawings.

Where joints are to be assembled with high-strength bolts, the design documents shall indicate the connection type (slip-critical, tension or bearing).

Camber of trusses, beams and girders, if required, shall be called for in the design documents. The requirements for stiffeners and bracing shall be shown on the design documents.

2. Standard Symbols and Nomenclature

Welding and inspection symbols used on plans and shop drawings shall preferably be the American Welding Society symbols. Other adequate welding symbols may be used, provided a complete explanation thereof is shown in the design documents.

3. Notation for Welding

Notes shall be made in the design documents and on the shop drawings of those joints or groups of joints in which the welding sequence and technique of welding should be carefully controlled to minimize distortion.

Weld lengths called for in the design documents and on the shop drawings shall be the net effective lengths.

CHAPTER B. DESIGN REQUIREMENTS

This chapter contains provisions which are common to the Specification as a whole.

B1. GROSS AREA

The gross area A_g of a member at any point is the sum of the products of the thickness and the gross width of each element measured normal to the axis of the member. For angles, the gross width is the sum of the widths of the legs less the thickness.

Plate girders, coverplated beams and rolled or welded beams shall be proportioned on the basis of the gross section. No deduction shall be made for shop or field bolt holes in either flange unless the reduction of the area of either flange by such holes, calculated in accordance with the provisions of Sect. B2, exceeds 15% of the gross flange area, in which case the area in excess of 15% shall be deducted.

Hybrid girders may be proportioned on the basis of their gross section, provided that they are not required to resist an axial force greater than ϕ_b times $0.15 A_g F_{yf}$. No limit is placed on the flexural stress in the web.

Flanges of welded plate girders may be varied in thickness or width by splicing plates or by using cover plates.

B2. NET AREA

The net area A_n of a member is the sum of the products of the thickness and the net width of each element computed as follows.

In computing net area for tension, the width of a bolt hole shall be taken as 1/16-in. greater than the nominal dimension of the hole normal to the direction of applied stress. For shear the width shall be taken as the nominal dimension of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Sect. J3.7, of all holes in the chain, and adding, for each gage space in the chain, the quantity $s^2/4g$

where

s = longitudinal center-to-center spacing (pitch) of any two consecutive holes, in.
g = transverse center-to-center spacing (gage) between fastener gage lines, in.

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness.

The critical net area A_n of the part is obtained from that chain which gives the least net width.

In determining the net area across plug or slot welds, the weld metal shall not be considered as adding to the net area.

B3. EFFECTIVE NET AREA

When the load is transmitted directly to each of the cross-sectional elements by connectors, the effective net area A_e is equal to the net area A_n.

When the load is transmitted by bolts or rivets through some but not all of the cross-sectional elements of the member, the effective net area A_e shall be computed as:

$$A_e = U A_n \tag{B3-1}$$

where

A_n = net area of the member, in.^{2}
U = reduction coefficient

When the load is transmitted by welds through some but not all of the cross-sectional elements of the member, the effective net area A_e shall be computed as:

$$A_e = U A_g \tag{B3-2}$$

where

A_g = gross area of member, in.^{2}

Unless a larger coefficient can be justified by tests or other rational criteria, the following values of U shall be used:

a. W, M or S shapes with flange widths not less than ⅔ the depth, and structural tees cut from these shapes, provided the connection is to the flanges. Bolted or riveted connections shall have no fewer than three fasteners per line in the direction of stress .... $U = 0.90$
b. W, M or S shapes not meeting the conditions of subparagraph a, structural tees cut from these shapes and all other shapes, including built-up cross sections. Bolted or riveted connections shall have no fewer than three fasteners per line in the direction of stress .... $U = 0.85$
c. All members with bolted or riveted connections having only two fasteners per line in the direction of stress .... $U = 0.75$

When load is transmitted by transverse welds to some but not all of the cross sectional elements of W, M or S shapes and structural tees cut from these shapes, A_e shall be taken as the area of the directly connected elements.

When the load is transmitted to a plate by longitudinal welds along both edges at the end of the plate, the length of the welds shall not be less than the width of the plate. The effective net area A_e shall be computed as:

$$A_e = UA_g \tag{B3-2}$$

Unless a larger coefficient can be justified by tests or other rational criteria, the following values of U shall be used:

a. When $l > 2w$ .... $U = 1.0$
b. When $2w > l > 1.5w$ .... $U = 0.87$
c. When $1.5w > l > w$ .... $U = 0.75$

where

l = weld length, in.
w = plate width (distance between welds), in.

B4. STABILITY

General stability shall be provided for the structure as a whole and for each compression element.

Consideration shall be given to significant load effects resulting from the deflected shape of the structure or of individual elements of the lateral load resisting system.

B5. LOCAL BUCKLING

1. Classification of Steel Sections

Steel sections are classified as compact, noncompact and slender element sections. For a section to qualify as compact, its flanges must be continuously connected to the web or webs and the width-thickness ratios of its compression elements must not exceed the limiting width-thickness ratios λ_p from Table B5.1. If the width-thickness ratio of one or more compression elements exceeds λ_p but does not exceed λ_r, the section is noncompact. If the width-thickness ratio exceeds λ_r from Table B5.1, the element is referred to as a slender compression element.

For unstiffened elements which are supported along only one edge, parallel to the direction of the compression force, the width shall be taken as follows:

a. For flanges of I-shaped members and tees, the width b is half the full nominal width.
b. For legs of angles and flanges of channels and zees, the width b is the full nominal dimension.
c. For plates, the width b is the distance from the free edge to the first row of fasteners or line of welds.
d. For stems of tees, d is taken as the full nominal depth.

For stiffened elements, i.e., supported along two edges parallel to the direction of the compression force, the width shall be taken as follows:

a. For webs of rolled or formed sections, h is the clear distance between flanges less the fillet or corner radius at each flange; h_c is twice the distance from the neutral axis to the inside face of the compression flange less the fillet or corner radius.
b. For webs of built-up sections, h is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, and h_c is twice the distance from the neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used.
c. For flange or diaphragm plates in built-up sections, the width b is the distance between adjacent lines of fasteners or lines of welds.
d. For flanges of rectangular hollow structural sections, the width b is the clear distance between webs less the inside corner radius on each side. If the corner radius is not known, the flat width may be taken as the total section width minus three times the thickness.

For tapered flanges of rolled sections, the thickness is the nominal value halfway between the free edge and the corresponding face of the web.

TABLE B5.1
Limiting Width-Thickness Ratios for Compression Elements

Description of Element	Width-Thickness Ratio	Limiting Width-Thickness Ratios	
		λ_p	λ_r
Flanges of I-shaped rolled beams and channels in flexure	b/t	$65/\sqrt{F_y}$ [c]	$141/\sqrt{F_y - 10}$
Flanges of I-shaped hybrid or welded beams in flexure	b/t	$65/\sqrt{F_{yf}}$ [c]	$\dfrac{106}{\sqrt{F_{yw} - 16.5}}$
Flanges of I-shaped sections in pure compression, plates projecting from compression elements; outstanding legs of pairs of angles in continuous contact; flanges of channels in pure compression	b/t	NA	$95/\sqrt{F_y}$
Flanges of square and rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$190/\sqrt{F_y}$	$238/\sqrt{F_y - F_r}$ [e]
Unsupported width of cover plates perforated with a succession of access holes[b]	b/t	NA	$317/\sqrt{F_y - F_r}$ [e]
Legs of single angle struts; legs of double angle struts with separators; unstiffened elements, i.e., supported along one edge	b/t	NA	$76/\sqrt{F_y}$
Stems of tees	d/t	NA	$127/\sqrt{F_y}$
All other uniformly compressed stiffened elements, i.e., supported along two edges	b/t h_c/t_w	NA	$253/\sqrt{F_y}$
Webs in flexural compression[a]	h_c/t_w	$640/\sqrt{F_y}$ [c]	$970/\sqrt{F_y}$
Webs in combined flexural and axial compression	h_c/t_w	for $P_u/\phi_b P_y \le 0.125$ $\dfrac{640}{\sqrt{F_y}}\left(1 - \dfrac{2.75P_u}{\phi_b P_y}\right)$ [c] for $P_u/\phi_b P_y > 0.125$ $\dfrac{191}{\sqrt{F_y}}\left(2.33 - \dfrac{P_u}{\phi_b P_y}\right) \ge \dfrac{253}{\sqrt{F_y}}$ [c]	$970/\sqrt{F_y}$
Circular hollow sections In axial compression In flexure	D/t	 $2{,}070/F_y$ [d] $2{,}070/F_y$ [d]	 $3{,}300/F_y$ $8{,}970/F_y$

[a]For hybrid beams, use the yield strength of the flange F_{yf} instead of F_y.
[b]Assumes net area of plate at widest hole.
[c]Assumes an inelastic rotation capacity of 3. For structures in zones of high seismicity, a greater rotation capacity may be required. See Table C-B5.1.
[d]For plastic design use $1{,}300/F_y$.
[e]F_r = compressive residual stress in flange
= 10 ksi for rolled shapes
= 16.5 ksi for welded shapes.

2. Sections for Plastic Analysis

Plastic analysis is permitted when flanges subject to compression involving hinge rotation and all webs have a width-thickness ratio less than or equal to the limiting λ_p from Table B5.1. For circular hollow sections see Footnote d of Table B5.1.

Plastic analysis is subject to the limitations as outlined in Sect. A5.1.

3. Slender Compression Elements

For the flexural design of I-shaped sections, channels and rectangular or circular sections with slender compression elements, see Appendix F1.7. For other shapes in flexure or members in axial compression that have slender compression elements, see Appendix B5.3. For plate girders with $h_c/t_w > 970/\sqrt{F_{yf}}$, see Appendix G.

B6. BRACING AT SUPPORTS

At points of support for beams, girders and trusses, restraint against rotation about their longitudinal axis shall be provided unless restraint against rotation is otherwise assured.

B7. LIMITING SLENDERNESS RATIOS

For members whose design is based on compressive force, the slenderness ratio Kl/r preferably should not exceed 200.

For members whose design is based on tensile force, the slenderness ratio L/r preferably should not exceed 300. The above limitation does not apply to rods in tension. Such tension members may be subject to compressive force not exceeding 50% of the design compressive strength when such compressive force is due to wind or earthquake.

Notes

CHAPTER C.
FRAMES AND OTHER STRUCTURES

This chapter specifies general requirements to assure stability of the structure as a whole.

C1. SECOND ORDER EFFECTS

Second order ($P\Delta$) effects shall be considered in the design of frames.

C2. FRAME STABILITY

1. Braced Frames

In trusses and frames where lateral stability is provided by diagonal bracing, shear walls or equivalent means, the effective length factor K for compression members shall be taken as unity, unless structural analysis shows that a smaller value may be used.

The vertical bracing system for a braced multistory frame shall be adequate, as determined by structural analysis, to prevent buckling of the structure and maintain the lateral stability of the structure, including the overturning effects of drift, under the factored loads given in Sect. A4.

The vertical bracing system for a multistory frame may be considered to function together with in-plane shear-resisting exterior and interior walls, floor slabs and roof decks, which are properly secured to the structural frames. The columns, girders, beams and diagonal members, when used as the vertical bracing system, may be considered to comprise a vertical cantilever, simply connected truss in the analyses for frame buckling and lateral instability. Axial deformation of all members in the vertical bracing system shall be included in the lateral stability analysis.

Girders and beams included in the vertical bracing system of a braced multistory frame shall be proportioned for axial force and moment caused by concurrent factored horizontal and gravity loads.

2. Unbraced Frames

In frames where lateral stability depends upon the bending stiffness of rigidly connected beams and columns, the effective length factor K of compression members shall be determined by structural analysis and shall be not less than unity.

Analysis of the required strength of unbraced multistory frames shall include the effects of frame instability and column axial deformation under the factored loads given in Sect. A4.

In plastic design the axial force in the columns caused by factored gravity plus factored horizontal loads shall not exceed $0.75A_gF_y$.

CHAPTER D. TENSION MEMBERS

This section applies to prismatic members subject to axial tension caused by static forces acting through the centroidal axis. For members subject to combined axial tension and flexure, see Sect. H1. For members subject to fatigue, see Sect. K4. For tapered members, see Appendix F4. For threaded rods see Sect. J3.

D1. DESIGN TENSILE STRENGTH

The design strength of tension members $\phi_t P_n$ shall be the lower value obtained according to the limit states of yielding in the gross section and fracture in the net section.

a. For yielding in the gross section:

$$\phi_t = 0.90$$
$$P_n = F_y A_g \tag{D1-1}$$

b. For fracture in the net section:

$$\phi_t = 0.75$$
$$P_n = F_u A_e \tag{D1-2}$$

where

A_e = effective net area, in.^{2}
A_g = gross area of member, in.^{2}
F_y = specified minimum yield stress, ksi
F_u = specified minimum tensile strength, ksi
P_n = nominal axial strength, kips

When members without holes are fully connected by welds, the effective net section used in Formula D1-2 shall be computed using the smaller of the gross area of the member or the effective area of the welds as defined in Sect. J2. When holes are present in a welded member between end connections, or at the welded connection in the case of plug or slot welds, the net section through the holes shall be used in Formula D1-2.

D2. BUILT-UP MEMBERS

The longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates shall not exceed:

24 times the thickness of the thinner plate or 12 in. for painted members or unpainted members not subject to corrosion.
14 times the thickness of the thinner plate or 7 in. for unpainted members of weathering steel subject to atmospheric corrosion.

The longitudinal spacing of connectors between components should preferably limit the slenderness ratio in any component between the connectors to 300 or less.

Either perforated cover plates or tie plates without lacing may be used on the open sides of built-up tension members. Tie plates shall have a length not less than ⅔ the distance between the lines of welds or fasteners connecting them to the components of the member. The thickness of such tie plates shall not be less than 1/50 of the distance between these lines. The longitudinal spacing of intermittent welds or fasteners at tie plates shall not exceed 6 in. The spacing of tie plates shall be such that the slenderness ratio of any component in the length between tie plates should preferably not exceed 300.

D3. EYEBARS AND PIN-CONNECTED MEMBERS

The design strength of eyebars shall be determined in accordance with Sect. D1a with A_g taken as the cross-sectional area of the body.

Eyebars shall be of uniform thickness, without reinforcement at the pin holes, and have circular heads whose periphery is concentric with the pin hole.

The radius of transition between the circular head and the eyebar body shall be not less than the head diameter.

The width of the body of the eyebars shall not exceed eight times its thickness.

The thickness can be less than ½-in. only if external nuts are provided to tighten pin plates and filler plates into snug contact. The width b from the hole edge to the plate edge perpendicular to the direction of applied load shall be greater than ⅔ and, for the purpose of calculation, not more than ¾ times the eyebar body width.

The pin diameter shall not be less than ⅞ times the eyebar body width.

The pin-hole diameter shall not be more than 1/32-in. greater than the pin diameter.

For steels having a yield stress greater than 70 ksi, the hole diameter shall not exceed five times the plate thickness and the width of the eyebar body shall be reduced accordingly.

In pin-connected members the pin hole shall be located midway between the edges of the member in the direction normal to the applied force. For pin-connected members in which the pin is expected to provide for relative movement between connected parts while under full load, the diameter of pin hole shall not be more than 1/32-in. greater than the diameter of the pin. The width of the plate beyond the pin hole shall be not less than the effective width on either side of the pin hole.

In pin-connected plates other than eyebars, the design strength shall be determined according to Formula D1-2 and the bearing strength of the projected area of the pin shall be determined according to Sect. J8. The minimum net area beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than ⅔ of the net area required for strength across the pin hole.

The design strength of a pin-connected member $\phi_t P_n$ shall be the lowest value of the following limit states.

a. Tension on the net effective area:

$$\begin{aligned} \phi_t &= 0.75 \\ P_n &= 2tb_{eff}F_u \end{aligned} \qquad \text{(D3-1)}$$

b. Shear on the effective area:

$$\phi_{sf} = 0.75 \qquad \text{(D3-2)}$$
$$P_n = A_{sf} F_y$$

c. Bearing on the projected area of the pin:

$$\phi = 1.0 \qquad \text{(D3-3)}$$
$$P_n = A_{pb} F_y$$

where

a = shortest distance from edge of the pin hole to the edge of the member measured parallel to the direction of the force, in.
A_{pb} = projected bearing area, in.^{2}
A_{sf} = $2t\,(a + d/2)$, in.^{2}
b_{eff} = $2t + 0.63$, but not more than the actual distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force, in.
d = pin diameter, in.
t = thickness of plate, in.

The corners beyond the pin hole may be cut at 45° to the axis of the member, provided the net area beyond the pin hole, on a plane perpendicular to the cut, is not less than that required beyond the pin hole parallel to the axis of the member.

Thickness limitations on both eyebars and pin-connected plates may be waived whenever external nuts are provided so as to tighten pin plates and filler plates into snug contact. When the plates are thus contained, the bearing strength shall be determined according to Sect. J8.

CHAPTER E. COLUMNS AND OTHER COMPRESSION MEMBERS

This section applies to prismatic members subject to axial compression through the centroidal axis. For members subject to combined axial compression and flexure, see Chap. H. For tapered members, see Appendix F4.

E1. EFFECTIVE LENGTH AND SLENDERNESS LIMITATIONS

1. Effective Length

The effective length factor K shall be determined in accordance with Sect. C2.

2. Plastic Analysis

Plastic analysis, as limited in Sect. A5.1, is permitted if the column slenderness parameter λ_c defined by Formula E2-4 does not exceed $1.5K$.

E2. DESIGN COMPRESSIVE STRENGTH

The design strength of compression members whose elements have width-thickness ratios less than λ_r of Sect. B5.1 is $\phi_c P_n$

$$\phi_c = 0.85$$

$$P_n = A_g F_{cr} \tag{E2-1}$$

for $\lambda_c \leq 1.5$

$$F_{cr} = (0.658^{\lambda_c^2})F_y \tag{E2-2}$$

for $\lambda_c > 1.5$

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right]F_y \tag{E2-3}$$

where

$$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}} \tag{E2-4}$$

A_g = gross area of member, in.^{2}
F_y = specified yield stress, ksi
E = modulus of elasticity, ksi
K = effective length factor
l = unbraced length of member, in.
r = governing radius of gyration about plane of buckling, in.

For members whose elements do not meet the requirements of Sect. B5.1, see Appendix B5.3.

E3. FLEXURAL-TORSIONAL BUCKLING

Singly symmetric and unsymmetric columns, such as angle or tee-shaped columns, and doubly symmetric columns such as cruciform or built-up columns with very thin walls, may require consideration of the limit states of flexural-torsional and torsional buckling. See Appendix E3 for the determination of design strength for these limit states.

E4. BUILT-UP MEMBERS

At the ends of built-up compression members bearing on base plates or milled surfaces, all components in contact with one another shall be connected by a weld having a length not less than the maximum width of the member or by bolts spaced longitudinally not more than four diameters apart for a distance equal to 1½ times the maximum width of the member.

Along the length of built-up compression members between the end connections required above, longitudinal spacing for intermittent welds, bolts or rivets shall be adequate to provide for the transfer of calculated stress. However, where a component of a built-up compression member consists of an outside plate, except as provided in the next sentence, the maximum spacing shall not exceed the thickness of the thinner outside plate times $127/\sqrt{F_y}$, nor 12 in., when intermittent welds are provided along the edges of the components or when fasteners are provided on all gage lines at each section. When fasteners are staggered, the maximum spacing on each gage line shall not exceed the thickness of the thinner outside plate times $190/\sqrt{F_y}$, nor 18 in.

For unpainted built-up members made of weathering steel which will be exposed to atmospheric corrosion, the fasteners connecting a plate and a rolled shape or two-plate components in contact with one another shall not exceed 14 times the thickness of the thinnest part nor 7 in. and the maximum edge distance shall not exceed eight times the thickness of the thinnest part, nor 5 in.[91]

Compression members composed of two or more rolled shapes shall be connected to one another at intervals such that the slenderness ratio L/r of either shape, between the fasteners, does not exceed the governing slenderness ratio of the built-up member. The least radius of gyration r shall be used in computing the slenderness ratio of each component part.

The design strength of built-up members composed of two or more shapes shall be determined in accordance with Section E2 or Appendix E3 subject to the following modification. If the buckling mode involves relative deformation that produce shear forces in the connectors between individual shapes, Kl/r is replaced by $(Kl/r)_m$ determined as follows:

a. for snug-tight bolted connectors:

$$\left(\frac{Kl}{r}\right)_m = \sqrt{\left(\frac{Kl}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad \text{(E4-1)}$$

b. for welded connectors and for fully tightened bolted connectors as required for slip-critical joints:

with $\frac{a}{r_i} > 50$

$$\left(\frac{Kl}{r}\right)_m = \sqrt{\left(\frac{Kl}{r}\right)_o^2 + \left(\frac{a}{r_i} - 50\right)^2} \qquad \text{(E4-2)}$$

with $\frac{a}{r_i} \leq 50$

$$\left(\frac{Kl}{r}\right)_m = \left(\frac{Kl}{r}\right)_o \qquad \text{(E4-3)}$$

where

$\left(\frac{Kl}{r}\right)_o$ = column slenderness of built-up member acting as a unit

$\frac{a}{r_i}$ = largest column slenderness of individual components

$\left(\frac{Kl}{r}\right)_m$ = modified column slenderness of built-up member

a = distance between connectors

r_i = minimum radius of gyration of individual component

Open sides of compression members built up from plates or shapes shall be provided with continuous cover plates perforated with a succession of access holes. The unsupported width of such plates at access holes, as defined in Sect. B5.1, is assumed to contribute to the design strength provided that:

a. The width-thickness ratio conforms to the limitations of Sect. B5.1.
b. The ratio of length (in direction of stress) to width of hole shall not exceed two.
c. The clear distance between holes in the direction of stress shall be not less than the transverse distance between nearest lines of connecting fasteners or welds.
d. The periphery of the holes at all points shall have a minimum radius of 1½ in.

The function of perforated cover plates may be performed by lacing with tie plates at each end and at intermediate points if the lacing is interrupted. Tie plates shall be as near the ends as practicable. In main members providing design strength, the end tie plates shall have a length of not less than the distance between the lines of fasteners or welds connecting them to the components of the member. Intermediate tie plates shall have a length not less than ½ of this distance. The thickness of tie plates shall be not less than 1/50 of the distance between lines of welds or fasteners connecting them to the segments of the members. In welded construction, the welding on each line connecting a tie plate shall aggregate not less than ⅓ the length of the plate. In bolted and riveted construction, the spacing in the direction of stress in tie plates shall be not more than six diameters and the tie plates shall be connected to each segment by at least three fasteners.

Lacing, including flat bars, angles, channels or other shapes employed as lacing, shall be so spaced that the L/r ratio of the flange included between their connections shall not exceed the governing slenderness ratio for the member as a whole. Lacing shall be proportioned to provide a shearing strength normal to the axis of the member equal to 2% of the compressive design strength of the member. The L/r ratio for lacing bars arranged in single systems shall not exceed 140. For double lacing this ratio shall not exceed 200. Double lacing bars shall be joined at their intersections. For lacing bars in compression, L may be taken as the unsupported length of the lacing bar between welds or fasteners connecting it to the components of the built-up member for single lacing, and 70% of that distance for double lacing. The inclination of lacing bars to the axis of the member shall preferably be not less than 60° for single lacing and 45° for double lacing. When the distance between the lines of welds or fasteners in the flanges is more than 15 in., the lacing shall preferably be double or be made of angles.

E5. PIN-CONNECTED COMPRESSION MEMBERS

Pin-connections of pin-connected compression members shall conform to the requirements of Sect. D3 except Formulas D3-1 and D3-2 do not apply.

CHAPTER F.
BEAMS AND OTHER FLEXURAL MEMBERS

This section applies to singly or doubly symmetric beams including hybrid beams and girders loaded in the plane of symmetry. It also applies to channels loaded in a plane passing through the shear center parallel to the web or restrained against twisting at load points and points of support. For design flexural strength for members not covered in Sect. F1, see Appendix F1.7. For members subject to combined flexural and axial force, see Sect. H1. For unsymmetric beams and beams subject to torsion combined with flexure, see Sect. H2.

F1. DESIGN FOR FLEXURE

1. Unbraced Length for Plastic Analysis

Plastic analysis, as limited in Sect. A5, is permitted when the laterally unbraced length L_b of the compression flange at plastic hinge locations associated with the failure mechanism, for a compact section bent about the major axis, does not exceed L_{pd}, determined as follows:

a. For doubly symmetric and singly symmetric I-shaped members with the compression flange larger than the tension flange (including hybrid members) loaded in the plane of the web

$$L_{pd} = \frac{3{,}600 + 2{,}200(M_1/M_p)}{F_y} r_y \tag{F1-1}$$

where

F_y = specified minimum yield stress of the compression flange, ksi
M_1 = smaller moment at end of unbraced length of beam, kip-in.
M_p = plastic moment (= $F_y Z$ for homogeneous sections; computed from fully plastic stress distribution for hybrids), kip-in.
r_y = radius of gyration about minor axis, in.
(M_1/M_p) is positive when moments cause reverse curvature

b. For solid rectangular bars and symmetric box beams

$$L_{pd} = \frac{5{,}000 + 3{,}000(M_1/M_p)}{F_y} r_y \geq 3{,}000\, r_y/F_y \tag{F1-2}$$

There is no limit on L_b for members with circular or square cross sections nor for any beam bent about its minor axis.

In the region of the last hinge to form, and in regions not adjacent to a plastic hinge, the flexural design strength shall be determined in accordance with Sect. F1.2.

2. Flexural Design Strength

The flexural design strength, determined by the limit state of lateral-torsional buckling, is $\phi_b M_n$, where the nominal strength M_n shall be determined in accordance with the following sections, and $\phi_b = 0.90$.

3. Compact Section Members with $L_b \leq L_r$

For laterally unsupported compact section members bent about the major axis:

$$M_n = C_b\left[M_p - (M_p - M_r)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p \tag{F1-3}$$

where

$C_b = 1.75 + 1.05(M_1/M_2) + 0.3(M_1/M_2)^2 \leq 2.3$ where M_1 is the smaller and M_2 the larger end moment in the unbraced segment of the beam; M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

$C_b = 1.0$ for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.*

$L_b =$ distance between points braced against lateral displacement of the compression flange, or between points braced to prevent twist of the cross section.

For I-shaped members including hybrid sections and channels bent about their major axis:

$$L_p = \frac{300r_y}{\sqrt{F_{yf}}} \tag{F1-4}$$

For solid rectangular bars and box beams:

$$L_p = \frac{3{,}750r_y}{M_p}\sqrt{JA} \tag{F1-5}$$

where

$A =$ cross-sectional area, in.^{2}
$J =$ torsional constant, in.^{4}

The limiting laterally unbraced length L_r and the corresponding buckling moment M_r shall be determined as follows:

a. For I-shaped members, doubly symmetric and singly symmetric with the compression flange larger than or equal to the tension flange, and channels loaded in the plane of the web:

$$L_r = \frac{r_y X_1}{(F_{yw} - F_r)}\sqrt{1 + \sqrt{1 + X_2(F_{yw} - F_r)^2}} \tag{F1-6}$$

$$M_r = (F_{yw} - F_r)S_x \tag{F1-7}$$

where

$$X_1 = \frac{\pi}{S_x}\sqrt{\frac{EGJA}{2}} \tag{F1-8}$$

$$X_2 = 4\frac{C_w}{I_y}\left(\frac{S_x}{GJ}\right)^2 \tag{F1-9}$$

*For the use of larger C_b values, see Structural Stability Research Council *Guide to Stability Design Criteria for Metal Structures*, 3rd Ed., pg. 135.

S_x = section modulus about major axis, in.^{3}
E = modulus of elasticity of steel (29,000 ksi)
G = shear modulus of elasticity of steel (11,200 ksi)
F_{yw} = yield stress of web, ksi
I_y = moment of inertia about y-axis
C_w = warping constant, in.^{6}
F_r = compressive residual stress in flange; 10 ksi for rolled shapes, 16.5 for welded shapes

b. For singly symmetric, I-shaped members with the compression flange larger than the tension flange, use S_{xc} in place of S_x in Formulas F1-7 through F1-9, or see Table A-F1.1.

c. For symmetric box section bent about the major axis and loaded in the plane of symmetry, M_r and L_r shall be determined from Formula F1-7 and F1-10 respectively.

d. For solid rectangular bars bent about the major axis:

$$L_r = \frac{57{,}000 r_y \sqrt{JA}}{M_r} \qquad \text{(F1-10)}$$

$$M_r = F_y S_x \qquad \text{(F1-11)}$$

4. Compact Section Members with $L_b > L_r$

For laterally unsupported members with compact section members bent about the major axis:

$$M_n = M_{cr} \leq C_b M_r \qquad \text{(F1-12)}$$

where M_{cr} is the critical elastic moment, determined as follows:

a. For I-shaped members, doubly symmetric and singly symmetric with compression flange larger than the tension flange (including hybrid members) and channels loaded in the plane of the web:

$$M_{cr} = C_b \frac{\pi}{L_b} \sqrt{E I_y G J + \left(\frac{\pi E}{L_b}\right)^2 I_y C_w} \qquad \text{(F1-13)}$$

$$= \frac{C_b S_x X_1 \sqrt{2}}{L_b / r_y} \sqrt{1 + \frac{X_1^2 X_2}{2(L_b/r_y)^2}}$$

b. For solid rectangular bars and symmetric box sections:

$$M_{cr} = \frac{57{,}000\ C_b \sqrt{JA}}{L_b / r_y} \qquad \text{(F1-14)}$$

5. Tees and Double-angle Beams

The nominal strength of tees and double-angle beams loaded in the plane of symmetry, with flange and web slenderness ratios less than the corresponding values of λ_r in Table B5.1:

$$M_n = M_{cr} = \frac{C_b \pi \sqrt{E I_y G J}}{L_b} [B + \sqrt{1 + B^2}] \leq M_y \qquad \text{(F1-15)}$$

where

$$B = \pm 2.3\ (d/L_b)\sqrt{I_y/J} \tag{F1-16}$$

The plus sign for B applies when the stem is in tension and the minus sign applies when the stem is in compression.

6. Noncompact Plate Girders

The nominal strength of a doubly symmetric, single-web plate girder, including hybrid sections, shall be calculated by the provisions of Appendix F1.7 if $h_c/t_w \leq 970/\sqrt{F_{yf}}$ or by the provisions of Appendix G if $h_c/t_w > 970/\sqrt{F_{yf}}$.

7. Nominal Flexural Strength of Other Sections

There is no lateral torsional buckling limit state for circular or square shapes nor for any shape bent about its minor axis.

For the nominal strength M_n of other cross section types, including noncompact sections or sections with slender elements, see Appendix F1.7. See Appendix G for design of plate girders with slender webs.

F2. DESIGN FOR SHEAR

This section applies to the web (or webs in the case of multiple web members) of singly or doubly symmetric beams, including hybrid beams, subject to shear in the plane of symmetry, and channels subject to shear in the web. Where failure might occur by shear along a plane through fasteners, refer to Sect. J4. For members subjected to high shear from concentrated loads, see Sect. K1.7.

1. Web Area Determination

The web area A_w shall be taken as the overall depth d times the web thickness t_w.

2. Design Shear Strength

The design shear strength of webs is $\phi_v V_n$, where $\phi_v = 0.90$ and the nominal shear strength V_n is determined as follows:

For $\dfrac{h}{t_w} \leq 187\sqrt{k/F_{yw}}$

$$V_n = 0.6\ F_{yw} A_w \tag{F2-1}$$

for $187\sqrt{k/F_{yw}} < \dfrac{h}{t_w} \leq 234\sqrt{k/F_{yw}}$

$$V_n = 0.6\ F_{yw} A_w \frac{187\sqrt{k/F_{yw}}}{h/t_w} \tag{F2-2}$$

for $\dfrac{h}{t_w} > 234\sqrt{k/F_{yw}}$

$$V_n = A_w \frac{26{,}400k}{(h/t_w)^2} \tag{F2-3}$$

The web plate buckling coefficient k is given by

$$k = 5 + \frac{5}{(a/h)^2} \tag{F2-4}$$

Except that k shall be taken as 5 if a/h exceeds 3.0 or $[260/(h/t_w)]^2$. When stiffeners are not required, $k = 5$. In unstiffened girders, h/t shall not exceed 260.

Maximum (h/t_w) limits are given in Appendix G1.

An alternative design method for plate girders utilizing tension field action is given in Appendix G.

F3. TRANSVERSE STIFFENERS

Transverse stiffeners are not required when $h/t_w \leq 418/\sqrt{F_{yw}}$, or when the required shear V_u, as determined by structural analysis for the factored loads, is less than or equal to $\phi_v V_n$ for $k = 5$ given in Sect. F2. Transverse stiffeners used to develop the web design shear strength as provided in Sect. F2 shall have a moment of inertia about an axis in the web center for stiffener pairs or about the face in contact with the web plate for single stiffeners, which shall not be less than $at_w^3 j$, where

$$j = \frac{2.5}{(a/h)^2} - 2 \geq 0.5 \tag{F3-1}$$

Intermediate stiffeners may be stopped short of the tension flange, provided bearing is not needed to transmit a concentrated load or reaction. The weld by which intermediate stiffeners are attached to the web shall be terminated not less than four times nor more than six times the web thickness from the near toe of the web-to-flange weld. When single stiffeners are used, they shall be attached to the compression flange, if it consists of a rectangular plate, to resist any uplift tendency due to torsion in the plate. When lateral bracing is attached to a stiffener, or a pair of stiffeners, these, in turn, shall be connected to the compression flange to transmit one percent of the total flange stress, unless the flange is composed only of angles.

Bolts connecting stiffeners to the girder web shall be spaced not more than 12 in. o. c. If intermittent fillet welds are used, the clear distance between welds shall not be more than 16 times the web thickness nor more than 10 in.

F4. WEB-TAPERED MEMBERS

See Appendix F4.

CHAPTER G. PLATE GIRDERS

Plate girders shall be distinguished from beams on the basis of the web slenderness ratio h_c/t_w. When this value is greater than $970/\sqrt{F_{yf}}$ the provisions of Appendix G shall apply for design flexural strength, otherwise Appendix F1.7 is applicable.

For design shear strength and transverse stiffener design see appropriate sections in Chap. F or see Appendix G3 and G4 if tension field action is utilized.

CHAPTER H. MEMBERS UNDER TORSION AND COMBINED FORCES

This section applies to prismatic members subjected to axial force and flexure about one or both axes of symmetry, with or without torsion, and torsion only. For web-tapered members, see Appendix F4.

H1. SYMMETRIC MEMBERS SUBJECT TO BENDING AND AXIAL FORCE

1. Doubly and Singly Symmetric Members in Flexure and Tension

The interaction of flexure and tension in symmetric shapes shall be limited by Formulas H1-1a and H1-1b

for $\dfrac{P_u}{\phi P_n} \geq 0.2$

$$\frac{P_u}{\phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \qquad \text{(H1-1a)}$$

for $\dfrac{P_u}{\phi P_n} < 0.2$

$$\frac{P_u}{2\phi P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \qquad \text{(H1-1b)}$$

where

P_u = required tensile strength, kips
P_n = nominal tensile strength determined in accordance with Sect. D1, kips
M_u = required flexural strength, kip-in.
M_n = nominal flexural strength determined in accordance with Sect. F1, kip-in.
ϕ_t = resistance factor for tension, $\phi_t = 0.90$ (see Sect. D1)
ϕ_b = resistance factor for flexure = 0.90

Second order effects may be considered in the determination of M_u for use in Formulas H1-1a and H1-1b. A more detailed analysis of the interaction of flexure and tension may be made in lieu of using Formulas H1-1a and H1-1b.

2. Doubly and Singly Symmetric Members in Flexure and Compression

The interaction of flexure and compression in symmetric shapes shall be limited by Formulas H1-1a and H1-1b where

P_u = required compressive strength, kips
P_n = nominal compressive strength determined in accordance with Sect. E2, kips
M_u = required flexural strength determined in accordance with subsection a, below, kip-in.
M_n = nominal flexural strength determined in accordance with subsection b, below, kip-in.
ϕ_c = resistance factor for compression, $\phi_c = 0.85$ (see Sect. E2)
ϕ_b = resistance factor for flexure = 0.90

a. Determination of M_u

In structures designed on the basis of elastic analysis, M_u may be determined from a second order elastic analysis using factored loads. In structures designed on the basis of plastic analysis, M_u shall be determined from a plastic analysis that satisfies the requirements of Sects. C1 and C2. In structures designed on the basis of elastic first order analysis the following procedure for the determination of M_u may be used in lieu of a second order analysis:

$$M_u = B_1 M_{nt} + B_2 M_{\ell t} \qquad \text{(H1-2)}$$

where

M_{nt} = required flexural strength in member assuming there is no lateral translation of the frame, kip-in.
$M_{\ell t}$ = required flexural strength in member as a result of lateral translation of the frame only, kip-in.

$$B_1 = \frac{C_m}{(1 - P_u/P_e)} \geq 1 \qquad \text{(H1-3)}$$

P_e = $A_g F_y / \lambda_c^2$ where λ_c is defined by Formula E2-4 with $K \leq 1.0$ in the plane of bending.

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

i. For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending,

$$C_m = 0.6 - 0.4(M_1/M_2) \tag{H1-4}$$

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

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

for members whose ends are restrained . . . . $C_m = 0.85$
for members whose ends are unrestrained. . . . . $C_m = 1.0$

$$B_2 = \frac{1}{1 - \Sigma P_u \left(\dfrac{\Delta_{oh}}{\Sigma HL} \right)} \tag{H1-5}$$

or

$$B_2 = \frac{1}{1 - \dfrac{\Sigma P_u}{\Sigma P_e}} \tag{H1-6}$$

ΣP_u = required axial load strength of all columns in a story, kips
Δ_{oh} = translation deflection of the story under consideration, in.
ΣH = sum of all story horizontal forces producing Δ_{oh}, kips
L = story height, in.
P_e = $A_g F_y / \lambda_c^2$, kips, where λ_c is the slenderness parameter defined by Formula E2-4, in which the effective length factor K in the plane of bending shall be determined in accordance with Sect. C2.2, but shall not be less than unity.

b. Determination of M_n

In the use of Formulas H1-1a and H1-1b, M_{nx} shall be determined in accordance with Sect. F1. The actual value of C_b from Sect. F1.3 may be used, provided that the maximum moment M_{ux} occurs at the end of the member or at the end of an unbraced segment of a member. When the maximum moment occurs between the ends, M_{nx} shall be determined with $C_b = 1.0$. When Formula H1-2 is used for determining M_u, the maximum moment for a braced member bent about the strong axis and laterally braced only at its ends will occur at an end whenever the calculated value of B_1 is equal to or less than 1.

H2. UNSYMMETRIC MEMBERS AND MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE AND/OR AXIAL FORCE

The design strength ϕF_y of the member shall equal or exceed the required strength expressed in terms of the normal stress f_{un} or the shear stress f_{uv}, determined by elastic analysis for the factored loads:

a. For the limit state of yielding under normal stress:

$$\phi F_y \geq f_{un} \tag{H2-1}$$

$$\phi = 0.90$$

b. For the limit state of yielding under shear stress:

$$0.6\phi F_y \geq f_{uv} \tag{H2-2}$$

$$\phi = 0.90$$

c. For the limit state of buckling:

$$\phi_c F_{cr} \geq f_{un} \text{ or } f_{uv}\text{, as applicable} \tag{H2-3}$$

where

ϕ_c = 0.85 and F_{cr} may be determined from Formula A-E3-2 or A-E3-3, as applicable.

Some constrained local yielding is permitted in areas adjacent to areas which remain elastic.

H3. ALTERNATE INTERACTION EQUATIONS FOR MEMBERS UNDER COMBINED STRESS

See Appendix H3.

CHAPTER I.
COMPOSITE MEMBERS

This chapter applies to composite columns composed of rolled or built-up structural steel shapes, pipe or tubing and structural concrete acting together and to steel beams supporting a reinforced concrete slab so interconnected that the beams and the slab act together to resist bending. Simple and continuous composite beams with shear connectors and concrete-encased beams, constructed with or without temporary shores, are included.

I1. DESIGN ASSUMPTIONS

Force Determination In determining forces in members and connections of a structure that includes composite beams, consideration must be given to the effective sections at the time each increment of load is applied.

Elastic Analysis For an elastic analysis of continuous composite beams without haunched ends, it is acceptable to assume that the stiffness of a beam is uniform throughout the beam length and may be computed using the moment of inertia of the composite transformed section in the positive moment region.

Plastic Analysis When plastic analysis is used, the strength of flexural composite members shall be determined from plastic stress distributions as specified in Sect. I3.

Plastic Stress Distribution for Positive Moment If the slab in the positive moment region is connected to the steel beam with shear connectors, a concrete stress of $0.85 f_c'$ may be assumed uniformly distributed throughout the effective compression zone. Concrete tensile strength shall be neglected. A uniformly distributed steel stress of F_y shall be assumed throughout the tension zone and throughout the compression zone in the structural steel section. The net tensile force in the steel section shall be equal to the compressive force in the concrete slab.

Plastic Stress Distribution for Negative Moment If the slab in the negative moment region is connected to the steel beam with shear connectors, a tensile stress of F_{yr} shall be assumed in all adequately developed longitudinal reinforcing bars within the effective width of the concrete slab. Concrete tensile strength shall be neglected. A uniformly distributed steel stress of F_y shall be assumed throughout the tension zone and throughout the compression zone in the structural steel section. The net compressive force in the steel section shall be equal to the total tensile force in the reinforcing steel.

Elastic Stress Distribution When a determination of elastic stress distribution is required, strains in steel and concrete shall be assumed directly proportional to the distance from the neutral axis. The stress shall equal strain times E or E_c. Concrete tensile strength shall be neglected. Maximum stress in the steel shall not exceed F_y. Maximum compressive stress in the concrete shall not exceed $0.85 f_c'$. In composite hybrid beams, the maximum stress in the steel flange shall not exceed F_{yf} but the strain in the web may exceed the yield strain; the stress shall be taken as F_{yw} at such locations.

Fully Composite Beam Shear connectors are provided in sufficient numbers to develop the maximum flexural strength of the composite beam. For elastic stress distribution it may be assumed that no slip occurs.

Partially Composite Beam The shear strength of shear connectors governs the flexural strength of the partially composite beam. Elastic computations such as those for deflections, fatigue and vibrations should include the effect of slip.

Concrete-encased Beam A beam totally encased in concrete cast integrally with the slab may be assumed to be interconnected to the concrete by natural bond, without additional anchorage, provided that: (1) concrete cover over beam sides and soffit is at least 2 in.; (2) the top of the beam is at least 1½ in. below the top and 2 in. above the bottom of the slab; and (3) concrete encasement contains adequate mesh or other reinforcing steel to prevent spalling of concrete.

Composite Column A steel column fabricated from rolled or built-up steel shapes and encased in structural concrete or fabricated from steel pipe or tubing and filled with structural concrete.

I2. COMPRESSION MEMBERS

1. Limitations

To qualify as a composite column, the following limitations shall be met.

a. The cross-sectional area of the steel shape, pipe or tubing must comprise at least 4% of the total composite cross section.
b. Concrete encasement of a steel core shall be reinforced with longitudinal load carrying bars, longitudinal bars to restrain concrete and lateral ties. Longitudinal load carrying bars shall be continuous at framed levels; longitudinal restraining bars may be interrupted at framed levels. The spacing of ties shall be not greater than ⅔ of the least dimension of the composite cross section. The cross-sectional area of the transverse and longitudinal reinforcement shall be at least 0.007 sq. in. per inch of bar spacing. The encasement shall provide at least 1.5 in. of clear cover outside of both transverse and longitudinal reinforcement.
c. Concrete shall have a specified compressive strength f_c' of not less than 3 ksi nor more than 8 ksi for normal weight concrete and not less than 4 ksi for light weight concrete.
d. The specified minimum yield stress of structural steel and reinforcing bars used in calculating the strength of a composite column shall not exceed 55 ksi.
e. The minimum wall thickness of structural steel pipe or tubing filled with concrete shall be equal to $b\sqrt{F_y/3E}$ for each face of width b in rectangular sections and $D\sqrt{F_y/8E}$ for circular sections of outside diameter D.

2. Design Strength

The design strength of axially loaded composite columns is $\phi_c P_n$, where $\phi_c = 0.85$ and the nominal axial compressive strength P_n shall be determined from Formulas E2-1 through E2-4 with the following modifications:

a. A_s = gross area of steel shape, pipe or tubing, in.^{2} (replaces A_g)
   r_m = radius of gyration of the steel shape, pipe or tubing except that for steel shapes it shall not be less than 0.3 times the overall thickness of the composite cross section in the plane of buckling, in. (replaces r)
b. Replace F_y with modified yield stress F_{my} from Formula I2-1 and replace E with modified modulus of elasticity E_m from Formula I2-2:

$$F_{my} = F_y + c_1 F_{yr}(A_r/A_s) + c_2 f_c'(A_c/A_s) \qquad \text{(I2-1)}$$

$$E_m = E + c_3 E_c(A_c/A_s) \qquad \text{(I2-2)}$$

where

A_c = area of concrete, in.^{2}
A_r = area of longitudinal reinforcing bars, in.^{2}
A_s = area of steel, in.^{2}
E = modulus of elasticity of steel, ksi
E_c = modulus of elasticity of concrete,* ksi
F_y = specified minimum yield stress of steel shape, pipe or tubing, ksi
F_{yr} = specified minimum yield stress of longitudinal reinforcing bars, ksi
f'_c = specified compressive strength of concrete, ksi
c_1, c_2, c_3 = numerical coefficients. For concrete-filled pipe and tubing: $c_1 = 1.0$, $c_2 = 0.85$ and $c_3 = 0.4$; for concrete encased shapes $c_1 = 0.7, c_2 = 0.6$ and $c_3 = 0.2$

3. Columns with Multiple Steel Shapes

If the composite cross section includes two or more steel shapes, the shapes must be interconnected with lacing, tie plates or batten plates to prevent buckling of individual shapes before hardening of concrete.

4. Load Transfer

The portion of the design strength of axially loaded composite columns resisted by concrete shall be developed by direct bearing at connections. When the supporting concrete area is wider than the loaded area on one or more sides and otherwise restrained against lateral expansion on the remaining sides, the maximum design strength of concrete shall be $1.7\phi_c f'_c A_B$, where $\phi_c = 0.60$ is the resistance factor in bearing on concrete and A_B is the loaded area.

I3. FLEXURAL MEMBERS

1. Effective Width

The portion of the effective width of the concrete slab on each side of the beam center-line shall not exceed:

a. One-eighth of the beam span, center to center of supports;
b. One-half the distance to the centerline of the adjacent beam; or
c. The distance from the beam centerline to the edge of the slab.

2. Strength of Beams with Shear Connectors

The positive design flexural strength $\phi_b M_n$ shall be determined as follows:

a. For $h_c/t_w \leq 640/\sqrt{F_{yf}}$:
$\phi_b = 0.85$; M_n shall be determined from the plastic stress distribution on the composite section.
b. For $h_c/t_w > 640/\sqrt{F_{yf}}$:
$\phi_b = 0.90$; M_n shall be determined from the superposition of elastic stresses, considering the effects of shoring.

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*E_c may be computed from $E_c = w^{1.5}\sqrt{f'_c}$ where w, the unit weight of concrete, is expressed in lbs./cu. ft and f'_c is expressed in ksi.

The negative design flexural strength $\phi_b M_n$ shall be determined for the steel section alone, in accordance with the requirements of Sect. F.

Alternatively, the negative design flexural strength $\phi_b M_n$ may be computed with $\phi_b = 0.85$ and M_n determined from the plastic stress distribution on the composite section, provided that:

a. Steel beam is an adequately braced compact section, as defined in Sect. B5.
b. Shear connectors connect the slab to the steel beam in the negative moment region.
c. Slab reinforcement parallel to the steel beam, within the effective width of the slab, is properly developed.

3. Strength of Concrete-encased Beams

The design flexural strength $\phi_b M_n$ shall be computed with $\phi_b = 0.90$ and M_n determined from the superposition of elastic stresses, considering the effects of shoring.

Alternatively, the design flexural strength $\phi_b M_n$ may be computed with $\phi_b = 0.90$ and M_n determined from the plastic stress distribution on the steel section alone.

4. Strength During Construction

When temporary shores are not used during construction, the steel section alone shall have adequate strength to support all loads applied prior to the concrete attaining 75% of its specified strength f_c'. The design flexural strength of the steel section shall be determined in accordance with the requirements of Sect. F1.

5. Formed Steel Deck

a. General

The design flexural strength $\phi_b M_n$ of composite construction consisting of concrete slabs on formed steel deck connected to steel beams shall be determined by the applicable portions of Sect. I3.2, with the following modifications.

This section is applicable to decks with nominal rib height not greater than 3 in. The average width of concrete rib or haunch w_r shall be not less than 2 in., but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck. See Sect. I3.5c for additional restrictions.

The concrete slab shall be connected to the steel beam with welded stud shear connectors ¾-in. or less in diameter (AWS D1.1). Studs may be welded either through the deck or directly to the steel beam. Stud shear connectors, after installation, shall extend not less than 1½ in. above the top of the steel deck.

The slab thickness above the steel deck shall be not less than 2 in.

b. Deck Ribs Oriented Perpendicular to Steel Beam

Concrete below the top of the steel deck shall be neglected in determining section properties and in calculating A_c for deck ribs oriented perpendicular to the steel beams.

The spacing of stud shear connectors along the length of a supporting beam shall not exceed 32 in.

The nominal strength of a stud shear connector shall be the value stipulated in Sect. I5 multiplied by the following reduction factor:

$$\frac{0.85}{\sqrt{N_r}}(w_r/h_r)\,[(H_s/h_r)-1.0] \leq 1.0 \qquad \text{(I3-1)}$$

where

h_r = nominal rib height, in.
H_s = length of stud connector after welding, in., not to exceed the value $(h_r + 3)$ in computations, although actual length may be greater
N_r = number of stud connectors in one rib at a beam intersection, not to exceed three in computations, although more than three studs may be installed
w_r = average width of concrete rib or haunch (as defined in Sect. I3.5a), in.

To resist uplift, steel deck shall be anchored to all supporting members at a spacing not to exceed 16 in. Such anchorage may be provided by stud connectors, a combination of stud connectors and arc spot (puddle) welds or other devices specified by the designer.

c. Deck Ribs Oriented Parallel to Steel Beam

Concrete below the top of the steel deck may be included in determining section properties and shall be included in calculating A_c for Sect. I5.

Steel deck ribs over supporting beams may be split longitudinally and separated to form a concrete haunch.

When the nominal depth of steel deck is 1½ in. or greater, the average width w_r of the supported haunch or rib shall be not less than 2 in. for the first stud in the transverse row plus 4 stud diameters for each additional stud.

The nominal strength of a stud shear connector shall be the value stipulated in Sect. I5, except that when w_r/h_r is less than 1.5, the value from Sect. I5 shall be multiplied by the following reduction factor:

$$0.6\,(w_r/h_r)\,[(H_s/h_r)-1.0] \leq 1.0 \qquad \text{(I3-2)}$$

where h_r and H_s are as defined in Sect. I3.5b and w_r is the average width of concrete rib or haunch as defined in Sect. I3.5a.

6. Design Shear Strength

The design shear strength of composite beams shall be determined by the shear strength of the steel web, in accordance with the requirements of Sect. F2.

I4. COMBINED COMPRESSION AND FLEXURE

The interaction of axial compression and flexure in the plane of symmetry on composite members shall be limited by Formulas H1-1 through H1-6 with the following modifications:

M_n = nominal flexural strength determined from plastic stress distribution on the composite cross section except as provided below, kip-in.
P_e = $A_s F_{my}/\lambda_c^2$, elastic buckling load, kips
F_{my} = modified yield stress, ksi, see Sect. I2

ϕ_b = resistance factor for flexure from Sect. I3
ϕ_c = 0.85
λ_c = column slenderness parameter defined by Formula E2-4 as modified in Sect. I2.2

When the axial term in Formulas H1-1a and H1-1b is less than 0.3, the nominal flexural strength M_n shall be determined by straight line transition between the nominal flexural strength determined from the plastic distribution on the composite cross sections at $(P_u/\phi_b P_n) = 0.3$ and the flexural strength at $P_u = 0$ as determined from Sect. I3. If shear connectors are required at $P_u = 0$, they shall be provided whenever $P_u/\phi_b P_n$ is less than 0.3.

I5. SHEAR CONNECTORS

This section applies to the design of stud and channel shear connectors. For connectors of other types, see Sect. I6.

1. Materials

Shear connectors shall be headed steel studs not less than four stud diameters in length after installation, or hot rolled steel channels. The stud connectors shall conform to the requirements of Sect. A3.6. The channel connectors shall conform to the requirements of Sect. A3. Shear connectors shall be embedded in concrete slabs made with ASTM C33 aggregate or with rotary kiln produced aggregates conforming to ASTM C330, with concrete unit weight not less than 90 pcf.

2. Horizontal Shear Force

Except for concrete-encased beams as defined in Sect. I1, the entire horizontal shear at the interface between the steel beam and the concrete slab shall be assumed to be transferred by shear connectors. For composite action with concrete subject to flexural compression, the total horizontal shear force between the point of maximum positive moment and the point of zero moment shall be taken as the smallest of: (1) $0.85 f_c' A_c$; (2) $A_s F_y$*; and (3) ΣQ_n;

where

f_c' = specified compressive strength of concrete, ksi
A_c = area of concrete slab within effective width, in.^{2}
A_s = area of steel cross section, in.^{2}
F_y = minimum specified yield stress, ksi
ΣQ_n = sum of nominal strengths of shear connectors between the point of maximum positive moment and the point of zero moment, kips

In continuous composite beams where longitudinal reinforcing steel in the negative moment regions is considered to act compositely with the steel beam, the total horizontal shear force between the point of maximum negative moment and the point of zero moment shall be taken as the smaller of $A_r F_{yr}$ and ΣQ_n;

For hybrid beams, the yield force mus be computed separately for each component of the cross section; $A_s F_y$ of the entire cross section is the sum of the component yield forces.

where

A_r = area of adequately developed longitudinal reinforcing steel within the effective width of the concrete slab, in.^{2}
F_{yr} = minimum specified yield stress of the reinforcing steel, ksi
ΣQ_n = sum of nominal strengths of shear connectors between the point of maximum negative moment and the point of zero moment, kips

3. Strength of Stud Shear Connectors

The nominal strength of one stud shear connector embedded in a solid concrete slab is

$$Q_n = 0.5\, A_{sc} \sqrt{f_c' E_c} \leq A_{sc} F_u \qquad \text{(I5-1)}$$

where

A_{sc} = cross-sectional area of a stud shear connector, in.^{2}
f_c' = specified compressive strength of concrete, ksi
F_u = minimum specified tensile strength of a stud shear connector, ksi
E_c = modulus of elasticity of concrete,* ksi

For a stud shear connector embedded in a slab on a formed steel deck, refer to Sect. I3 for reduction factors given by Formulas I3-1 and I3-2 as applicable. The reduction factors should be applied only to $0.5A_{sc}\sqrt{f_c'E_c}$ term in Formula I5-1.

4. Strength of Channel Shear Connectors

The nominal strength of one channel shear connector embedded in a solid concrete slab is

$$Q_n = 0.3\,(t_f + 0.5\, t_w)\, L_c \sqrt{f_c' E_c} \qquad \text{(I5-2)}$$

where

t_f = flange thickness of channel shear connector, in.
t_w = web thickness of channel shear connector, in.
L_c = length of channel shear connector, in.

5. Required Number of Shear Connectors

The number of shear connectors required between the section of maximum bending moment, positive or negative, and the adjacent section of zero moment shall be equal to the horizontal shear force as determined from Sect. I5.2 divided by the nominal strength of one shear connector as determined from Sect. I5.3 or I5.4.

6. Shear Connector Placement and Spacing

Shear connectors required each side of the point of maximum bending moment, positive or negative, may be distributed uniformly between that point and the adjacent points of zero moment. However, the number of shear connectors placed between any

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*E_c, in ksi, may be computed from $E_c = w^{1.5}\sqrt{f_c'}$ where w, the unit weight of concrete, is expressed in lbs./cu. ft and f_c' is expressed in ksi.

concentrated load and the nearest point of zero moment shall be sufficient to develop the maximum moment required at the concentrated load point.

Except for connectors installed in the ribs of formed steel decks, shear connectors shall have at least 1 in. of lateral concrete cover. Unless located over the web, the diameter of studs shall not be greater than 2.5 times the thickness of the flange to which they are welded. The minimum c.-to-c. spacing of stud connectors shall be 6 diameters along the longitudinal axis of the supporting composite beam and 4 diameters transverse to the longitudinal axis of the supporting composite beam, except that within the ribs of formed steel decks the c.-to-c. spacing may be as small as 4 diameters in any direction. The maximum c.-to-c. spacing of shear connectors shall not exceed 8 times the total slab thickness.

I6. SPECIAL CASES

When composite construction does not conform to the requirements of Sects. I1 through I5, the strength of shear connectors and details of construction shall be established by a suitable test program.

CHAPTER J. CONNECTIONS, JOINTS AND FASTENERS

J1. GENERAL PROVISIONS

1. Design Basis

Connections consist of connecting elements (e.g., stiffeners, gussets, angles, brackets) and connectors (welds, bolts, rivets). These components shall be proportioned so that their design strength equals or exceeds the required strength determined by (a) structural analysis for factored loads acting on the structure or (b) a specified proportion of the strength of the connected members, whichever is appropriate.

2. Simple Connections

Except as otherwise indicated in the design documents, connections of beams, girders or trusses shall be designed as flexible, and may ordinarily be proportioned for the reaction shears only. Flexible beam connections shall accommodate end rotations of unrestrained (simple) beams. To accomplish this, inelastic deformation in the connection is permitted.

3. Moment Connections

End connections of restrained beams, girders and trusses shall be designed for the combined effect of forces resulting from moment and shear induced by the rigidity of the connections.

4. Compression Members with Bearing Joints

When columns bear on bearing plates or are finished to bear at splices, there shall be sufficient connectors to hold all parts securely in place.

When other compression members are finished to bear, the splice material and its connectors shall be arranged to hold all parts in line and shall be proportioned for 50% of the factored strength of the member.

All compression joints shall be proportioned to resist any tension developed by the factored loads specified by Formula A4-6.

5. Minimum Strength of Connections

Except for lacing, sag rods or girts, connections providing design strength shall be designed to support a factored load not less than 10 kips.

6. Placement of Welds and Bolts

Groups of welds or bolts at the ends of any member which transmit axial force into that member shall be sized so that the center of gravity of the group coincides with the center of gravity of the member, unless provision is made for the eccentricity. The foregoing provision is not applicable to end connections of statically-loaded single angle, double angle and similar members.

7. Bolts in Combination with Welds

In new work, A307 bolts or high-strength bolts proportioned as bearing-type connections shall not be considered as sharing the load in combination with welds. Welds, if used, shall be provided to carry the entire force in the connection. High-strength bolts proportioned for slip-critical connections may be considered as sharing the load with the welds.

In making welded alterations to structures, existing rivets and high-strength bolts tightened to the requirements for slip-critical connections may be utilized for carrying loads resulting from existing dead loads, and the welding need only provide the additional design strength required.

8. High-strength Bolts in Combination with Rivets

In both new work and alterations, in connections designed as slip-critical connections in accordance with the provisions of Sect. J3, high-strength bolts may be considered as sharing the load with rivets.

9. Limitations on Bolted and Welded Connections

Fully tensioned high-strength bolts (see Table J3.1), or welds shall be used for the following connections:

- Column splices in all tier structures 200 ft or more in height
- Column splices in tier structures 100 to 200 ft in height, if the least horizontal dimension is less than 40% of the height
- Column splices in tier structures less than 100 ft in height, if the least horizontal dimension is less than 25% of the height
- Connections of all beams and girders to columns and of any other beams and girders on which the bracing of columns is dependent, in structures over 125 ft in height
- In all structures carrying cranes of over 5-ton capacity: roof-truss splices and connections of trusses to columns, column splices, column bracing, knee braces and crane supports
- Connections for supports of running machinery, or of other live loads which produce impact or reversal of stress
- Any other connections stipulated on the design plans.

In all other cases connections may be made with A307 bolts or snug-tight high-strength bolts.

For the purpose of this Section, the height of a tier structure shall be taken as the vertical distance from the curb level to the highest point of the roof beams in the case of flat roofs, or to the mean height of the gable in the case of roofs having a rise of more than 2⅔ in 12. Where the curb level has not been established, or where the structure does not adjoin a street, the mean level of the adjoining land shall be used instead of curb level. Penthouses may be excluded in computing the height of structure.

J2. WELDS

All provisions of the American Welding Society *Structural Welding Code—Steel*, AWS D1.1, except Sects. 2.3.2.4, 2.5, 8.13.1, 9, and 10 as applicable, apply to work performed under this Specification.

1. Groove Welds

a. Effective Area

The effective area of groove welds shall be considered as the effective length of the weld times the effective throat thickness.

The effective length of a groove weld shall be the width of the part joined.

The effective throat thickness of a complete-penetration groove weld shall be the thickness of the thinner part joined.

The effective throat thickness of a partial-penetration groove weld shall be as shown in Table J2.1.

The effective throat thickness of a flare groove weld when flush to the surface of the solid section of the bar shall be as shown in Table J2.2. Random sections of production welds for each welding procedure, or such test sections as may be required by design documents, shall be used to verify that the effective throat is consistently obtained.

Larger effective throat thicknesses than those in Table J2.2 are permitted, provided the fabricator can establish by qualification that he can consistently provide such larger effective throat thicknesses. Qualification shall consist of sectioning the weld normal to its axis, at mid-length and terminal ends. Such sectioning shall be made on a number of combinations of material sizes representative of the range to be used in the fabrication or as required by the designer.

b. Limitations

The minimum effective throat thickness of a partial-penetration groove weld shall be as shown in Table J2.4. Weld size is determined by the thicker of the two parts joined, except that the weld size need not exceed the thickness of the thinnest part joined

TABLE J2.1
Effective Throat Thickness of Partial Penetration Groove Welds

Welding Process	Welding Position	Included Angle at Root of Groove	Effective Throat Thickness
Shielded metal arc Submerged arc	All	J or U joint	Depth of chamfer
		Bevel or V joint $\geq 60°$	
Gas metal arc Flux-cored arc		Bevel or V joint $< 60°$ but $\geq 45°$	Depth of chamfer minus ⅛-in.

TABLE J2.2
Effective Throat Thickness of Flare Groove Welds

Type of Weld	Radius (R) of Bar or Bend	Effective Throat Thickness
Flare bevel groove	All	$\frac{5}{16}R$
Flare V-groove	All	$\frac{1}{2}R$[a]

[a]Use $\frac{3}{8}R$ for Gas Metal Arc Welding (except short circuiting transfer process) when $R \geq 1$ in.

when a larger size is required by calculated strength. For this exception, particular care shall be taken to provide sufficient preheat for soundness of the weld.

2. Fillet Welds

a. Effective Area

The effective area of fillet welds shall be taken as the effective length times the effective throat thickness.

The effective length of fillet welds, except fillet welds in holes and slots, shall be the overall length of full-size fillets, including returns.

The effective throat thickness of a fillet weld shall be the shortest distance from the root of the joint to the face of the diagrammatic weld, except that for fillet welds made by the submerged arc process, the effective throat thickness shall be taken equal to the leg size for 3/8-in. and smaller fillet welds, and equal to the theoretical throat plus 0.11-in. for fillet welds over 3/8-in.

For fillet welds in holes and slots, the effective length shall be the length of the centerline of the weld along the center of the plane through the throat. In the case of overlapping fillets, the effective area shall not exceed the nominal cross-sectional area of the hole or slot, in the plane of the faying surface.

b. Limitations

The *minimum size of fillet welds* shall be as shown in Table J2.5. Minimum weld size is

TABLE J2.4
Minimum Effective Throat Thickness of Partial-penetration Groove Welds

Material Thickness of Thicker Part Joined (in.)	Minimum Effective Throat Thickness[a] (in.)
To 1/4 inclusive	1/8
Over 1/4 to 1/2	3/16
Over 1/2 to 3/4	1/4
Over 3/4 to 1 1/2	5/16
Over 1 1/2 to 2 1/4	3/8
Over 2 1/4 to 6	1/2
Over 6	5/8

[a]See Sect. J2.

TABLE J2.5
Minimum Size of Fillet Welds

Material Thickness of Thicker Part Joined (in.)	Minimum Size of Fillet Weld[a] (in.)
To 1/4 inclusive	1/8
Over 1/4 to 1/2	3/16
Over 1/2 to 3/4	1/4
Over 3/4	5/16

[a]Leg dimension of fillet welds.

determined by the thicker of the two parts joined, except that the weld size need not exceed the thickness of the thinner part. For this exception, particular care shall be taken to provide sufficient preheat for soundness of the weld. Weld sizes larger than the thinner part joined are permitted if required by calculated strength. In the as-welded condition, the distance between the edge of the base metal and the toe of the weld may be less than 1/16-in. provided the weld size is clearly verifiable.

The *maximum size of fillet welds* that may be used along edges of connected parts shall be:

Along edges of material less than 1/4-in. thick, not greater than the thickness of the material.

Along edges of material 1/4-in. or more in thickness, not greater than the thickness of the material minus 1/16-in., unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness.

The *minimum effective length of fillet welds* designed on the basis of strength shall be not less than 4 times the nominal size, or else the size of the weld shall be considered not to exceed 1/4 of its effective length. If longitudinal fillet welds are used alone in end connections of flat bar tension members, the length of each fillet weld shall be not less than the perpendicular distance between them. The transverse spacing of longitudinal fillet welds used in end connections of tension members shall not exceed 8 in., unless the member is designed on the basis of effective net area in accordance with Sect. B3.

Intermittent fillet welds may be used to transfer calculated stress across a joint or faying surfaces when the strength required is less than that developed by a continuous fillet weld of the smallest permitted size, and to join components of built-up members. The effective length of any segment of intermittent fillet welding shall be not less than 4 times the weld size, with a minimum of 1½ in.

In *lap joints*, the minimum amount of lap shall be 5 times the thickness of the thinner part joined, but not less than 1 in. Lap joints joining plates or bars subjected to axial stress shall be fillet welded along the end of both lapped parts, except where the deflection of the lapped parts is sufficiently restrained to prevent opening of the joint under maximum loading.

Side or end fillet welds terminating at ends or sides, respectively, of parts or members shall, wherever practicable, be returned continuously around the corners for a distance not less than 2 times the nominal size of the weld. This provision shall apply to side and top fillet welds connecting brackets, beam seats and similar connections, on the plane about which bending moments are computed. For framing angles and simple end plate connections which depend upon flexibility of the outstanding legs for connection flexibility, end returns shall not exceed four times the nominal size of the weld. Fillet welds which occur on opposite sides of a common plane shall be interrupted at the corner common to both welds. End returns shall be indicated on the design and detail drawings.

Fillet welds in holes or slots may be used to transmit shear in lap joints or to prevent the buckling or separation of lapped parts and to join components of built-up members. Such fillet welds may overlap, subject to the provisions of Sect. J2. Fillet welds in holes or slots are not to be considered plug or slot welds.

3. Plug and Slot Welds

a. Effective Area

The effective shearing area of plug and slot welds shall be considered as the nominal cross-sectional area of the hole or slot in the plane of the faying surface.

b. Limitations

Plug or slot welds may be used to transmit shear in lap joints or to prevent buckling of lapped parts and to join component parts of built-up members.

The diameter of the holes for a plug weld shall be not less than the thickness of the part containing it plus 5⁄16-in., rounded to the next larger odd 1⁄16-in., nor greater than 2¼ times the thickness of the weld metal.

The minimum c.-to-c. spacing of plug welds shall be four times the diameter of the hole.

The length of slot for a slot weld shall not exceed 10 times the thickness of the weld. The width of the slot shall be not less than the thickness of the part containing it plus 5⁄16-in., rounded to the next larger odd 1⁄16-in., nor shall it be larger than 2¼ times the thickness of the weld. The ends of the slot shall be semicircular or shall have the corners rounded to a radius not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

The minimum spacing of lines of slot welds in a direction transverse to their length shall be 4 times the width of the slot. The minimum c.-to-c. spacing in a longitudinal direction on any line shall be 2 times the length of the slot.

The thickness of plug or slot welds in material ⅝-in. or less in thickness shall be equal to the thickness of the material. In material over ⅝-in. in thickness, the thickness of the weld shall be at least ½ the thickness of the material but not less than ⅝-in.

4. Design Strength

The design strength of welds shall be the lower value of ϕF_{BM} and ϕF_w, when applicable, where F_{BM} and F_w are the nominal strengths of the base material and the weld electrode material, respectively. The values of ϕ, F_{BM} and F_w and limitations thereon are given in Table J2.3.

5. Combination of Welds

If two or more of the general types of welds (groove, fillet, plug, slot) are combined in a single joint, the design strength of each shall be separately computed with reference to the axis of the group in order to determine the design strength of the combination.

6. Matching Steel

The choice of electrode for use with complete-penetration groove welds subject to tension normal to the effective area is dictated by the requirements for matching steels given in the AWS *Structural Welding Code—Steel* D1.1.

J3. BOLTS, THREADED PARTS AND RIVETS

1. High-strength Bolts

Except as otherwise provided in this Specification, use of high-strength bolts shall conform to the provisions of the *Specification for Structural Joints Using ASTM A325 or A490 Bolts*—1985, as approved by the Research Council on Structural Connections.

If required to be tightened to more than 50% of their minimum specified tensile strength, ASTM A449 bolts in tension and bearing-type shear connections shall have an ASTM F436 hardened washer installed under the bolt head, and the nuts shall meet

TABLE J2.3
Design Strength of Welds

Types of Weld and Stress[a]	Material	Resistance Factor ϕ	Nominal strength F_{BM} or F_w	Required Weld strength level[b,c]
	Complete Penetration Groove Weld			
Tension normal to effective area	Base	0.90	F_y	"Matching" weld must be used.
Compression normal to effective area	Base	0.90	F_y	Weld metal with a strength level equal to or less than "matching" may be used.
Tension or compression parallel to axis of weld				
Shear on effective area	Base Weld electrode	0.90 0.80	$0.60F_y$ $0.60F_{EXX}$	
	Partial Penetration Groove Welds			
Compression normal to effective area	Base	0.90	F_y	Weld metal with a strength level equal to or less than "matching" weld metal may be used.
Tension or compression parallel to axis of weld[d]				
Shear parallel to axis of weld	Base[e] Weld electrode	0.75	$0.60F_{EXX}$	
Tension normal to effective area	Base Weld Electrode	0.90 0.80	F_y $0.60F_{EXX}$	
	Fillet Welds			
Stress on effective area	Base[e] Weld electrode	0.75	$0.60F_{EXX}$	Weld metal with a strength level equal to or less than "matching" weld metal may be used.
Tension or compression parallel to axis of weld[d]	Base	0.90	F_y	
	Plug or Slot Welds			
Shear parallel to faying surfaces (on effective area)	Base[e] Weld Electrode	0.75	$0.60F_{EXX}$	Weld metal with a strength level equal to or less than "matching" weld metal may be used.

[a]For definition of effective area, see Sect. J2.
[b]For "matching" weld metal, see Table 4.1.1, AWS D1.1.
[c]Weld metal one strength level stronger than "matching" weld metal will be permitted.
[d]Fillet welds and partial-penetration groove welds joining component elements of built-up members, such as flange-to-web connections, may be designed without regard to the tensile or compressive stress in these elements parallel to the axis of the welds.
[e]The design of connected material is governed by Sect. J4.

TABLE J3.1
Minimum Bolt Tension, kips[a]

Bolt Size, in.	A325 Bolts	A490 Bolts
½	12	15
⅝	19	24
¾	28	35
⅞	39	49
1	51	64
1⅛	56	80
1¼	71	102
1⅜	85	121
1½	103	148

[a]Equal to 0.70 of minimum tensile strength of bolts, rounded off to nearest kip, as specified in ASTM specifications for A325 and A490 bolts with UNC threads.

the requirements of ASTM A563. When assembled, all joint surfaces, including those adjacent to the washers, shall be free of scale, except tight mill scale. Except as noted below, all A325 and A490 bolts shall be tightened to a bolt tension not less than that given in Table J3.1. Tightening shall be done by the turn-of-nut method, a direct tension indicator or by calibrated wrench.

Bolts in connections not subject to tension loads, where slip can be permitted and where loosening or fatigue due to vibration or load fluctuations are not design considerations, need only to be tightened to the snug-tight condition. The snug-tight condition is defined as the tightness attained by a few impacts of an impact wrench or the full effort of a worker with an ordinary spud wrench and must bring the connected plies into firm contact. The nominal strength value given in Table J3.2 for bearing-type connections shall be used for bolts tightened to the snug-tight condition. Bolts to be tightened only to the snug-tight condition shall be clearly identified on the design and erection drawings.

2. Effective Bearing Area

The effective bearing area of bolts, threaded parts and rivets shall be the diameter multiplied by the length in bearing, except that for countersunk bolts and rivets ½ the depth of the countersink shall be deducted.

3. Design Tension or Shear Strength

The design strength of bolts and threaded parts shall be taken as the product of the resistance factor ϕ and the nominal strength given in Table J3.2 of the unthreaded nominal body area of bolts and threaded parts other than upset rods (see footnote c, Table J3.2). High-strength bolts required to support the applied load by means of direct tension shall be proportioned so that their average required strength, computed on the basis of nominal bolt area and independent of any initial tightening force, will not exceed the design strength. The applied load shall be the sum of the factored external loads and any tension resulting from prying action produced by deformation of the connected parts.

TABLE J3.2
Design Strength of Fasteners

Description of Fasteners	Tensile Strength: Resistance Factor ϕ	Tensile Strength: Nominal Strength, ksi	Shear Strength in Bearing-type Connections: Resistance Factor ϕ	Shear Strength in Bearing-type Connections: Nominal Strength ksi
A307 bolts	0.75	45.0[a]	0.60	27.0[b,e]
A325 bolts, when threads are *not* excluded from shear planes	0.75	90.0[d]	0.65	54.0[e]
A325 bolts, when threads *are* excluded from shear planes		90.0[d]		72.0[e]
A490 bolts, when threads are *not* excluded from shear planes		112.5[d]		67.5[e]
A490 bolts, when threads *are* excluded from the shear planes		112.5[d]		90.0[e]
Threaded parts meeting the requirements of Sect. A3, when threads are *not* excluded from the shear planes		$0.75F_u$[a,c]		$0.45F_u$
Threaded parts meeting the requirements of Sect. A3, when threads *are* excluded from the shear planes		$0.75F_u$[a,c]		$0.60F_u$
A502, Gr. 1, hot-driven rivets		45.0[a]		36.0[e]
A502, Gr. 2 & 3, hot-driven rivets		60.0[a]		48.0[e]

[a]Static loading only.
[b]Threads permitted in shear planes.
[c]The nominal tensile strength of the threaded portion of an upset rod, based upon the cross-sectional area at its major thread diameter, A_b shall be larger than the nominal body area of the rod before upsetting times F_y.
[d]For A325 and A490 bolts subject to tensile fatigue loading, see Appendix K4.
[e]When bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 50 in., tabulated values shall be reduced by 20%.

TABLE J3.3
Tension Stress Limit (F_t), ksi, for Fasteners in Bearing-type Connections

Description of Fasteners	Threads Included in the Shear Plane	Threads Excluded from the Shear Plane
A307 bolts	$39 - 1.8f_v \leq 30$	
A325 bolts	$85 - 1.8f_v \leq 68$	$85 - 1.4f_v \leq 68$
A490 bolts	$106 - 1.8f_v \leq 84$	$106 - 1.4f_v \leq 84$
Threaded parts A449 bolts over 1½-in. diameter	$0.73F_u - 1.8f_v \leq 0.56F_u$	$0.73F_u - 1.4f_v \leq 0.56F_u$
A502 Gr. 1 rivets	$44 - 1.3f_v \leq 34$	
A502 Gr. 2 rivets	$59 - 1.3f_v \leq 45$	

4. Combined Tension and Shear in Bearing-type Connections

Bolts and rivets subject to combined tension and shear shall be so proportioned that the tension stress f_t produced by factored loads on the nominal body area A_b does not exceed the values computed from the formulas in Table J3.3. The value of f_v, the shear produced by the same factored loads, shall not exceed the values for shear given in Sect. J3.3.

5. High-strength Bolts in Slip-critical Joints

The design shear resistance of slip-critical joints shall be determined by using the tabulated values from Table J3.4 multiplied by $\phi = 1.0$, except $\phi = 0.85$ for the long-slotted holes when the load is in the direction of the slot. The shear on the bolt due to service loads shall be less than the tabulated values. When the loading combination includes wind or seismic loads in addition to dead and live loads, the total of the combined load effects, at service loads, may be multiplied by 0.75 in accordance with ANSI 58.1.

When specified by the designer, the nominal slip resistance for connections having special faying surface conditions may be increased to the applicable values in RCSC Load and Resistance Factor Design Specification.

When a bolt in a slip-critical connection is subjected to a service tensile force T, the nominal resistance in Table J3.4 shall be multiplied by the reduction factor $(1 - T/T_b)$ where T_b is the minimum pretension load from Table J3.1.

6. Bearing Strength at Bolt Holes

When L is not less than $1\frac{1}{2}\,d$ and the distance center to center of bolts is not less than

TABLE J3.4
Nominal Slip-critical Shear Strength, ksi, of High-strength Bolts[a]

Type of Bolt	Nominal Shear Strength		
	Standard Size Holes	Oversized and Short-slotted Holes	Long-slotted Holes[b]
A325	17	15	12
A490	21	18	15

[a]Class A (slip coefficient 0.33). Clean mill scale and blast cleaned surfaces with class A coatings. For design strengths with other coatings see RCSC "Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts."
[b]Tabulated values are for the case of load application transverse to the slot. When the load is parallel to the slot multiply tabulated values by 0.85.

$3d$, the design bearing strength on two or more bolts in the line of force is ϕR_n, where $\phi = 0.75$.

In standard or short-slotted holes,

$$R_n = 2.4\, dt\, F_u \tag{J3-1a}$$

In long-slotted holes perpendicular to the load,

$$R_n = 2.0\, dt\, F_u \tag{J3-1b}$$

For the bolt closest to the edge, in all connections not covered by Formulas J3-1a and J3-1b, the design bearing of a single bolt, or two or more bolts in line of force, each with an end distance less than 1½ d, shall be determined by ϕR_n, where $\phi = 0.75$.

$$R_n = Lt\, F_u \tag{J3-1c}$$

If deformation around the bolt hole is not a design consideration (see Commentary) and adequate spacing and edge distance as required by Sect. J3.9 and J3.10 is provided, the following formula may be used in lieu of J3-1a and J3-1b, where $\phi = 0.75$.

$$R_n = 3.0\, dt\, F_u \tag{J3-1d}$$

where

d = nominal dia. of bolt, in.
t = thickness of connected part, in.
F_u = specified tensile strength of connected part, ksi
L = distance in line of force from the center of a standard or oversized hole or from the center of the end of a slotted hole to an edge of a connected part, in.

7. Size and Use of Holes

a. The *maximum sizes* of holes for rivets and bolts are given in Table J3.5, except that larger holes, required for tolerance on location of anchor bolts in concrete foundations, may be used in column base details.

TABLE J3.5
Nominal Hole Dimensions

Bolt Dia.	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-slot (Width × length)	Long-slot (Width × length)
½	9/16	5/8	9/16 × 11/16	9/16 × 1¼
5/8	11/16	13/16	11/16 × 7/8	11/16 × 1 9/16
¾	13/16	15/16	13/16 × 1	13/16 × 1 7/8
7/8	15/16	1 1/16	15/16 × 1 1/8	15/16 × 2 3/16
1	1 1/16	1¼	1 1/16 × 1 5/16	1 1/16 × 2½
≥1 1/8	d + 1/16	d + 5/16	(d + 1/16) × (d + 3/8)	(d + 1/16) × (2.5 × d)

b. *Standard holes* shall be provided in member-to-member connections, unless oversized, short-slotted or long-slotted holes in bolted connections are approved by the designer. Finger shims up to ¼-in. may be introduced into slip-critical connections designed on the basis of standard holes without reducing the nominal shear strength of the fastener to that specified for slotted holes.

c. *Oversized holes* may be used in any or all plies of slip-critical connections, but they shall not be used in bearing-type connections. Hardened washers shall be installed over oversized holes in an outer ply.

d. *Short-slotted holes* may be used in any or all plies of slip-critical or bearing-type connections. The slots may be used without regard to direction of loading in slip-critical connections, but the length shall be normal to the direction of the load in bearing-type connections. Washers shall be installed over short-slotted holes in an outer ply; when high-strength bolts are used, such washers shall be hardened.

e. *Long-slotted holes* may be used in only one of the connected parts of either a slip-critical or bearing-type connection at an individual faying surface. Long-slotted holes may be used without regard to direction of loading in slip-critical connections, but shall be normal to the direction of load in bearing-type connections. Where long-slotted holes are used in an outer ply, plate washers or a continuous bar with standard holes, having a size sufficient to completely cover the slot after installation, shall be provided. In high-strength bolted connections, such plate washers or continuous bars shall be not less than 5/16-in. thick and shall be of structural grade material, but need not be hardened. If hardened washers are required for use of high-strength bolts, the hardened washers shall be placed over the outer surface of the plate washer or bar.

f. When A490 bolts over 1 in. in diameter are used in slotted or oversize holes in external plies, a single hardened washer conforming to ASTM F436, except with 5/16-in. minimum thickness, shall be used in lieu of the standard washer.

8. Long Grips

A307 bolts providing design strength, and for which the grip exceeds five diameters, shall have their number increased 1% for each additional 1/16-in. in the grip.

TABLE J3.6
Values of Spacing Increment C_1, in.

Nominal Dia. of Fastener	Oversize Holes	Slotted Holes		
		Perpendicular to Line of Force	Parallel to Line of Force	
			Short-slots	Long-slots[a]
$\leq 7/8$	$1/8$	0	$3/16$	$1\frac{1}{2}d - 1/16$
1	$3/16$	0	$1/4$	$1\frac{7}{16}$
$\geq 1\frac{1}{8}$	$1/4$	0	$5/16$	$1\frac{1}{2}d - 1/16$

[a]When length of slot is less than maximum allowed in Table J3.5, C_1 may be reduced by the difference between the maximum and actual slot lengths.

9. Minimum Spacing

The distance between centers of standard, oversized or slotted fastener holes shall not be less than $2\frac{2}{3}$ times the nominal diameter of the fastener* nor less than that required by the following paragraph, if applicable.

Along a line of transmitted forces, the distance between centers of holes shall be not less than $3d$ when R_n is determined by J3-1a and J3-1b. Otherwise the distance between centers of holes shall be not less than the following:

a. For standard holes:

$$\frac{P}{\phi F_u t} + \frac{d_h}{2} \tag{J3-2}$$

where

$\phi = 0.75$
P = force transmitted by one fastener to the critical connected part, kips
F_u = specified minimum tensile strength of the critical connected part, ksi
t = thickness of the critical connected part, in.
d_h = diameter of standard size hole, in.

b. For oversized and slotted holes, the distance required for standard holes in subparagraph a, above, plus the applicable increment C_1 from Table J3.6, but the clear distance between holes shall not be less than one bolt diameter.

*A distance of $3d$ is preferred.

TABLE J3.7
Minimum Edge Distance, in.
(Center of Standard Hole[a] to Edge of Connected Part)

Nominal Rivet or Bolt Diameter (in.)	At Sheared Edges	At Rolled Edges of Plates, Shapes or Bars or Gas Cut Edges[b]
½	⅞	¾
⅝	1⅛	⅞
¾	1¼	1
⅞	1½[c]	1⅛
1	1¾[c]	1¼
1⅛	2	1½
1¼	2¼	1⅝
Over 1¼	1¾ × Diameter	1¼ × Diameter

[a]For oversized or slotted holes, see Table J3.8.
[b]All edge distances in this column may be reduced ⅛-in. when the hole is at a point where stress does not exceed 25% of the maximum design strength in the element.
[c]These may be 1¼ in. at the ends of beam connection angles.

10. Minimum Edge Distance

The distance from the center of a standard hole to an edge of a connected part shall be not less than the applicable value from Table J3.7 nor the value from Formula J3-3, as applicable.

Along a line of transmitted force, in the direction of the force, the distance from the center of a standard hole to the edge of the connected part shall be not less than $1\frac{1}{2}d$ when R_n is determined by J3-1a or J3-1b. Otherwise the edge distance shall be not less than

$$\frac{P}{\phi F_u t} \qquad \text{(J3-3)}$$

where ϕ, P, F_u, t are defined in Sect. J3.9.

The distance from the center of an oversized or slotted hole to an edge of a connected part shall be not less than that required for a standard hole plus the applicable increment C_2 from Table J3.8.

11. Maximum Edge Distance and Spacing

The maximum distance from the center of any rivet or bolt to the nearest edge of parts in contact shall be 12 times the thickness of the connected part under consideration, but shall not exceed 6 in. Bolted joints in unpainted steel exposed to atmospheric corrosion require special limitations on pitch and edge distance, see Sect. E4.

J4. DESIGN SHEAR RUPTURE STRENGTH

The design strength for the limit state of rupture along a shear failure path in main members shall be taken as $\phi F_n A_{ns}$

TABLE J3.8
Values of Edge Distance Increment C_2, in.

Nominal Diameter of Fastener (in.)	Oversized Holes	Slotted Holes		
		Perpendicular to Edge		Parallel to Edge
		Short Slots	Long Slots[a]	
≤ 7/8	1/16	1/8	3/4d	0
1	1/8	1/8		
≤ 1 1/8	1/8	3/16		

[a]When length of slot is less than maximum allowable (see Table J3.5), C_2 may be reduced by one-half the difference between the maximum and actual slot lengths.

where

$\phi = 0.75$
$F_n = 0.6\,F_u$ (J4-1)
A_{ns} = net area subject to shear.

J5. CONNECTING ELEMENTS

This section applies to the design of connecting elements, such as stiffeners, gussets, angles and brackets and the panel zones of beam-to-column connections.

1. Eccentric Connections

Intersecting axially stressed members shall have their gravity axes intersect at one point, if practicable; if not, provision shall be made for bending and shearing stresses due to the eccentricity.

2. Design Strength of Connecting Elements

The design strength ϕR_n of welded, bolted and riveted connecting elements statically loaded in tension (e.g., splice and gusset plates) shall be the lower value obtained according to the limit states of yielding, fracture of the connecting element and block shear rupture.

a. For yielding of the connecting element:

$$\phi = 0.90$$
$$R_n = A_g F_y \qquad \text{(J5-1)}$$

b. For fracture of the connecting element where $A_n \leq 0.85A_g$:

$$\phi = 0.75$$
$$R_n = A_n F_u \qquad \text{(J5-2)}$$

c. For block shear rupture

Block shear is a failure mode in which the resistance is determined by the sum of the shear strength on a failure path(s) and the tensile strength on a perpendicular segment. When ultimate strength on the net section is used to determine the resistance on one segment, yielding on the gross section shall be used on the perpendicular segment; $\phi = 0.75$. Design strength shall be the larger of the two failure modes.

At beam end connections where the top flange is coped, and in similar situations, failure can occur by shear along a plane through the fasteners, acting in combination with tension along a perpendicular plane. In such cases, the ultimate strength on the net section (shear or tension) shall be used to determine the resistance of one segment and yielding on the gross section (shear or tension) shall be used for the perpendicular segment, with $\phi = 0.75$ for both. By alternating the choice of which segment resistance is based on ultimate strength, two possible values of design strength are obtained. The larger value shall be taken as the design strength.

For all other connecting elements, the design strength ϕR_n shall be determined for the applicable limit state to insure that the design strength is equal to or greater than the required strength, where R_n is the nominal strength appropriate to the geometry and type of loading on the connecting element. The shear limit state is governed by:

$$\begin{aligned} \phi &= 0.80 \\ R_n &= 0.7 A_g F_y \end{aligned} \tag{J5-3}$$

J6. FILLERS

In welded construction, any filler ¼-in. or more in thickness shall extend beyond the edges of the splice plate and shall be welded to the part on which it is fitted with sufficient weld to transmit the splice plate load, applied at the surface of the filler. The welds joining the splice plate to the filler shall be sufficient to transmit the splice plate load and shall be long enough to avoid overloading the filler along the toe of the weld. Any filler less than ¼-in. thick shall have its edges made flush with the edges of the splice plate and the weld size shall be the sum of the size necessary to carry the splice plus the thickness of the filler plate.

When bolts or rivets carrying loads pass through fillers thicker than ¼-in., except in connections designed as slip-critical connections, the fillers shall be extended beyond the splice material and the filler extension shall be secured by enough bolts or rivets to distribute the total stress in the member uniformly over the combined section of the member and the filler, or an equivalent number of fasteners shall be included in the connection.

J7. SPLICES

Groove-welded splices in plate girders and beams shall develop the full strength of the smaller spliced section. Other types of splices in cross sections of plate girders and beams shall develop the strength required by the forces at the point of splice.

J8. BEARING STRENGTH

The strength of surfaces in bearing is ϕR_n, where $\phi = 0.75$ and R_n is defined below for various types of bearing.

1. Milled or Finished Surfaces

For milled surfaces, pins in reamed, drilled or bored holes, and ends of fitted bearing stiffeners,

$$R_n = 2.0\ F_y\ A_{pb} \tag{J8-1}$$

where

F_y = specified minimum yield stress, ksi
A_{pb} = projected bearing area, in.^{2}

2. Expansion Rollers and Rockers

For expansion rollers and rockers,

$$R_n = 1.5\ (F_y - 13)\ell d/20 \tag{J8-2}$$

where

d = diameter, in.
ℓ = length of bearing, in.

J9. COLUMN BASES AND BEARING ON CONCRETE

Proper provision shall be made to transfer the column loads and moments to the footings and foundations.

In the absence of code regulations, design bearing loads on concrete may be taken as $\phi_c P_p$:

On the full area of a concrete support .... $P_p = 0.85 f_c' A_1$
On less than the full area of a concrete support .... $P_p = 0.85 f_c' A_1 \sqrt{A_2/A_1}$

where

ϕ_c = 0.60
A_1 = area of steel concentrically bearing on a concrete support, in.
A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2}

$\sqrt{A_2/A_1} \leq 2$

J10. ANCHOR BOLTS AND EMBEDMENTS

1. Anchor Bolts

Anchor bolts shall be designed to provide resistance to all design conditions on completed structures of tension and shear at the bases of columns, including the net tensile components of any bending moment which may result from column base restraint.

2. Embedments

The concrete structure shall be designed to safely support the loads from the embedment with an appropriate factor of safety to ensure that the embedment strength is not reduced as a result of local or gross failure of the supporting concrete structure.

The strength and design of the structural steel elements of the embedment shall be in accordance with this Specification. Bolts, studs and bars functioning as embedment anchors resisting tensile loads shall be designed to transfer the design load to the concrete by means of bond, shear, bearing or a combination thereof.

Shear loads shall be considered to be transmitted by the embedment to the concrete by either shear lugs or shear friction.

The friction force V, kips, to resist shear shall be computed as:

$$V = \mu P \tag{J10-1}$$

where

P = normal force, kips
μ = coefficient of friction

The coefficient of friction μ shall be 0.90 for concrete placed against as-rolled steel with contact plane a full plate thickness below the concrete surface; 0.70 for concrete or grout placed against as-rolled steel with contact plane coincidental with the concrete surface; 0.55 for grouted conditions with the contact plane between grout and as-rolled steel above the concrete surface.

3. Prestressed Embedments

Anchorage to concrete structures by means of post-tensioned high-strength steel members is permissible. The material and the design requirements of the high-strength steel members and associated anchorage, as well as the fabrication and installation procedures, shall conform to the appropriate provisions of applicable codes.

CHAPTER K. STRENGTH DESIGN CONSIDERATIONS

This chapter covers additional member strength design considerations related to introduction of concentrated forces, ponding, torsion, and fatigue.

K1. WEBS AND FLANGES WITH CONCENTRATED FORCES

1. Design Basis

Members with concentrated loads applied normal to *one flange* and symmetric to the web shall have a flange and web design strength sufficient to satisfy the local flange bending, web yielding strength, web crippling and sidesway web buckling criteria of Sects. K1.2, K1.3, K1.4 and K1.5. Members with concentrated loads applied to *both flanges* shall have a web design strength sufficient to satisfy the web yielding, web crippling and column web buckling criteria of Sects. K1.3, K1.4 and K1.6.

Where pairs of stiffeners are provided on opposite sides of the web, at concentrated loads, and extend at least half the depth of the member, Sects K1.2 and K1.3 need not be checked.

For column webs subject to high shears, see Sect. K1.7; for bearing stiffeners, see Sect. K1.8.

2. Local Flange Bending

The flange design strength in bending due to a tensile load shall be ϕR_n, kips

where

$$\begin{aligned} \phi &= 0.90 \\ R_n &= 6.25 t_f^2 F_{yf} \end{aligned} \qquad \text{(K1-1)}$$

F_{yf} = specified minimum yield stress of flange, ksi
t_f = thickness of the loaded flange, in.

If the length of loading measured across the member flange is less than $0.15b$, where b is the member flange width, Formula K1-1 need not be checked.

3. Local Web Yielding

The design strength of the web at the toe of the fillet under concentrated loads shall be ϕR_n, kips, where $\phi = 1.0$ and R_n is determined as follows:

a. When the force to be resisted is a concentrated load producing tension or compression, applied at a distance from the member end that is greater than the depth of the member,

$$R_n = (5k + N) F_{yw} t_w \qquad \text{(K1-2)}$$

b. When the force to be resisted is a concentrated load applied at or near the end of the member,

$$R_n = (2.5k + N) F_{yw} t_w \qquad \text{(K1-3)}$$

where

F_{yw} = specified minimum yield stress of the web, ksi
N = length of bearing, in.
k = distance from outer face of flange to web toe of fillet, in.
t_w = web thickness, in.

4. Web Crippling

For unstiffened portions of webs of members under concentrated loads, the design compressive strength shall be ϕR_n, kips, where $\phi = 0.75$ and the nominal strength R_n is determined as follows:

a. When the concentrated load is applied at a distance not less than $d/2$ from the end of the member:

$$R_n = 135t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{F_{yw}t_f/t_w} \tag{K1-4}$$

b. When the concentrated load is applied less than a distance $d/2$ from the end of the member:

$$R_n = 68t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{F_{yw}t_f/t_w} \tag{K1-5}$$

where

d = overall depth of the member, in.
t_f = flange thickness, in.

If stiffeners are provided and extend at least one-half the web depth, Formulas K1-4 and K1-5 need not be checked.

5. Sidesway Web Buckling

For webs of members with flanges not restrained against relative movement by stiffeners or lateral bracing and subject to concentrated compressive loads, the design compressive strength shall be ϕR_n, kips, where $\phi = 0.85$ and the nominal strength R_n is determined as follows:

a. If the loaded flange is restrained against rotation and $(d_c/t_w)/(\ell/b_f)$ is less than 2.3:

$$R_n = \frac{12{,}000t_w^3}{h}\left[1 + 0.4\left(\frac{d_c/t_w}{\ell/b_f}\right)^3\right] \tag{K1-6}$$

b. If the loaded flange is *not* restrained against rotation and $(d_c/t_w)/(\ell/b_f)$ is less than 1.7:

$$R_n = \frac{12{,}000t_w^3}{h}\left[0.4\left(\frac{d_c/t_w}{\ell/b_f}\right)^3\right] \tag{K1-7}$$

where

ℓ = largest laterally unbraced length along either flange at the point of load, in.
b_f = flange width, in.
t_w = web thickness, in.
$d_c = d - 2k$ = web depth clear of fillets, in.

Formulas K1-6 and K1-7 need not be checked providing $(d_c/t_w)/(\ell/b_f)$ exceeds 2.3 or 1.7, respectively, or for webs subject to distributed load.

If a concentrated load is located at a point where the web flexural stress due to factored load is below yielding, 24,000 may be used in lieu of 12,000 in Formulas K1-6 and K1-7.

6. Compression Buckling of the Web

For unstiffened portions of webs of members under concentrated loads to both flanges, the design compressive strength shall be ϕR_n, kips, where

$$\phi = 0.90$$

$$R_n = \frac{4{,}100\, t_w^3 \sqrt{F_{yw}}}{d_c} \qquad \text{(K1-8)}$$

R_n may be exceeded provided that a transverse stiffener or pair of stiffeners is attached to the web to satisfy Sect. F3.

7. Compression Members with Web Panels Subject to High Shear

For compression members subject to high shear stress in the web, the design web shear strength shall be ϕR_v, kips, where $\phi = 0.90$ and R_v is determined as follows:

a. For $P_u \leq 0.75 P_n$:

$$R_v = 0.7\, F_y d_c t_w \qquad \text{(K1-9)}$$

b. For $P_u > 0.75 P_n$:

$$R_v = 0.7\, F_y d_c t_w\, [1.9 - 1.2(P_u/P_n)] \qquad \text{(K1-10)}$$

where

P_u = required axial strength, kips
P_n = nominal axial strength, kips

8. Stiffener Requirements for Concentrated Loads

When required, stiffeners shall be placed in pairs at unframed ends of beams and girders. They shall be placed in pairs at points of concentrated load on the interior of beams, girders or columns if the load exceeds the nominal strength ϕR_n as determined from Sects. K1.2 through K1.6, as applicable.

If the concentrated load, tension or compression exceeds the criteria for ϕR_n of Sects. K1.2 or K1.3, respectively, stiffeners need not extend more than one-half the depth of the web, except as follows:

If concentrated compressive loads are applied to the members and if the load exceeds the compressive strength of the web ϕR_n given in Sects. K1.4 or K1.6, the stiffeners shall be designed as axially compressed members (columns) in accordance with requirements of Sect. E2 with an effective length equal to $0.75h$, a cross section composed of two stiffeners and a strip of the web having a width of $25t_w$ at interior stiffeners and $12t_w$ at the ends of members.

When the load normal to the flange is tensile, the stiffeners shall be welded to the loaded flange. When the load normal to the flange is compressive, the stiffeners shall either bear on or be welded to the loaded flange.

K2. PONDING

The roof system shall be investigated by structural analysis to assure adequate strength and stability under ponding conditions, unless the roof surface is provided with sufficient slope toward points of free drainage or adequate individual drains to prevent the accumulation of rainwater.

The roof system shall be considered stable and no further investigation is needed if:

$$C_p + 0.9C_s \leq 0.25 \quad \text{(K2-1)}$$

$$\text{and } I_d \geq 25\ (S^4)10^{-6} \quad \text{(K2-2)}$$

where

$$C_p = \frac{32L_s L_p^4}{10^7 I_p}$$

$$C_s = \frac{32SL_s^4}{10^7 I_s}$$

L_p = column spacing in direction of girder (length of primary members), ft
L_s = column spacing perpendicular to direction of girder (length of secondary members), ft
S = spacing of secondary members, ft
I_p = moment of inertia of primary members, in.^{4}
I_s = moment of inertia of secondary members, in.^{4}
I_d = moment of inertia of the steel deck supported on secondary members, in.^{4} per ft

For trusses and steel joists, the moment of inertia I_s shall be decreased 15% when used in the above equation. A steel deck shall be considered a secondary member when it is directly supported by the primary members.

See Appendix K2 for an alternate determination of flat roof framing stiffness.

K3. TORSION

For limiting values of normal and shear stress, due to torsion and other loading, see Sect. H2. Some constrained local yielding may be permitted.

K4. FATIGUE

Few members or connections in conventional buildings need to be designed for fatigue, since most load changes in such structures occur only a small number of times or produce only minor stress fluctuations. The occurrence of full design wind or earthquake loads is too infrequent to warrant consideration in fatigue design. However, crane runways and supporting structures for machinery and equipment are often subject to fatigue loading conditions.

Members and their connections subject to fatigue loading shall be proportioned in accordance with the provisions of Appendix K4 for service loads.

CHAPTER L.
SERVICEABILITY DESIGN CONSIDERATIONS

This chapter is intended to provide design guidance for serviceability considerations not covered elsewhere. Serviceability is a state in which the function of a building, its appearance, maintainability, durability and comfort of its occupants are preserved under normal usage.

The general design requirement for serviceability is given in Sect. A5.4. Limiting values of structural behavior to ensure serviceability (e.g., maximum deflections, accelerations, etc.) shall be chosen with due regard to the intended function of the structure.

Where necessary, serviceability shall be checked using realistic loads for the appropriate serviceability limit state.

L1. CAMBER

If any special camber requirements are necessary to bring a loaded member into proper relation with the work of other trades, as for the attachment of runs of sash, the requirements shall be set forth in the design documents.

Beams and trusses detailed without specified camber shall be fabricated so that after erection any camber due to rolling or shop assembly shall be upward. If camber involves the erection of any member under a preload, this shall be noted in the design documents.

L2. EXPANSION AND CONTRACTION

Adequate provision shall be made for expansion and contraction appropriate to the service conditions of the structure.

L3. DEFLECTIONS, VIBRATION AND DRIFT

1. Deflections

Deformations in structural members and structural systems due to service loads shall not impair the serviceability of the structure.

2. Vibration

Vibration shall be considered in designing beams and girders supporting large areas free of partitions or other sources of damping, where vibration due to pedestrian traffic or other sources within the building might not be acceptable.

3. Drift

Lateral deflection or drift of structures due to code-specified wind or seismic loads shall not cause collision with adjacent structures nor exceed the limiting values of such drifts which may be specified or appropriate.

L4. CONNECTION SLIP

For the design of slip-resistant connections see Sect. J3.5.

L5. CORROSION

When appropriate, structural components shall be designed to tolerate corrosion or shall be protected against corrosion that may impair the strength or serviceability of the structure.

CHAPTER M. FABRICATION, ERECTION AND QUALITY CONTROL

M1. SHOP DRAWINGS

Shop drawings giving complete information necessary for the fabrication of th component parts of the structure, including the location, type and size of all weld bolts and rivets, shall be prepared in advance of the actual fabrication. These drawing shall clearly distinguish between shop and field welds and bolts and shall clearl identify slip-critical high-strength bolted connections.

Shop drawings shall be made in conformity with the best practice and with du regard to speed and economy in fabrication and erection.

M2. FABRICATION

1. Cambering, Curving and Straightening

Local application of heat or mechanical means may be used to introduce or correc camber, curvature and straightness. The temperature of heated areas, as measured b approved methods, shall not exceed 1,100°F for A514 steel nor 1,200°F for othe steels.

2. Thermal Cutting

Thermal cutting shall preferably be done by machine. Thermally cut edges which will be subjected to substantial stress, or which are to have weld metal deposited on them, shall be reasonably free from notches or gouges; notches or gouges not more than $3/16$-in. deep will be permitted. Notches or gouges greater than $3/16$-in. deep that remain from cutting shall be removed by grinding or repaired by welding. All re-entrant corners shall be shaped to provide a smooth transition. If a specific contour is required, it must be shown in the design documents.

3. Planing of Edges

Planing or finishing of sheared or thermally cut edges of plates or shapes will not be required unless specifically called for in the design documents or included in a stipulated edge preparation for welding.

4. Welded Construction

The technique of welding, the workmanship, appearance and quality of welds and the methods used in correcting nonconforming work shall be in accordance with "Sect. 3—Workmanship" and "Sect. 4—Technique" of the AWS *Structural Welding Code—Steel*, D1.1.

5. Bolted Construction

All parts of bolted members shall be pinned or bolted and rigidly held together while assembling. Use of a drift pin in bolt holes during assembling shall not distort the metal or enlarge the holes. Poor matching of holes shall be cause for rejection.

If the thickness of the material is not greater than the nominal diameter of the bolt plus ⅛-in., the holes may be punched. If the thickness of the material is greater than the nominal diameter of the bolt plus ⅛-in., the holes shall be either drilled or sub-punched and reamed. The die for all sub-punched holes, and the drill for all sub-drilled holes, shall be at least 1/16-in. smaller than the nominal diameter of the bolt. Holes in A514 steel plates over ½-in. thick shall be drilled.

Fully inserted finger shims, with a total thickness of not more than ¼-in. within a joint, may be used in joints without changing the design load (based upon hole type) for the design of connections. The orientation of such shims is independent of the direction of application of the load.

The use of high-strength bolts shall conform to the requirements of the RCSC Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts.

6. Compression Joints

Compression joints which depend on contact bearing as part of the splice capacity shall have the bearing surfaces of individual fabricated pieces prepared by milling, sawing or other suitable means.

7. Dimensional Tolerances

Dimensional tolerances shall be as permitted in the *Code of Standard Practice* of the American Institute of Steel Construction, Inc.

8. Finishing of Column Bases

Column bases and base plates shall be finished in accordance with the following requirements:

a. Steel bearing plates 2 in. or less in thickness may be used without milling, provided a satisfactory contact bearing is obtained. Steel bearing plates over 2 in. but not over 4 in. in thickness may be straightened by pressing or, if presses are not available, by milling for all bearing surfaces (except as noted in subparagraphs b and c of this section), to obtain a satisfactory contact bearing. Steel bearing plates over 4 in. in thickness shall be milled for all bearing surfaces (except as noted in subparagraphs b and c of this section).
b. Bottom surfaces of bearing plates and column bases which are grouted to insure full bearing contact on foundations need not be milled.
c. Top surfaces of bearing plates need not be milled when full-penetration welds are provided between the column and the bearing plate.

M3. SHOP PAINTING

1. General Requirements

Shop painting and surface preparation shall be in accordance with the provisions of the *Code of Standard Practice* of the American Institute of Steel Construction.

Unless otherwise specified, steelwork which will be concealed by interior building finish or will be in contact with concrete need not be painted. Unless specifically excluded, all other steelwork shall be given one coat of shop paint.

2. Inaccessible Surfaces

Except for contact surfaces, surfaces inaccessible after shop assembly shall be cleaned and painted prior to assembly, if required by the design documents.

3. Contact Surfaces

Paint is permitted unconditionally in bearing-type connections. For slip-critical connections where the design is based on special faying surface conditions in accordance with Sect. J3.5, shop contact surfaces shall be cleaned prior to assembly in accordance with the provisions of the *Code of Standard Practice* of the American Institute of Steel Construction, but shall not be painted. Field contact surfaces and surfaces meeting the requirements of Sect. J3.5 shall be shop cleaned in accordance with the design documents, except as provided by Sect. M3.5.

4. Finished Surfaces

Machine-finished surfaces shall be protected against corrosion by a rust-inhibiting coating that can be removed prior to erection, or which has characteristics that make removal prior to erection unnecessary.

5. Surfaces Adjacent to Field Welds

Unless otherwise specified in the design documents, surfaces within 2 in. of any field weld location shall be free of materials that would prevent proper welding or produce toxic fumes during welding.

M4. ERECTION

1. Alignment of Column Bases

Column bases shall be set level and to correct elevation with full bearing on concrete or masonry.

2. Bracing

The frame of steel skeleton buildings shall be carried up true and plumb within the limits defined in the *Code of Standard Practice* of the American Institute of Steel Construction. Temporary bracing shall be provided, in accordance with the requirements of the *Code of Standard Practice*, wherever necessary to take care of all loads to which the structure may be subjected, including equipment and the operation of same. Such bracing shall be left in place as long as may be required for safety.

3. Alignment

No permanent bolting or welding shall be performed until as much of the structure as will be stiffened thereby has been properly aligned.

4. Fit of Column Compression Joints

Lack of contact bearing not exceeding a gap of 1/16-in., regardless of the type of splice used (partial-penetration groove welded or bolted), shall be acceptable. If the gap exceeds 1/16-in., but is less than 1/4-in., and if an engineering investigation shows that sufficient contact area does not exist, the gap shall be packed out with non-tapered steel shims. Shims need not be other than mild steel, regardless of the grade of the main material.

5. Field Welding

Any shop paint on surfaces adjacent to joints to be field welded shall be wire brushed to reduce the paint film to a minimum.

Field welding of attachments to installed embedments in contact with concrete shall be done in such a manner as to avoid excessive thermal expansion of the embedment which could result in spalling or cracking of the concrete or excessive stress in the embedment anchors.

6. Field Painting

Responsibility for touch-up painting, cleaning and field painting shall be allocated in accordance with accepted local practices, and this allocation shall be set forth explicitly in the design documents.

7. Field Connections

As erection progresses, the work shall be securely bolted or welded to take care of all dead load, wind and erection stresses.

M5. QUALITY CONTROL

The fabricator shall provide quality control procedures to the extent that he deems necessary to assure that all work is performed in accordance with this Specification. In addition to the fabricator's quality control procedures, material and workmanship at all times may be subject to inspection by qualified inspectors representing the purchaser. If such inspection by representatives of the purchaser will be required, it shall be so stated in the design documents.

1. Cooperation

As far as possible, all inspection by representatives of the purchaser shall be made at the fabricator's plant. The fabricator shall cooperate with the inspector, permitting access for inspection to all places where work is being done. The purchaser's inspector shall schedule his work for minimum interruption to the work of the fabricator.

2. Rejections

Material or workmanship not in reasonable conformance with the provisions of this Specification may be rejected at any time during the progress of the work. The fabricator shall receive copies of all reports furnished to the purchaser by the inspection agency.

3. Inspection of Welding

The inspection of welding shall be performed in accordance with the provisions of Sect. 6 of the AWS *Structural Welding Code—Steel*, D1.1.

When visual inspection is required to be performed by AWS certified welding inspectors, it shall be so specified in the design documents.

When nondestructive testing is required, the process, extent and standards of acceptance shall be clearly defined in the design documents.

4. Inspection of Slip-critical High-strength Bolted Connections

The inspection of slip-critical high-strength bolted connections shall be in accordance with the provisions of the RCSC Load and Resistance Factor Design Specification for Structural Joints Using ASTM A325 or A490 Bolts.

5. Identification of Steel

The fabricator shall be able to demonstrate by a written procedure and by actual practice a method of material application and identification, visible at least through the "fit-up" operation, of the main structural elements of a shipping piece.

The identification method shall be capable of verifying proper material application as it relates to:

1. Material specification designation
2. Heat number, if required
3. Material test reports for special requirements.

APPENDIX B.
DESIGN REQUIREMENTS

B5. LOCAL BUCKLING

3. Slender Compression Elements

Axially loaded members containing elements subject to compression which have a width-thickness ratio in excess of the applicable λ_r as stipulated in Sect. B5.1 shall be proportioned according to this Appendix. Flexural members with slender compression elements shall be designed in accordance with Appendix F1.7. Rolled flexural members with proportions not covered by Appendix F1.7 shall be designed in accordance with this Appendix.

a. Unstiffened Compression Elements

The design strength of unstiffened compression elements whose width-thickness ratio exceeds the applicable limit λ_r as stipulated in Sect. B5.1 shall be subject to a reduction factor Q_s. The value of Q_s shall be determined by Formulas A-B5-1 through A-B5-6, as applicable. When such elements comprise the compression flange of a flexural member, the maximum required bending stress shall not exceed $\phi_b F_y Q_s$, where $\phi_b = 0.90$. The design strength of axially loaded compression members shall be modified by the appropriate reduction factor Q_s, as provided in paragraph c.

For single angles:

When $76.0/\sqrt{F_y} < b/t < 155/\sqrt{F_y}$:

$$Q_s = 1.340 - 0.00447(b/t)\sqrt{F_y} \tag{A-B5-1}$$

When $b/t \geq 155/\sqrt{F_y}$:

$$Q_s = 15{,}500/[F_y(b/t)^2] \tag{A-B5-2}$$

For angles or plates projecting from columns or other compression members, and for projecting elements of compression flanges of girders:

When $95.0/\sqrt{F_y} < b/t < 176/\sqrt{F_y}$:

$$Q_s = 1.415 - 0.00437(b/t)\sqrt{F_y} \tag{A-B5-3}$$

When $b/t \geq 176/\sqrt{F_y}$:

$$Q_s = 20{,}000/[F_y(b/t)^2] \tag{A-B5-4}$$

For stems of tees:

When $127/\sqrt{F_y} < b/t < 176/\sqrt{F_y}$:

$$Q_s = 1.908 - 0.00715(b/t)\sqrt{F_y} \tag{A-B5-5}$$

When $b/t \geq 176/\sqrt{F_y}$:

$$Q_s = 20{,}000/[F_y(b/t)^2] \tag{A-B5-6}$$

where

b = width of unstiffened compression element as defined in Sect. B5.1, in.
t = thickness of unstiffened element, in.
F_y = specified minimum yield stress, ksi

Unstiffened elements of tees whose proportions exceed the limits of Sect. B5.1 shall conform to the limits given in Table A-B5.1.

b. Stiffened Compression Elements

When the width-thickness ratio of uniformly compressed stiffened elements (except perforated cover plates) exceeds the limit λ_r stipulated in Sect. B5.1, a reduced effective width b_e shall be used in computing the design properties of the section containing the element.

TABLE A-B5.1
Limiting Proportions for Tees

Shape	Ratio of Full Flange Width to Profile Depth	Ratio of Flange Thickness to Web or Stem Thickness
Built-up tees	≥ 0.50	≥ 1.25
Rolled tees	≥ 0.50	≥ 1.10

i. For flanges of square and rectangular sections of uniform thickness:

$$b_e = \frac{326t}{\sqrt{f}}\left[1 - \frac{64.9}{(b/t)\sqrt{f}}\right] \leq b \tag{A-B5-7}$$

ii. For other uniformly compressed elements:

$$b_e = \frac{326t}{\sqrt{f}}\left[1 - \frac{57.2}{(b/t)\sqrt{f}}\right] \leq b \tag{A-B5-8}$$

where

b = actual width of a stiffened compression element, as defined in Sect. B5.1, in.
b_e = reduced width, in.
t = element thickness, in.
f = computed elastic compressive stress in the stiffened elements, based on the design properties as specified in Sect. C of this Appendix, ksi. If unstiffened elements are included in the total cross section, f for the stiffened element must be such that the maximum compressive stress in the unstiffened element does not exceed $\phi_c F_{cr}$ as defined in Appendix B5.3c with $Q = Q_s$ and $\phi_c = 0.85$, or $\phi_b F_y Q_s$ with $\phi_b = 0.90$, as applicable.

iii. For axially loaded circular sections:

Members with diameter-to-thickness ratios D/t greater than $3{,}300/F_y$, but having a diameter-to-thickness ratio of less than $13{,}000/F_y$;

$$Q = \frac{1{,}100}{F_y(D/t)} + \frac{2}{3} \qquad \text{(A-B5-9)}$$

where

D = outside diameter, in.
t = wall thickness, in.

c. Design Properties

Properties of sections shall be determined using the full cross section, except as follows:

In computing the moment of inertia and elastic section modulus of flexural members, the effective width of uniformly compressed stiffened elements, as determined in Appendix B5.3b, shall be used in determining effective cross-sectional properties.

For unstiffened elements of the cross section, Q_s is determined from Appendix B5.3a. For stiffened elements of the cross section

$$Q_a = \frac{\text{effective area}}{\text{actual area}} \qquad \text{(A-B5-10)}$$

where the effective area is equal to the summation of the effective areas of the cross section.

For axially loaded compression members the gross cross-sectional area and the radius of gyration r shall be computed on the basis of the actual cross section. However, when $\lambda_c \sqrt{Q} \leq 1.5$, the critical stress F_{cr} shall be determined by

$$F_{cr} = Q(0.658^{Q\lambda_c^2})F_y \qquad \text{(A-B5-11)}$$

where

$$Q = Q_s Q_a \qquad \text{(A-B5-12)}$$

i. Cross sections composed entirely of unstiffened elements, $Q = Q_s (Q_a = 1.0)$
ii. Cross sections composed entirely of stiffened elements, $Q = Q_a (Q_s = 1.0)$
iii. Cross sections composed of both stiffened and unstiffened elements, $Q = Q_s Q_a$

When $\lambda_c \sqrt{Q} > 1.5$, the critical stress F_{cr} shall be determined by

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right] F_y \qquad \text{(A-B5-13)}$$

APPENDIX E. COLUMNS AND OTHER COMPRESSION MEMBERS

E3. FLEXURAL-TORSIONAL BUCKLING

This section applies to the strength of singly symmetric and unsymmetric columns for the limiting states of flexural-torsional and torsional buckling.

Torsional buckling of symmetric shapes and flexural-torsional buckling of unsymmetric shapes are modes of buckling usually not considered in the design of hot-rolled columns. (Generally, either these modes do not govern or the critical load differs very little from the weak-axis planar buckling load.) Such buckling modes may, however, control the capacity of columns made from relatively thin plate elements and of unsymmetric columns.

The strength of compression members determined by the limit states of torsional and flexural-torsional buckling is $\phi_c P_n$, where

$\phi_c = 0.85$
P_n = nominal resistance in compression, kips
$= A_g F_{cr}$ (A-E3-1)
A_g = gross area of cross section, in.^{2}
Q = 1.0 for elements meeting the width-thickness ratios λ_r of Sect. B5.1
$= Q_s Q_a$ for elements not meeting the width-thickness ratios λ_r of Sect. B5.1 and determined in accordance with the provisions of Appendix B5.3

The nominal critical stress F_{cr} is determined as follows:

a. For $\lambda_e \sqrt{Q} \leq 1.5$:

$$F_{cr} = Q(0.658^{Q\lambda_e^2})F_y \qquad \text{(A-E3-2)}$$

b. For $\lambda_e \sqrt{Q} > 1.5$:

$$F_{cr} = \left[\frac{0.877}{\lambda_e^2}\right] F_y \qquad \text{(A-E3-3)}$$

where

$$\lambda_e = \sqrt{F_y / F_e} \qquad \text{(A-E3-4)}$$

F_y = specified minimum yield stress of steel, ksi

The critical torsional or flexural-torsional elastic buckling stress F_e is determined as follows:

a. For doubly symmetric shapes the critical torsional elastic buckling stress is

$$F_e = \left[\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right] \frac{1}{I_x + I_y} \qquad \text{(A-E3-5)}$$

b. For singly symmetric shapes, where y is the axis of symmetry, the critical flexural-torsional elastic buckling stress is

$$F_e = \frac{F_{ey} + F_{ez}}{2H}\left(1 - \sqrt{1 - \frac{4 F_{ey} F_{ez} H}{(F_{ey} + F_{ez})^2}}\right) \qquad \text{(A-E3-6)}$$

c. For unsymmetric shapes, the critical flexural-torsional elastic buckling stress F_e is the lowest root of the cubic equation

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2(F_e - F_{ey})(x_o / \bar{r}_o)^2 - F_e^2 (F_e - F_{ex})(y_o / \bar{r}_o)^2 = 0 \qquad \text{(A-E3-7)}$$

where

K_z = effective length factor for torsional buckling
E = modulus of elasticity, ksi
G = shear modulus, ksi
C_w = warping constant, in.^{6}
J = torsional constant, in.^{4}
I_x, I_y = moment of inertia about the principal axes, in.^{4}
x_o, y_o = coordinates of shear center with respect to the centroid, in.

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A} \qquad \text{(A-E3-8)}$$

$$H = 1 - \left(\frac{x_o^2 + y_o^2}{\bar{r}_o^2}\right) \qquad \text{(A-E3-9)}$$

$$F_{ex} = \frac{\pi^2 E}{(K_x L / r_x)^2} \qquad \text{(A-E3-10)}$$

$$F_{ey} = \frac{\pi^2 E}{(K_y L / r_y)^2} \qquad \text{(A-E3-11)}$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{(K_z L)^2} + \mathrm{GJ}\right) \frac{1}{A \bar{r}_o^2} \qquad \text{(A-E3-12)}$$

where

A = cross-sectional area of member, in.^{2}
L = unbraced length, in.
K_x, K_y = effective length factors in x and y directions
r_x, r_y = radii of gyration about the principal axes, in.

APPENDIX F. BEAMS AND OTHER FLEXURAL MEMBERS

F1. DESIGN FOR FLEXURE

7. Nominal Flexural Strength

Table A-F1.1 provides a tabular summary of Formulas F1-3 through F1-16 for determining the nominal flexural strength of beams and girders. For slenderness parameters of cross sections not included in Table A-F1.1, see Appendix B5.3.

The nominal flexural strength M_n is the lowest value obtained according to the limit states of: (a) lateral-torsional buckling (LTB); (b) flange local buckling (FLB); and (c) web local buckling (WLB).

The nominal flexural strength M_n shall be determined as follows for each limit state:

For $\lambda \leq \lambda_p$:

$$M_n = M_p \tag{A-F1-1}$$

For $\lambda_p < \lambda \leq \lambda_r$:

For the limit state of lateral-torsional buckling:

$$M_n = C_b \left[M_p - (M_p - M_r)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \leq M_p \tag{A-F1-2}$$

For the limit states of flange and web local buckling:

$$M_n = M_p - (M_p - M_r)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \tag{A-F1-3}$$

For $\lambda > \lambda_r$:

For the limit state of lateral-torsional buckling and for flange local buckling:

$$M_n = M_{cr} = S\,F_{cr} \qquad \text{(A-F1-4)}$$

For λ of the flange $> \lambda_r$ in shapes not included in Table A-F1.1, see Appendix B5.3.

For λ of the web $> \lambda_r$, see Appendix G.

For all slenderness parameters, the flexural strength of hybrid sections is limited by the flexural strength of equivalent homogeneous sections made up of the hybrid web and flange steel grade.

The terms used in the above equations are:

M_n = nominal flexural strength, kip-in.
M_p = plastic moment, kip-in.
M_{cr} = buckling moment, kip-in.
M_r = limiting buckling moment (equal to M_{cr} when $\lambda = \lambda_r$), kip-in.
λ = controlling slenderness parameter
= minor axis slenderness ratio L_b/r_y for lateral-torsional buckling
= flange width-thickness ratio b/t for flange local buckling as defined in Sect. B5.1
= web depth-thickness ratio h/t_w for web local buckling as defined in Sect. B5.1
λ_p = largest value of λ for which $M_n = M_p$
λ_r = largest value of λ for which buckling is inelastic
F_{cr} = critical stress, ksi
C_b = $1.75 + 1.05(M_1/M_2) + 0.3(M_1/M_2)^2 \leq 2.3$ where M_1 is the smaller and M_2 the larger end-moment in the unbraced segment of the beam; M_1/M_2 is positive when the moments cause reverse curvature.
S = section modulus, in.^{3}
L_b = laterally unbraced length, in.
r_y = radius of gyration about minor axis, in.

The applicable limit states and equations for M_p, M_r, F_{cr}, λ, λ_p and λ_r are given in Table A-F1.1 for the shapes covered in this Appendix. The terms used in the table are:

A = cross-sectional area, in.^{2}
F_r = compressive residual stress in flange
= 10 ksi for rolled shapes
= 16.5 ksi for welded shapes
F_y = specified minimum yield strength, ksi
F_{yf} = yield strength of the flange, ksi
J = torsional constant, in.^{4}
R_e = see Appendix G2
$(S_x)_{eff}$ = effective section modulus about major axis, in.^{3}
S_{xc} = section modulus of the outside fiber of the compression flange, in.^{3}
S_{xt} = section modulus of the outside fiber of the tension flange, in.^{3}
Z = plastic section modulus, in.^{3}
b = flange width, in.
d = overall depth, in.
h_c = twice the distance from the neutral axis to the inside face of the compression flange less the fillet or corner radius, in.
t_f = flange thickness, in.
t_w = web thickness, in.

TABLE A-F1.1
Nominal Strength Parameters

Shape	Plastic Moment M_p	Limit State of Buckling	Limiting Buckling Moment M_r
Channels and doubly and singly symmetric I-shaped beams (including hybrid beams) bent about major axis	$F_y Z_x$ (e)	LTB Doubly symmetric members and channels	$(F_{yw} - F_r)S_x$
		LTB Singly symmetric members	$(F_{yw} - F_r)S_{xc} \leq F_{yf} S_{xt}$
		FLB	$(F_{yw} - F_r)S_x$
		WLB	$R_e F_{yf} S_x$
Channels and doubly symmetric I-shaped members bent about minor axis	$F_y Z_y$	FLB	$F_y S_y$
Solid symmetric shapes, except rectangular bars, bent about major axis	$F_y Z_x$	Not Applicable	
Solid rectangular bars bent about major axis	$F_y Z_x$	LTB	$F_y S_x$

TABLE A-F1.1 (cont'd)
Nominal Strength Parameters

<table>
<tr><th rowspan="2">Critical Stress F_{cr}</th><th colspan="3">Slenderness Parameters</th><th rowspan="2">Limitations</th></tr>
<tr><th>λ</th><th>λ_p</th><th>λ_r</th></tr>
<tr><td>$\dfrac{C_b X_1 \sqrt{2}}{\lambda}\sqrt{1 + \dfrac{X_1^2 X_2}{2\lambda^2}}$
(b)</td><td>$\dfrac{L_b}{r_y}$</td><td>$\dfrac{300}{\sqrt{F_{yf}}}$</td><td>(a,b)</td><td rowspan="4">1. Applicable for I-shaped members if $h_c/t_w \leq 970 / \sqrt{F_{yf}}$
when $h_c/t_w > 970 / \sqrt{F_{yf}}$
See Appendix G
2. When $\lambda > \lambda_r$
a) Applicable for built-up channels if $b/d \leq 0.25$ and $t_f/t_w \leq 3.0$
b) Applicable for rolled channels if $b/d \leq 0.25$ and $t_f/t_w \leq 2.0$</td></tr>
<tr><td>(c)</td><td>$\dfrac{L_b}{r_y}$</td><td>$\dfrac{300}{\sqrt{F_{yf}}}$</td><td>Value of λ for which $M_{cr} = S_{xc}(F_{yf} - F_r)$</td></tr>
<tr><td>(g)</td><td>$\dfrac{b}{t}$</td><td>$\dfrac{65}{\sqrt{F_{yf}}}$</td><td>(h)</td></tr>
<tr><td>N.A.</td><td>$\dfrac{h_c}{t_w}$</td><td>$\dfrac{640}{\sqrt{F_{yf}}}$</td><td>$\dfrac{970}{\sqrt{F_{yf}}}$</td></tr>
<tr><td colspan="4">Same as for major axis</td><td></td></tr>
<tr><td colspan="5">Not Applicable</td></tr>
<tr><td>$\dfrac{57{,}000 C_b \sqrt{JA}}{S_x}$</td><td>$\dfrac{L_b}{r_y}$</td><td>$\dfrac{3{,}750 \sqrt{JA}}{M_p}$</td><td>$\dfrac{57{,}000 \sqrt{JA}}{M_r}$</td><td></td></tr>
</table>

TABLE A-F1.1 (cont'd)
Nominal Strength Parameters

Shape	Plastic Moment M_p	Limit State of Buckling	Limiting Buckling Moment M_r
Symmetric box sections loaded in a plane of symmetry	$F_y Z_x$	LTB	$(F_{yf} - F_r)S_x$
		FLB	$F_y S_x$
		WLB	Same as for I-shape
Circular Tubes	$F_y Z$	LTB	Not Applicable
		FLB	$M_n = \left(\dfrac{600}{D/t} + F_y\right)S^{(f)}$
		WLB	Not Applicable

a $\lambda_r = \dfrac{X_1}{(F_{yw} - F_r)}\sqrt{1 + \sqrt{1 + X_2 (F_{yw} - F_r)^2}}$

b $X_1 = \dfrac{\pi}{S_x}\sqrt{\dfrac{EGJA}{2}}$ $\quad X_2 = 4\dfrac{C_w}{I_y}\left(\dfrac{S_x}{GJ}\right)^2$

c $M_{cr} = \dfrac{57{,}000C_b}{L_b}\sqrt{I_y J}\,(B_1 + \sqrt{(1 + B_2 + B_1^2)}) \leq S_{xt} F_y$

where

$B_1 = 2.25\,[2(I_{yc}/I_y) - 1](h/L_b)\sqrt{(I_y/J)}$ $\qquad B_2 = 25(1 - I_{yc}/I_y)(I_{yc}/J)(h/L_b)^2$

If the compression flange is larger than the tension flange, M_{cr} may be evaluated conservatively by the formula for doubly symmetric members, using for X_1 and X_2 the values for a symmetric section both of whose flanges are the same as the compression flange of the singly symmetric section.

TABLE A-F1.1 (cont'd)
Nominal Strength Parameters

Critical Stress F_{cr}	Slenderness Parameters λ	λ_p	λ_r	Limitations
$\dfrac{57{,}000 C_b \sqrt{JA}}{\lambda S_x}$	$\dfrac{L_b}{r_y}$	$\dfrac{3{,}750\sqrt{JA}}{M_p}$	$\dfrac{57{,}000\sqrt{JA}}{M_r}$	Applicable if $h_c/t_w \le 970/\sqrt{F_{yf}}$
$\dfrac{(S_x)_{eff}}{S_x} F_y$ (d)	$\dfrac{b}{t}$	$\dfrac{190}{F_y}$	$\dfrac{238}{\sqrt{(F_y - F_r)}}$	LTB applies only if $d > b$
Same as for I-shape				
Not Applicable				
$\dfrac{9{,}750}{D/t}$	D/t	$\dfrac{2{,}070}{F_y}$	$\dfrac{8{,}970}{F_y}$	$D/t < \dfrac{13{,}000}{F_y}$
Not Applicable				

[d] $(S_x)_{eff}$ is the section modulus for a section with a compression-flange width b_e given by $b_e = \dfrac{326\,t}{\sqrt{f}}\left(1 - \dfrac{64.9}{(b/t)\sqrt{f}}\right)$ for flanges of square and rectangular sections of uniform thickness

[e] Computed from fully plastic stress distribution for hybrid sections.

[f] This equation is to be used in place of Formula A-F1-3.

[g] $F_{cr} = \dfrac{20{,}000}{\lambda^2}$ for rolled shapes

$F_{cr} = \dfrac{11{,}200}{\lambda^2}$ for welded shapes

[h] $\lambda_r = \dfrac{141}{\sqrt{F_{yw} - 10}}$ for rolled shapes

$\lambda_r = \dfrac{106}{\sqrt{F_{yw} - 16.5}}$ for welded shapes

F4. WEB-TAPERED MEMBERS

The design of tapered members meeting the requirements of this section shall be governed by the provisions of Chap. F, except as modified by this Appendix.

1. General Requirements

In order to qualify under this Specification, a tapered member must meet the following requirements:

a. It shall possess at least one axis of symmetry which shall be perpendicular to the plane of bending if moments are present.
b. The flanges shall be of equal and constant area.
c. The depth shall vary linearly as

$$d = d_o\left(1 + \gamma \frac{z}{L}\right) \qquad \text{(A-F4-1)}$$

where

d_o = depth at smaller end of member, in.
d_L = depth at larger end of member, in.
γ = $(d_L - d_o)/d_o \leq$ the smaller of $0.268(L/d_o)$ or 6.0
z = distance from the smaller end of member, in.
L = unbraced length of member measured between the center of gravity of the bracing members, in.

2. Design Tensile Strength

The design strength of tapered tension members shall be determined in accordance with Sect. D1.

3. Design Compressive Strength

The design strength of tapered compression members shall be determined in accordance with Sect. E2, using an effective slenderness parameter λ_{eff} computed as follows:

$$\lambda_{eff} = \frac{S}{\pi}\sqrt{\frac{QF_y}{E}} \qquad \text{(A-F4-2)}$$

where

S = KL/r_{oy} for weak axis bending and $K_\gamma L/r_{ox}$ for strong axis bending
K = effective length factor for a prismatic member
K_γ = effective length factor for a tapered member as determined by a rational analysis
r_{ox} = strong axis radius of gyration at the smaller end of a tapered member, in.
r_{oy} = weak axis radius of gyration at the smaller end of a tapered member, in.
F_y = specified minimum yield stress, ksi
Q = reduction factor
= 1.0 if all elements meet the limiting width-thickness ratios λ_r of Sect. B5.1
= $Q_s Q_a$, determined in accordance with Appendix B, if any stiffened and/or unstiffened elements exceed the ratios λ_r of Sect. B5.1
E = modulus of elasticity for steel, ksi

The smallest area of the tapered member shall be used for A_g in Formula E2-1.

4. Design Flexural Strength

The design flexural strength of tapered flexural members for the limit state of lateral-torsional buckling is $\phi_b M_n$, where $\phi_b = 0.90$ and the nominal strength is

$$M_n = (5/3)S'_x F_{b\gamma} \tag{A-F4-3}$$

where

S'_x = the section modulus of the critical section of the unbraced beam length under consideration

$$F_{b\gamma} = \frac{2}{3}\left[1.0 - \frac{F_y}{6B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2}}\right] F_y \leq 0.60 F_y \tag{A-F4-4}$$

unless $F_{b\gamma} \leq F_y/3$, in which case

$$F_{b\gamma} = B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2} \tag{A-F4-5}$$

In the above equations,

$$F_{s\gamma} = \frac{12 \times 10^3}{h_s L d_o / A_f} \tag{A-F4-6}$$

$$F_{w\gamma} = \frac{170 \times 10^3}{(h_w L / r_{To})^2} \tag{A-F4-7}$$

where

h_s = factor equal to $1.0 + 0.0230\gamma\sqrt{L d_o / A_f}$
h_w = factor equal to $1.0 + 0.00385\gamma\sqrt{L / r_{To}}$
r_{To} = radius of gyration of a section at the smaller end, considering only the compression flange plus ⅓ of the compression web area, taken about an axis in the plane of the web, in.
A_f = area of the compression flange, in.^{2}

and where B is determined as follows:

a. When the maximum moment M_2 in three adjacent segments of approximately equal unbraced length is located within the central segment and M_1 is the larger moment at one end of the three-segment portion of a member:*

$$B = 1.0 + 0.37\left(1.0 + \frac{M_1}{M_2}\right) + 0.50\gamma\left(1.0 + \frac{M_1}{M_2}\right) \geq 1.0 \tag{A-F4-8}$$

b. When the largest computed bending stress f_{b2} occurs at the larger end of two adjacent segments of approximately equal unbraced lengths and f_{b1} is the computed bending stress at the smaller end of the two-segment portion of a member:**

$$B = 1.0 + 0.58\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) - 0.70\gamma\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) \geq 1.0 \tag{A-F4-9}$$

---

*M_1/M_2 is considered as negative when producing single curvature. In the rare case where M_1/M_2 is positive, it is recommended that it be taken as zero.
**f_{b1}/f_{b2} is considered as negative when producing single curvature. If a point of contraflexure occurs in one of two adjacent unbraced segments, f_{b1}/f_{b2} is considered as positive. The ratio $f_{b1}/f_{b2} \neq 0$.

c. When the largest computed bending stress f_{b2} occurs at the smaller end of two adjacent segments of approximately equal unbraced length and f_{b1} is the computed bending stress at the larger end of the two-segment portion of a member:**

$$B = 1.0 + 0.55\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) + 2.20\gamma\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) \geq 1.0 \qquad \text{(A-F4-10)}$$

In the foregoing, $\gamma = (d_L - d_o)/d_o$ is calculated for the unbraced length that contains the maximum computed bending stress.

d. When the computed bending stress at the smaller end of a tapered member or segment thereof is equal to zero:

$$B = \frac{1.75}{1.0 + 0.25\sqrt{\gamma}} \qquad \text{(A-F4-11)}$$

where $\gamma = (d_L - d_o)/d_o$, calculated for the unbraced length adjacent to the point of zero bending stress.

5. Design Shear Strength

The design shear strength of tapered flexural members shall be determined in accordance with Sect. F2.

6. Combined Flexure and Axial Force

For tapered members with a single web taper subject to compression and bending about the major axis, Formulas H1-1 through H1-3 apply, with the following modifications: P_n and P_{ex} shall be determined for the properties of the smaller end, using appropriate effective length factors. M_{nx}, M_u and M_{px} shall be determined for the larger end; $M_{nx} = (5/3)S'_x F_{b\gamma}$, where S'_x is the elastic section modulus of the larger end, and $F_{b\gamma}$ is the design flexural stress of tapered members. C_{mx} is replaced by C'_m, determined as follows:

a. When the member is subjected to end moments which cause single curvature bending and approximately equal computed moments at the ends:

$$C'_m = 1.0 + 0.1\left(\frac{P_u}{\phi_b P_{ex}}\right) + 0.3\left(\frac{P_u}{\phi_b P_{ex}}\right)^2 \qquad \text{(A-F4-12)}$$

b. When the computed bending moment at the smaller end of the unbraced length is equal to zero:

$$C'_m = 1.0 + 0.9\left(\frac{P_u}{\phi_b P_{ex}}\right) + 0.6\left(\frac{P_u}{\phi_b P_{ex}}\right)^2 \qquad \text{(A-F4-13)}$$

When the effective slenderness parameter $\lambda_{eff} \geq 1.0$ and combined stress is checked incrementally along the length, the actual area and the actual section modulus at the section under investigation may be used.

**f_{b1}/f_{b2} is considered as negative when producing single curvature. If a point of contraflexure occurs in one of two adjacent unbraced segments, f_{b1}/f_{b2} is considered as positive. The ratio $f_{b1}/f_{b2} \neq 0$.

APPENDIX G
PLATE GIRDERS

G1. LIMITATIONS

Doubly and singly symmetric single-web non-hybrid and hybrid plate girders loaded in the plane of the web may be proportioned according to the provisions of this Appendix or Sect. F2, provided that these limits are satisfied.

a. For $\dfrac{a}{h} \leq 1.5$:

$$\frac{h}{t_w}\max = \frac{2{,}000}{\sqrt{F_{yf}}} \qquad \text{(A-G1-1)}$$

b. For $\dfrac{a}{h} > 1.5$:

$$\frac{h}{t_w}\max = \frac{14{,}000}{\sqrt{F_{yf}(F_{yf} + 16.5)}} \qquad \text{(A-G1-2)}$$

where

a = clear distance between transverse stiffeners, in.
h = clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, in.
t_w = web thickness, in.
F_{yf} = specified minimum yield stress of the flange, ksi

In unstiffened girders h/t_w must be less than 260.

G2. DESIGN FLEXURAL STRENGTH

The design flexural strength for plate girders with slender webs ($h_c/t_w > 970/\sqrt{F_{yf}}$) shall be $\phi_b M_n$, where $\phi_b = 0.90$ and M_n is the lower value obtained according to the limit states of tension-flange yield and buckling. When $h_c/t_w \leq 970/\sqrt{F_{yf}}$, see Appendix F1.7.

For tension-flange yield:

$$M_n = S_{xt} R_{PG} R_e F_{yt} \tag{A-G2-1}$$

For compression flange buckling:

$$M_n = S_{xc} R_{PG} R_e F_{cr} \tag{A-G2-2}$$

where

$$R_{PG} = 1 - 0.0005 a_r \left(\frac{h_c}{t_w} - \frac{970}{\sqrt{F_{cr}}} \right) \leq 1.0 \tag{A-G2-3}$$

R_e = hybrid girder factor
= $1.0 - 0.1(1.3 + a_r)(0.81 - m) \leq 1.0$ (for non-hybrid girders, $R_e = 1.0$)
a_r = ratio of web area to compression flange area
m = ratio of web yield stress to flange yield stress or F_{cr}
F_{cr} = critical compression flange stress, ksi
F_{yt} = yield stress of tension flange, ksi
S_{xc} = section modulus referred to compression flange, in.^{3}
S_{xt} = section modulus referred to tension flange, in.^{3}
h_c = twice the distance from the neutral axis to the inside face of the compression flange less the fillet or corner radius, in.

The critical stress F_{cr} to be used is dependent upon the slenderness parameters λ, λ_p, λ_r and C_{PG} as follows:

a. For $\lambda \leq \lambda_p$:

$$F_{cr} = F_{yf} \tag{A-G2-4}$$

b. For $\lambda_p < \lambda \leq \lambda_r$:

$$F_{cr} = C_b F_{yf} \left[1 - \frac{1}{2} \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) \right] \leq F_{yf} \tag{A-G2-5}$$

c. For $\lambda > \lambda_r$:

$$F_{cr} = \frac{C_{PG}}{\lambda^2} \tag{A-G2-6}$$

In the foregoing, the slenderness parameter shall be determined for both the limit state of lateral-torsional buckling and the limit state of flange local buckling; the slenderness parameter which results in the lowest value of F_{cr} governs.

For the limit state of lateral-torsional buckling:

$$\lambda = \frac{L_b}{r_T} \tag{A-G2-7}$$

$$\lambda_p = \frac{300}{\sqrt{F_{yf}}} \tag{A-G2-8}$$

$$\lambda_r = \frac{756}{\sqrt{F_{yf}}} \tag{A-G2-9}$$

$$C_{PG} = 286{,}000\ C_b \tag{A-G2-10}$$

where

$C_b = 1.75 + 1.05(M_1/M_2) + 0.3(M_1/M_2)^2 \leq 2.3$
r_T = radius of gyration of compression flange plus one-sixth the web, in.

For the limit state of flange local buckling:

$$\lambda = \frac{b_f}{2t_f} \qquad \text{(A-G2-11)}$$

$$\lambda_p = \frac{65}{\sqrt{F_{yf}}} \qquad \text{(A-G2-12)}$$

$$\lambda_r = \frac{150}{\sqrt{F_{yf}}} \qquad \text{(A-G2-13)}$$

$$C_{PG} = 11{,}200 \qquad \text{(A-G2-14)}$$

$$C_b = 1$$

The limit state of flexural web local buckling is not applicable.

G3. DESIGN SHEAR STRENGTH WITH TENSION FIELD ACTION

The design shear strength shall be $\phi_v V_n$, kips, where $\phi_v = 0.90$ and V_n is determined as follows:

a. For $h/t_w \leq 187\sqrt{k/F_{yw}}$:

$$V_n = 0.6\, A_w F_{yw} \qquad \text{(A-G3-1)}$$

b. For $h/t_w > 187\sqrt{k/F_{yw}}$:

$$V_n = 0.6\, A_w F_{yw}\left(C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}}\right) \qquad \text{(A-G3-2)}$$

where

C_v = ratio of "critical" web stress, according to linear buckling theory, to the shear yield stress of web material

except for end-panels in non-hybrid plate girders, for all panels in hybrid and web-tapered plate girders and when a/h exceeds 3.0 or $[260/(h/t_w)]^2$. In these cases, tension field action is not permitted and

$$V_n = 0.6\, A_w\, F_{yw}\, C_v \qquad \text{(A-G3-3)}$$

The web plate buckling coefficient k is given as

$$k = 5 + \frac{5}{(a/h)^2} \qquad \text{(A-G3-4)}$$

except that k shall be taken as 5.0 if a/h exceeds 3.0 or $[260/(h/t_w)]^2$. The shear coefficient C_v is determined as follows:

For $187\sqrt{\frac{k}{F_{yw}}} \leq \frac{h}{t_w} \leq 234\sqrt{\frac{k}{F_{yw}}}$:

$$C_v = \frac{187\sqrt{k/F_{yw}}}{h/t_w} \qquad \text{(A-G3-5)}$$

For $\dfrac{h}{t_w} > 234\sqrt{\dfrac{k}{F_{yw}}}$:

$$C_v = \frac{44{,}000\,k}{(h/t_w)^2 F_{yw}} \tag{A-G3-6}$$

G4. TRANSVERSE STIFFENERS

Transverse stiffeners are not required in plate girders when $h/t_w \leq 418/\sqrt{F_{yw}}$, or when the required shear V_u, as determined by structural analysis for the factored loads, is less than or equal to $0.6\phi A_w F_{yw} C_v$, where C_v is determined for $k = 5$ and $\phi = 0.90$. Stiffeners may be required in certain portions of a plate girder to develop the required shear or to satisfy the limitations given in Appendix G1.

The moment of inertia I_{st} of a transverse stiffener about an axis in the web center for stiffener pairs or about the face in contact with the web plate for single stiffeners shall not be less than $at_w^3 j$, where

$$j = \frac{2.5}{(a/h)^2} - 2 \geq 0.5 \tag{A-G4-1}$$

and the stiffener area A_{st} when designing for tension field action shall not be less than

$$\frac{F_{yw}}{F_{yst}}\left[0.15\,Dht_w(1 - C_v)\frac{V_u}{\phi_v V_n} - 18\,t_w^2\right] \geq 0 \tag{A-G4-2}$$

where

F_{yst} = specified yield stress of the stiffener material, ksi
D = 1 for stiffeners in pairs
= 1.8 for single angle stiffeners
= 2.4 for single plate stiffeners

C_v and V_n are defined in Appendix G3, and V_u is the required shear at the location of the stiffener.

G5. FLEXURE-SHEAR INTERACTION

Plate girders with webs that depend on tension field action must satisfy flexure-shear interaction criteria.

When stiffeners are required and

$$\frac{0.6\,V_n}{M_n} \leq \frac{V_u}{M_u} \leq \frac{V_n}{0.75\,M_n}$$

the following interaction equation must be satisfied:

$$\frac{M_u}{M_n} + .625\frac{V_u}{V_n} \leq 1.375\,\phi \tag{A-G5-1}$$

where M_n is the nominal flexural strength of plate girders from Appendix G2, $\phi = 0.90$ and V_n is the nominal shear strength from Appendix G3, except that M_u may not exceed ϕM_n ($\phi = 0.90$) and V_u may not exceed ϕV_n ($\phi = 0.90$).

APPENDIX H. MEMBERS UNDER TORSION AND COMBINED FORCES

H3. ALTERNATE INTERACTION EQUATIONS FOR MEMBERS UNDER COMBINED STRESS

For biaxially loaded I-shaped members used in braced frames only, the following interaction equations may be used in lieu of Formulas H1-1a and H1-1b.

$$\left(\frac{M_{ux}}{\phi_b\, M'_{px}}\right)^{\zeta} + \left(\frac{M_{uy}}{\phi_b\, M'_{py}}\right)^{\zeta} \le 1.0 \qquad \text{(A-H3-1)}$$

$$\left(\frac{C_{mx}\, M_{ux}}{\phi_b\, M'_{nx}}\right)^{\eta} + \left(\frac{C_{my}\, M_{uy}}{\phi_b M'_{ny}}\right)^{\eta} \le 1.0 \qquad \text{(A-H3-2)}$$

For $0.5 \le b_f/d \le 1.0$:

$$\zeta = 1.6 - \frac{P_u/P_y}{2[\ln(P_u/P_y)]} \qquad \text{(A-H3-3)}$$

For $b_f/d \ge 0.3$:

$$\eta = 0.4 + \frac{P_u}{P_y} + \frac{b_f}{d} \ge 1.0 \qquad \text{(A-H3-4)}$$

$$= 1.0 \text{ for } b_f/d < 0.3$$

where

b_f = flange width, in.
d = member depth, in.

$$M'_{px} = 1.2\, M_{px}[1 - (P_u/P_y)] \le M_{px} \qquad \text{(A-H3-5)}$$

$$M'_{py} = 1.2\, M_{py}[1 - (P_u/P_y)^2] \le M_{py} \qquad \text{(A-H3-6)}$$

$$M'_{nx} = M_{nx}\left(1 - \frac{P_u}{\phi_c P_n}\right)\left(1 - \frac{P_u}{P_{ex}}\right) \qquad \text{(A-H3-7)}$$

$$M'_{ny} = M_{ny}\left(1 - \frac{P_u}{\phi_c P_n}\right)\left(1 - \frac{P_u}{P_{ey}}\right) \qquad \text{(A-H3-8)}$$

where

P_n = nominal compressive strength determined in accordance with Sect. E2, kips
P_u = required axial strength, kips
P_y = compressive yield strength $A_g F_y$, kips
ϕ_b = resistance factor for flexure = 0.90
ϕ_c = resistance factor for compression = 0.85
P_e = Euler buckling strength $A_g F_y/\lambda_c^2$, where λ_c is the column slenderness parameter defined by Formula E2-4, kips
M_u = required flexural strength, kip-in.
M_n = nominal flexural strength, determined in accordance with Sect. F1, kip-in.
M_p = plastic moment, kip-in.
C_m = coefficient defined in Sect. H1.

Notes

APPENDIX K.
STRENGTH DESIGN CONSIDERATIONS

K2. PONDING

The provisions of this Appendix may be used when a more exact determination of flat roof framing stiffness is needed than that given by the provision of Sect. K2 that $C_p + 0.9C_s \leq 0.25$.

For any combination of primary and secondary framing, the stress index is computed as

$$U_p = \left(\frac{\phi_b F_y - f_o}{f_o}\right)_p \text{ for the primary member} \qquad \text{(A-K2-3)}$$

$$U_s = \left(\frac{\phi_b F_y - f_o}{f_o}\right)_s \text{ for the secondary member} \qquad \text{(A-K2-4)}$$

where

f_o = the stress due to $1.2D + 1.2R$ (D = nominal dead load, R = nominal load due to rain water or ice exclusive of the ponding contribution)*

ϕ_b = resistance factor for flexure = 0.90 (Sect. F)

---

*Depending upon geographic location, this loading should include such amount of snow as might also be present, although ponding failures have occurred more frequently during torrential summer rains when the rate of precipitation exceeded the rate of drainage runoff and the resulting hydraulic gradient over large roof areas caused substantial accumulation of water some distance from the eaves.

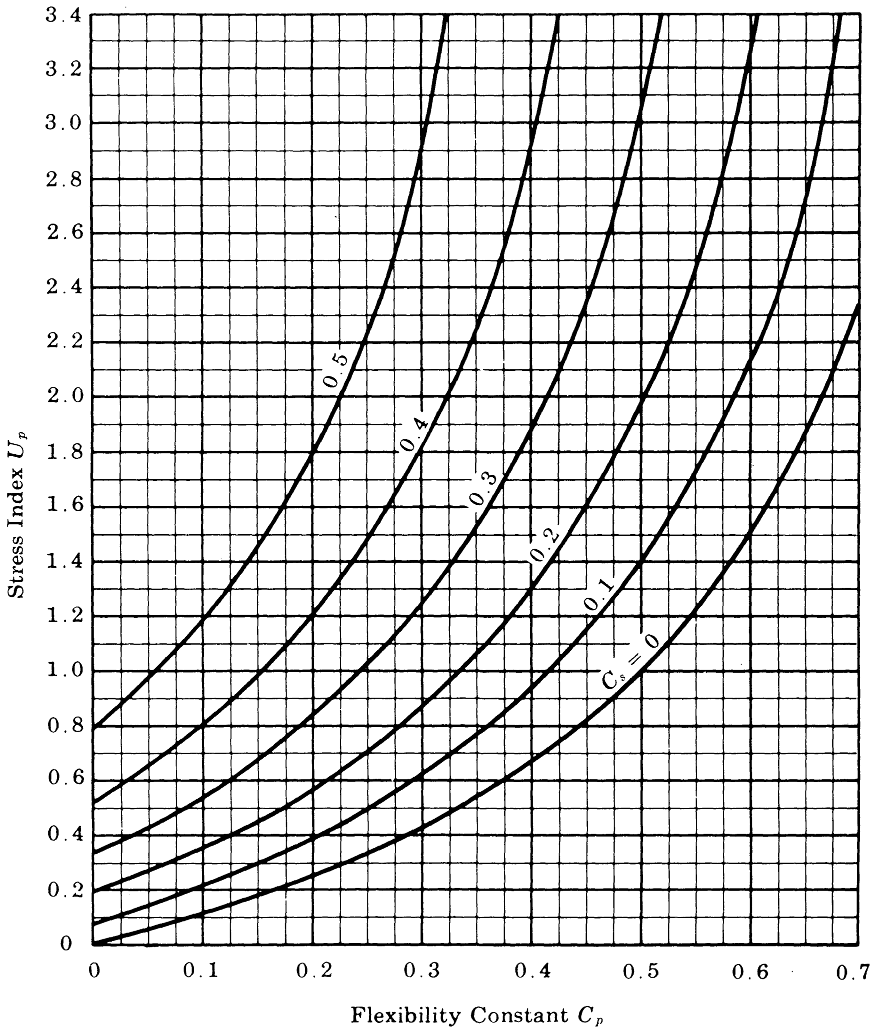


Fig. A-K2.1. Flexibility coefficients for combined primary and secondary systems

Enter Fig. A-K2.1 at the level of the computed stress index U_p determined for the primary beam; move horizontally to the computed C_s-value of the secondary beams and then downward to the abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility constant read from this latter scale is more than the value of C_p computed for the given primary member; if not, a stiffer primary or secondary beam, or combination of both, is required. In the above,

$$C_p = \frac{32 L_s L_p^4}{10^7 I_p}$$

$$C_s = \frac{32 S L_s^4}{10^7 I_s}$$

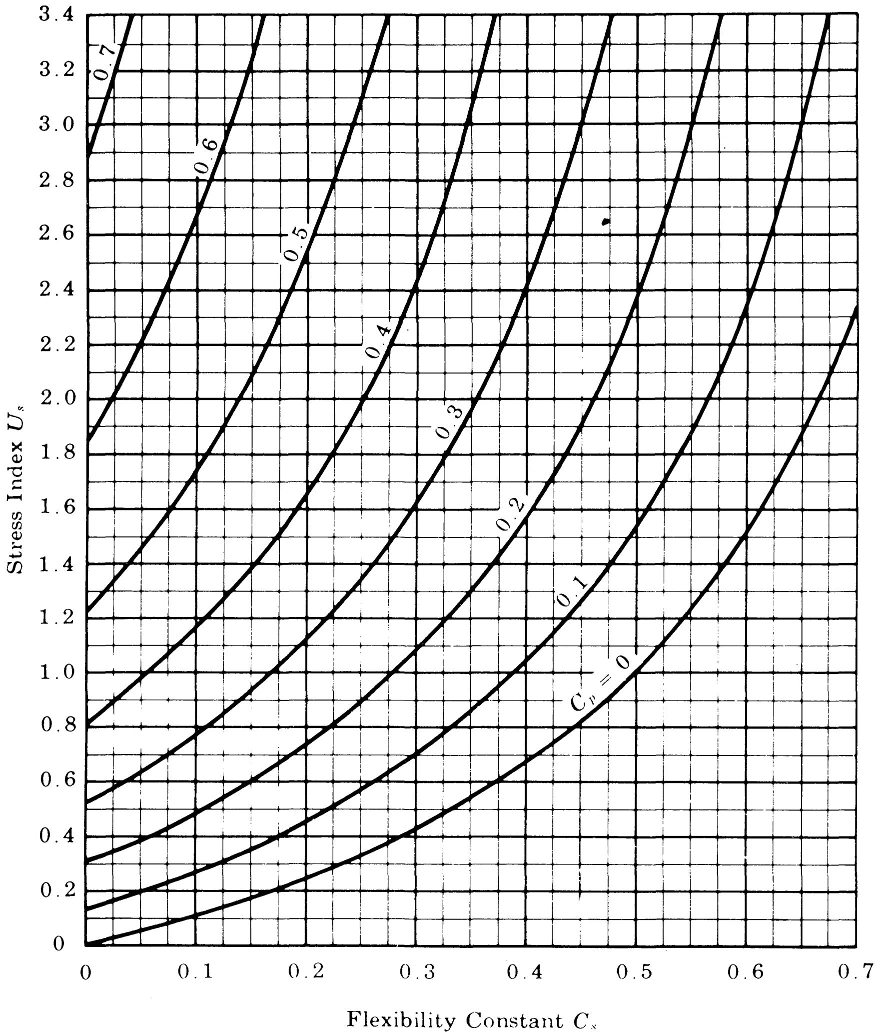


Fig. A-K2.2. Flexibility coefficients for secondary beams alone

where

L_p = column spacing in direction of girder (length of primary members), ft
L_s = column spacing perpendicular to direction of girder (length of secondary members), ft
S = spacing of secondary members, ft
I_p = moment of inertia of primary members, in.^{4}
I_s = moment of inertia of secondary members, in.^{4}

Roof framing consisting of a series of equally spaced wall-bearing beams is considered as consisting of secondary members supported on an infinitely stiff primary member. For this case, enter Fig. A-K2.2 with the computed stress index U_s. The limiting value of C_s is determined by the intercept of a horizontal line representing the U_s-value and the curve for $C_p = 0$.

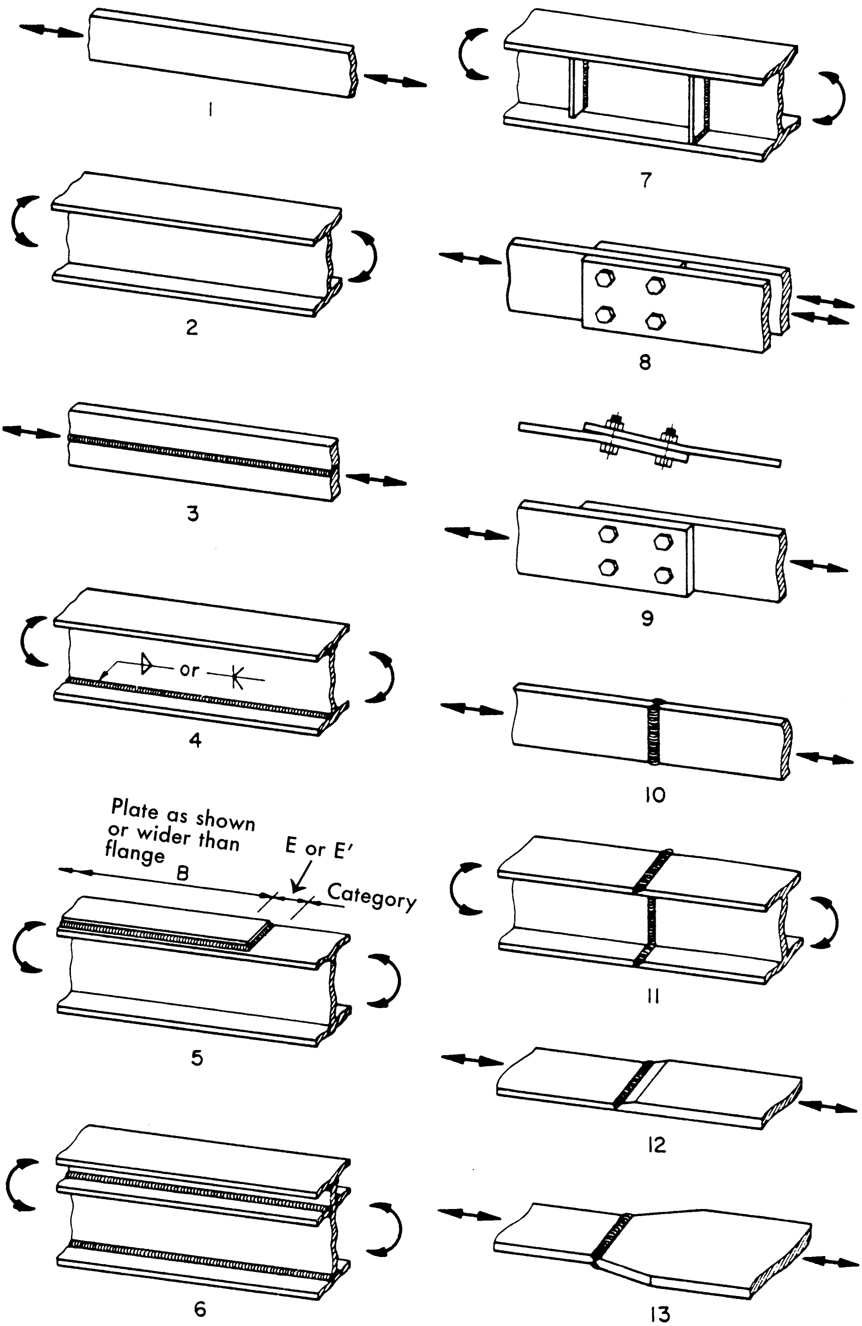


Fig. A-K4.1. Illustrative examples

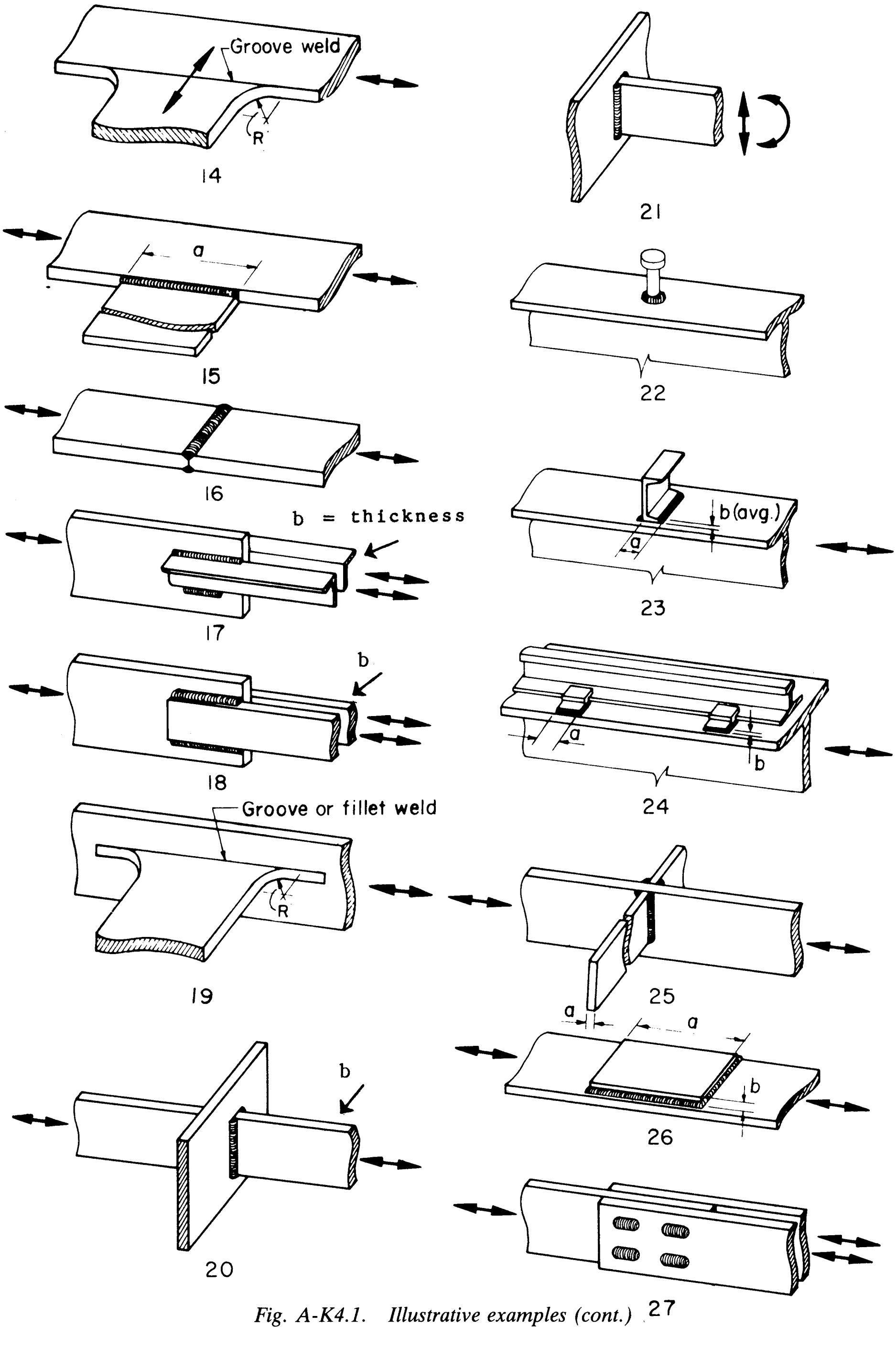


Fig. A-K4.1. Illustrative examples (cont.)

The ponding deflection contributed by a metal deck is usually such a small part of the total ponding deflection of a roof panel that it is sufficient merely to limit its moment of inertia (per foot of width normal to its span) to 0.000025 times the fourth power of its span length. However, the stability against ponding of a roof consisting of a metal roof deck of relatively slender depth-span ratio, spanning between beams supported directly on columns, may need to be checked. This can be done using Fig. A-K2.1 or A-K2.2 using as C_s the flexibility constant for one foot width of the roof deck ($S = 1.0$).

Since the shear rigidity of the web system of steel joists and trusses is less than that of a solid plate, their moment of inertia should be taken as 85% of their chords.

K4. FATIGUE

Members and connections subject to fatigue loading shall be proportioned in accordance with the provisions of this Appendix.

Fatigue, as used in this Specification, is defined as the damage that may result in fracture after a sufficient number of fluctuations of stress. Stress range is defined as the magnitude of these fluctuations. In the case of a stress reversal, the stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the sum of maximum shearing stresses of opposite direction at a given point, resulting from differing arrangement of live load.

1. Loading Conditions; Type and Location of Material

In the design of members and connections subject to repeated variation of live load, consideration shall be given to the number of stress cycles, the expected range of stress and the type and location of member or detail.

Loading conditions shall be classified according to Table A-K4.1.

The type and location of material shall be categorized according to Table A-K4.2.

2. Design Stress Range

The maximum range of stress at service loads shall not exceed the design stress range specified in Table A-K4.3.

3. Design Strength of Bolts in Tension

When subject to tensile fatigue loading, properly tightened A325 or A490 bolts shall be designed for the combined tensile design strength due to external and prying forces within limits given in Table A-K4.4.

TABLE A-K4.1
Number of Loading Cycles

Loading Condition	From	To
1	20,000[a]	100,000[b]
2	100,000	500,000[c]
3	500,000	2,000,000[d]
4	Over 2,000,000	

[a]Approximately equivalent to two applications every day for 25 years.
[b]Approximately equivalent to 10 applications every day for 25 years.
[c]Approximately equivalent to 50 applications every day for 25 years.
[d]Approximately equivalent to 200 applications every day for 25 years.

TABLE A-K4.2
Type and Location of Material

General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illustrative Example Nos. (see Fig. A-K4.1)[b]
Plain Material	Base metal with rolled or cleaned surface. Flame-cut edges with ANSI smoothness of 1,000 or less	T or Rev.	A	1,2
Built-up Members	Base metal and weld metal in members without attachments, built-up plates or shapes connected by continuous full-penetration groove welds or by continuous fillet welds parallel to the direction of applied stress	T or Rev.	B	3,4,5,6
	Base metal and weld metal in members without attachments, built-up plates, or shapes connected by full-penetration groove welds with backing bars not removed, or by partial-penetration groove welds parallel to the direction of applied stress	T or Rev.	B′	3,4,5,6
	Base metal at toe of welds on girder webs or flanges adjacent to welded transverse stiffeners	T or Rev.	C	7
	Base metal at ends of partial length welded coverplates narrower than the flange having square or tapered ends, with or without welds across the ends or wider than flange with welds across the ends Flange thickness ≤ 0.8 in. Flange thickness > 0.8 in.	 T or Rev. T or Rev.	 E E′	 5 5
	Base metal at end of partial length welded coverplates wider than the flange without welds across the ends		E′	5

[a]"T" signifies range in tensile stress only; "Rev." signifies a range involving reversal of tensile or compressive stress; "S" signifies range in shear, including shear stress reversal.
[b]These examples are provided as guidelines and are not intended to exclude other reasonably similar situations.
[c]Allowable fatigue stress range for transverse partial-penetration and transverse fillet welds is a function of the effective throat, depth of penetration and plate thickness. See Frank and Fisher, *Journal of the Structural Division*, Vol. 105 No. ST9, Sept. 1979.

TABLE A-K4.2 (cont'd) Type and Location of Material

General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illustrative Example Nos. (see Fig. A-K4.1)[b]
Groove Welds	Base metal and weld metal at full-penetration groove welded splices of parts of similar cross section ground flush, with grinding in the direction of applied stress and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of 9.25.2 or 9.25.3 of AWS D1.1-85	T or Rev.	B	10,11
	Base metal and weld metal at full-penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2½ with grinding in the direction of applied stress, and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of 9.25.2 or 9.25.3 of AWS D1.1-85			
	A514 base metal	T or Rev.	B′	12,13
	Other base metals	T or Rev.	B	12,13
	Base metal and weld metal at full-penetration groove welded splices, with or without transitions having slopes no greater than 1 to 2½ when reinforcement is not removed but weld soundness is established by radiographic or ultrasonic inspection in accordance with requirements of 9.25.2 or 9.25.3 of AWS D1.1-85	T or Rev.	C	10,11,12, 13
Partial-Penetration Groove Welds	Weld metal of partial-penetration transverse groove welds, based on effective throat area of the weld or welds	T or Rev.	F[c]	16

TABLE A-K4.2 (cont'd)
Type and Location of Material

General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illustrative Example Nos. (see Fig. A-K4.1)[b]
Fillet-welded Connections	Base metal at intermittent fillet welds	T or Rev.	E	
	Base metal at junction of axially loaded members with fillet-welded end connections. Welds shall be disposed about the axis of the member so as to balance weld stresses $b \leq 1$ in. $b > 1$ in.	 T or Rev. T or Rev.	 E E′	 17,18 17,18
	Base metal at members connected with transverse fillet welds $b \leq 1/2$ in. $b > 1/2$ in.	 T or Rev.	 C See Note c	 20,21
Fillet Welds	Weld metal of continuous or intermittent longitudinal or transverse fillet welds	S	F[c]	15,17,18 20,21
Plug or Slot Welds	Base metal at plug or slot welds	T or Rev.	E	27
	Shear on plug or slot welds	S	F	27
Mechanically Fastened Connections	Base metal at gross section of high-strength bolted slip-critical connections, except axially loaded joints which induce out-of-plane bending in connected material	T or Rev.	B	8
	Base metal at net section of other mechanically fastened joints	T or Rev.	D	8,9
	Base metal at net section of fully tensioned high-strength, bolted-bearing connections	T or Rev.	B	8,9

TABLE A-K4.2 (cont'd)
Type and Location of Material

General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illustrative Example Nos. (see Fig. A-K4.1)[b]
Attachments	Base metal at details attached by full-penetration groove welds subject to longitudinal and/or transverse loading when the detail embodies a transition radius R with the weld termination ground smooth and for transverse loading, the weld soundness established by radiographic or ultrasonic inspection in accordance with 9.25.2 or 9.25.3 of AWS D1.1-85			
	Longitudinal loading			
	$R > 24$ in.	T or Rev.	B	14
	24 in. $> R >$ 6 in.	T or Rev.	C	14
	6 in. $> R >$ 2 in.	T or Rev.	D	14
	2 in. $> R$	T or Rev.	E	14
	Detail base metal for transverse loading: equal thickness and reinforcement removed			
	$R > 24$ in.	T or Rev.	B	14
	24 in. $> R >$ 6 in.	T or Rev.	C	14
	6 in. $> R >$ 2 in.	T or Rev.	D	14
	2 in. $> R$	T or Rev.	E	14,15
	Detail base metal for transverse loading: equal thickness and reinforcement not removed			
	$R > 24$ in.	T or Rev.	C	14
	24 in. $> R >$ 6 in.	T or Rev.	C	14
	6 in. $> R >$ 2 in.	T or Rev.	D	14
	2 in. $> R$	T or Rev.	E	14,15

TABLE A-K4.2 (cont'd)
Type and Location of Material

General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illustrative Example Nos. (see Fig. A-K4.1)[b]
Attachments (cont'd)	Detail base metal for transverse loading: unequal thickness and reinforcement removed			
	$R > 2$ in.	T or Rev.	D	14
	2 in. $> R$	T or Rev.	E	14,15
	Detail base metal for transverse loading: unequal thickness and reinforcement not removed			
	all R	T or Rev.	E	14,15
	Detail base metal for transverse loading			
	$R > 6$ in.	T or Rev.	C	19
	6 in. $> R >$ 2 in.	T or Rev.	D	19
	2 in. $> R$	T or Rev.	E	19
	Base metal at detail attached by full-penetration groove welds subject to longitudinal loading			
	$2 < a < 12b$ or 4 in.	T or Rev.	D	15
	$a > 12b$ or 4 in. when $b \leq 1$ in.	T or Rev.	E	15
	$a > 12b$ or 4 in. when $b > 1$ in.	T or Rev.	E′	15
	Base metal at detail attached by fillet welds or partial-penetration groove welds subject to longitudinal loading			
	$a < 2$ in.	T or Rev.	C	15,23,24, 25,26
	2 in. $< a < 12b$ or 4 in.	T or Rev.	D	15,23, 24,26
	$a > 12b$ or 4 in. when $b \leq 1$ in.	T or Rev.	E	15,23, 24,26
	$a > 12b$ or 4 in. when $b > 1$ in.	T or Rev.	E′	15,23, 24,26

TABLE A-K4.2 (cont'd)
Type and Location of Material

General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illustrative Example Nos. (see Fig. A-K4.1)[b]
Attachments (cont'd)	Base metal attached by fillet welds or partial-penetration groove welds subjected to longitudinal loading when the weld termination embodies a transition radius with the weld termination ground smooth:			
	$R > 2$ in.	T or Rev.	D	19
	$R \leq 2$ in.	T or Rev.	E	19
	Fillet-welded attachments where the weld termination embodies a transition radius, weld termination ground smooth, and main material subject to longitudinal loading:			
	Detail base metal for transverse loading:			
	$R > 2$ in.	T or Rev.	D	19
	$R < 2$ in.	T or Rev.	E	19
	Base metal at stud-type shear connector attached by fillet weld or automatic end weld	T or Rev.	C	22
	Shear stress on nominal area of stud-type shear connectors	S	F	

TABLE A-K4.3
Allowable Stress Range, ksi

Category (From Table A-K4.2)	Loading Condition 1	Loading Condition 2	Loading Condition 3	Loading Condition 4
A	63	37	24	24
B	49	29	18	16
B′	39	23	15	12
C	35	21	13	10[a]
D	28	16	10	7
E	22	13	8	4.5
E′	16	9.2	5.8	2.6
F	15	12	9	8

[a]Flexural stress range of 12 ksi permitted at toe of stiffener welds or flanges.

TABLE A-K4.4
Design Strength of A325 or A490 Bolts Subject to Tension

Number of cycles	Design strength
Not more than 20,000	As specified in Section J3
From 20,000 to 500,000	$0.30\ A_b F_u$[a]
More than 500,000	$0.25\ A_b F_u$[a]

[a]At service loads.

Notes

NUMERICAL VALUES

TABLE 1
Design Strength as a Function of F_y

F_y (ksi)	Design Stress (ksi)				
	$0.54F_y$[a]	$0.56F_y$[b]	$0.63F_y$[c]	$0.85F_y$[d]	$0.90F_y$[e]
33	17.8	18.5	20.8	28.1	29.7
35	18.9	19.6	22.1	29.8	31.5
36	19.4	20.2	22.7	30.6	32.4
40	21.6	22.4	25.2	34.0	36.0
42	22.7	23.5	26.5	35.7	37.8
45	24.3	25.2	28.4	38.3	40.5
46	24.8	25.8	29.0	39.1	41.4
50	27.0	28.0	31.5	42.5	45.0
55	29.7	30.8	34.7	46.8	49.5
60	32.4	33.6	37.8	51.0	54.0
65	35.1	36.4	41.0	55.3	58.5
70	37.8	39.2	44.1	59.5	63.0
90	48.6	50.4	56.7	76.5	81.0
100	54.0	56.0	63.0	85.0	90.0

[a]See Sect. F2, Formulas F2-1 and F2-2
[b]See Sect. J5.2, Formula J5-3
[c]See Sect. K1.7, Formulas K1-9 and K1-10
[d]See Sect. E2, Formula E2-1
[e]See Sect. D1, Formula D1-1

Notes

TABLE 2
Design Strength as a Function of F_u

Item	ASTM Designation	F_y (ksi)	F_u (ksi)	Design Strength (ksi): Connected Part of Designated Steel: Tension $0.75 \times F_u$[a]	Design Strength (ksi): Connected Part of Designated Steel: Bearing $0.75 \times 2.4F_u$[b]	Design Strength (ksi): Bolt or Threaded Part of Designated Steel: Tension $0.75 \times 0.75F_u$[c]	Design Strength (ksi): Bolt or Threaded Part of Designated Steel: Shear $0.65 \times 0.45F_u$[d]	Design Strength (ksi): Bolt or Threaded Part of Designated Steel: Shear $0.65 \times 0.6F_u$[e]
Shapes, Plates, Bars, Sheet and Tubing or Threaded Parts	A36	36	58–80	43.5	104	32.6	17.0	22.6
	A53	35	60	45.0	108	—	—	—
	A242 A441 A588	50	70	52.5	126	39.4	20.5	27.3
		46	67	50.3	121	37.7	19.6	26.1
		42	63	47.3	113	35.4	18.4	24.6
		40[f]	60	45.0	108	33.8	17.6	23.4
	A500	33/39[g]	45	33.8	81	—	—	—
		42/46[g]	58	43.5	104	—	—	—
		46/50[g]	62	46.5	112	—	—	—
	A501	36	58	43.5	104	—	—	—
	A529	42	60–85	45.0	108	33.8	17.6	23.4
	A570	40	55	41.3	99	—	—	—
		42	58	43.5	104	—	—	—
	A572	42	60	45.0	108	33.8	17.6	23.4
		50	65	48.8	117	36.6	19.0	25.4
		60	75	56.3	135	42.2	21.9	29.3
		65	80	60.0	144	45.0	23.4	31.2
	A514	100	110–130	82.5	198	61.9	32.2	42.9
		90	100–130	75.0	180	56.3	29.3	39.0
	A606	45	65	48.8	117	—	—	—
		50	70	52.5	126	—	—	—
	A607	45	60	45.0	108	—	—	—
		50	65	48.8	117	—	—	—
		55	70	52.5	126	—	—	—
		60	75	56.3	135	—	—	—
		65	80	60.0	144	—	—	—
		70	85	63.8	153	—	—	—
	A618	50	70	52.5	126	—	—	—
		50	65	48.8	117	—	—	—
Bolts	A449	92	120	—	—	67.5	35.1	46.8
		81	105	—	—	59.1	30.7	41.0
		58	90	—	—	50.6	26.3	35.1

[a]On effective net area, see Sects. D1, J5.2.
[b]Produced by fastener in shear, see Sect. J3.6. Note that smaller maximum design bearing stresses, as a function of hole spacing, may be required by Sects. J3.9 and J3.10.
[c]On nominal body area, see Table J3.2.
[d]Threads not excluded from shear plane, see Table J3.2.
[e]Threads excluded from shear plane, see Table J3.2.
[f] For A441 material only.
[g]Smaller value for circular shapes, larger for square or rectangular shapes.
Note: For dimensional and size limitations, see the appropriate ASTM Specification.

TABLE 3-36
Design Stress for Compression Members of 36 ksi Specified Yield-stress Steel, $\phi_c = 0.85$[a]

$\frac{Kl}{r}$	$\phi_c F_{cr}$ (ksi)	$\frac{Kl}{r}$	$\phi_c F_{cr}$ (ksi)	$\frac{Kl}{r}$	$\phi_c F_{cr}$ (ksi)	$\frac{Kl}{r}$	$\phi_c F_{cr}$ (ksi)	$\frac{Kl}{r}$	$\phi_c F_{cr}$ (ksi)
1	30.60	41	28.01	81	21.66	121	14.16	161	8.23
2	30.59	42	27.89	82	21.48	122	13.98	162	8.13
3	30.59	43	27.76	83	21.29	123	13.80	163	8.03
4	30.57	44	27.64	84	21.11	124	13.62	164	7.93
5	30.56	45	27.51	85	20.92	125	13.44	165	7.84
6	30.54	46	27.37	86	20.73	126	13.27	166	7.74
7	30.52	47	27.24	87	20.54	127	13.09	167	7.65
8	30.50	48	27.11	88	20.36	128	12.92	168	7.56
9	30.47	49	26.97	89	20.17	129	12.74	169	7.47
10	30.44	50	26.83	90	19.98	130	12.57	170	7.38
11	30.41	51	26.68	91	19.79	131	12.40	171	7.30
12	30.37	52	26.54	92	19.60	132	12.23	172	7.21
13	30.33	53	26.39	93	19.41	133	12.06	173	7.13
14	30.29	54	26.25	94	19.22	134	11.88	174	7.05
15	30.24	55	26.10	95	19.03	135	11.71	175	6.97
16	30.19	56	25.94	96	18.84	136	11.54	176	6.89
17	30.14	57	25.79	97	18.65	137	11.37	177	6.81
18	30.08	58	25.63	98	18.46	138	11.20	178	6.73
19	30.02	59	25.48	99	18.27	139	11.04	179	6.66
20	29.96	60	25.32	100	18.08	140	10.89	180	6.59
21	29.90	61	25.16	101	17.89	141	10.73	181	6.51
22	29.83	62	24.99	102	17.70	142	10.58	182	6.44
23	29.76	63	24.83	103	17.51	143	10.43	183	6.37
24	29.69	64	24.67	104	17.32	144	10.29	184	6.30
25	29.61	65	24.50	105	17.13	145	10.15	185	6.23
26	29.53	66	24.33	106	16.94	146	10.01	186	6.17
27	29.45	67	24.16	107	16.75	147	9.87	187	6.10
28	29.36	68	23.99	108	16.56	148	9.74	188	6.04
29	29.28	69	23.82	109	16.37	149	9.61	189	5.97
30	29.18	70	23.64	110	16.19	150	9.48	190	5.91
31	29.09	71	23.47	111	16.00	151	9.36	191	5.85
32	28.99	72	23.29	112	15.81	152	9.23	192	5.79
33	28.90	73	23.12	113	15.63	153	9.11	193	5.73
34	28.79	74	22.94	114	15.44	154	9.00	194	5.67
35	28.69	75	22.76	115	15.26	155	8.88	195	5.61
36	28.58	76	22.58	116	15.07	156	8.77	196	5.55
37	28.47	77	22.40	117	14.89	157	8.66	197	5.50
38	28.36	78	22.22	118	14.70	158	8.55	198	5.44
39	28.25	79	22.03	119	14.52	159	8.44	199	5.39
40	28.13	80	21.85	120	14.34	160	8.33	200	5.33

[a]When element width-to-thickness ratio exceeds λ_r, see Appendix B5.3.

TABLE 3-50
Design Stress for Compression Members of 50 ksi Specified Yield-stress Steel, ϕ_c = 0.85[a]

$\frac{Kl}{r}$	$\phi_c F_{cr}$ (ksi)	$\frac{Kl}{r}$	$\phi_c F_{cr}$ (ksi)	$\frac{Kl}{r}$	$\phi_c F_{cr}$ (ksi)	$\frac{Kl}{r}$	$\phi_c F_{cr}$ (ksi)	$\frac{Kl}{r}$	$\phi_c F_{cr}$ (ksi)
1	42.50	41	37.59	81	26.31	121	14.57	161	8.23
2	42.49	42	37.36	82	26.00	122	14.33	162	8.13
3	42.47	43	37.13	83	25.68	123	14.10	163	8.03
4	42.45	44	36.89	84	25.37	124	13.88	164	7.93
5	42.42	45	36.65	85	25.06	125	13.66	165	7.84
6	42.39	46	36.41	86	24.75	126	13.44	166	7.74
7	42.35	47	36.16	87	24.44	127	13.23	167	7.65
8	42.30	48	35.91	88	24.13	128	13.02	168	7.56
9	42.25	49	35.66	89	23.82	129	12.82	169	7.47
10	42.19	50	35.40	90	23.51	130	12.62	170	7.38
11	42.13	51	35.14	91	23.20	131	12.43	171	7.30
12	42.05	52	34.88	92	22.89	132	12.25	172	7.21
13	41.98	53	34.61	93	22.58	133	12.06	173	7.13
14	41.90	54	34.34	94	22.28	134	11.88	174	7.05
15	41.81	55	34.07	95	21.97	135	11.71	175	6.97
16	41.71	56	33.79	96	21.67	136	11.54	176	6.89
17	41.61	57	33.51	97	21.36	137	11.37	177	6.81
18	41.51	58	33.23	98	21.06	138	11.20	178	6.73
19	41.39	59	32.95	99	20.76	139	11.04	179	6.66
20	41.28	60	32.67	100	20.46	140	10.89	180	6.59
21	41.15	61	32.38	101	20.16	141	10.73	181	6.51
22	41.02	62	32.09	102	19.86	142	10.58	182	6.44
23	40.89	63	31.80	103	19.57	143	10.43	183	6.37
24	40.75	64	31.50	104	19.28	144	10.29	184	6.30
25	40.60	65	31.21	105	18.98	145	10.15	185	6.23
26	40.45	66	30.91	106	18.69	146	10.01	186	6.17
27	40.29	67	30.61	107	18.40	147	9.87	187	6.10
28	40.13	68	30.31	108	18.12	148	9.74	188	6.04
29	39.97	69	30.01	109	17.83	149	9.61	189	5.97
30	39.79	70	29.70	110	17.55	150	9.48	190	5.91
31	39.62	71	29.40	111	17.27	151	9.36	191	5.85
32	39.43	72	29.09	112	16.99	152	9.23	192	5.79
33	39.25	73	28.79	113	16.71	153	9.11	193	5.73
34	39.06	74	28.48	114	16.42	154	9.00	194	5.67
35	38.86	75	28.17	115	16.13	155	8.88	195	5.61
36	38.66	76	27.86	116	15.86	156	8.77	196	5.55
37	38.45	77	27.55	117	15.59	157	8.66	197	5.50
38	38.24	78	27.24	118	15.32	158	8.55	198	5.44
39	38.03	79	26.93	119	15.07	159	8.44	199	5.39
40	37.81	80	26.62	120	14.82	160	8.33	200	5.33

[a]When element width-to-thickness ratio exceeds λ_r, see Appendix B5.3.

TABLE 4
Values of $\phi_c F_{cr}/F_y$, $\phi_c = 0.85$
For Determining Design Stress for Compression Members for Steel of Any Yield Stress[a]

All grades of steel

λ_c	$\phi_c F_{cr}/F_y$	λ_c	$\phi_c F_{cr}/F_y$	λ_c	$\phi_c F_{cr}/F_y$	λ_c	$\phi_c F_{cr}/F_y$
0.02	0.850	0.82	0.641	1.62	0.284	2.42	0.127
0.04	0.849	0.84	0.632	1.64	0.277	2.44	0.125
0.06	0.849	0.86	0.623	1.66	0.271	2.46	0.123
0.08	0.848	0.88	0.614	1.68	0.264	2.48	0.121
0.10	0.846	0.90	0.605	1.70	0.258	2.50	0.119
0.12	0.845	0.92	0.596	1.72	0.252	2.52	0.117
0.14	0.843	0.94	0.587	1.74	0.246	2.54	0.116
0.16	0.841	0.96	0.578	1.76	0.241	2.56	0.114
0.18	0.839	0.98	0.568	1.78	0.235	2.58	0.112
0.20	0.836	1.00	0.559	1.80	0.230	2.60	0.110
0.22	0.833	1.02	0.550	1.82	0.225	2.62	0.109
0.24	0.830	1.04	0.540	1.84	0.220	2.64	0.107
0.26	0.826	1.06	0.531	1.86	0.215	2.66	0.105
0.28	0.823	1.08	0.521	1.88	0.211	2.68	0.104
0.30	0.819	1.10	0.512	1.90	0.206	2.70	0.102
0.32	0.814	1.12	0.503	1.92	0.202	2.72	0.101
0.34	0.810	1.14	0.493	1.94	0.198	2.74	0.099
0.36	0.805	1.16	0.484	1.96	0.194	2.76	0.098
0.38	0.800	1.18	0.474	1.98	0.190	2.78	0.096
0.40	0.795	1.20	0.465	2.00	0.186	2.80	0.095
0.42	0.789	1.22	0.456	2.02	0.183	2.82	0.094
0.44	0.784	1.24	0.446	2.04	0.179	2.84	0.092
0.46	0.778	1.26	0.437	2.06	0.176	2.86	0.091
0.48	0.772	1.28	0.428	2.08	0.172	2.88	0.090
0.50	0.765	1.30	0.419	2.10	0.169	2.90	0.089
0.52	0.759	1.32	0.410	2.12	0.166	2.92	0.087
0.54	0.752	1.34	0.401	2.14	0.163	2.94	0.086
0.56	0.745	1.36	0.392	2.16	0.160	2.96	0.085
0.58	0.738	1.38	0.383	2.18	0.157	2.98	0.084
0.60	0.731	1.40	0.374	2.20	0.154	3.00	0.083
0.62	0.724	1.42	0.365	2.22	0.151	3.02	0.082
0.64	0.716	1.44	0.357	2.24	0.149	3.04	0.081
0.66	0.708	1.46	0.348	2.26	0.146	3.06	0.080
0.68	0.700	1.48	0.339	2.28	0.143	3.08	0.079
0.70	0.692	1.50	0.331	2.30	0.141	3.10	0.078
0.72	0.684	1.52	0.323	2.32	0.138	3.12	0.077
0.74	0.676	1.54	0.314	2.34	0.136	3.14	0.076
0.76	0.667	1.56	0.306	2.36	0.134	3.16	0.075
0.78	0.659	1.58	0.299	2.38	0.132	3.18	0.074
0.80	0.650	1.60	0.291	2.40	0.129	3.20	0.073

[a]When element width-to-thickness ratios exceed λ_r, see Appendix B5.3.
Values of $\lambda_c > 2.24$ exceed Kl/r of 200 for $F_y = 36$.
Values of $\lambda_c > 2.64$ exceed Kl/r of 200 for $F_y = 50$.

TABLE 5
Values of Kl/r for F_y = 36 and 50 ksi

λ_c	Kl/r $F_y = 36$	Kl/r $F_y = 50$	λ_c	Kl/r $F_y = 36$	Kl/r $F_y = 50$
0.02	1.8	1.5	0.82	73.1	62.0
0.04	3.6	3.0	0.84	74.9	63.6
0.06	5.3	4.5	0.86	76.7	65.1
0.08	7.1	6.1	0.88	78.5	66.6
0.10	8.9	7.6	0.90	80.2	68.1
0.12	10.7	9.1	0.92	82.0	69.6
0.14	12.5	10.6	0.94	83.8	71.1
0.16	14.3	12.1	0.96	85.6	72.6
0.18	16.0	13.6	0.98	87.4	74.1
0.20	17.8	15.1	1.00	89.2	75.7
0.22	19.6	16.6	1.02	90.9	77.2
0.24	21.4	18.2	1.04	92.7	78.7
0.26	23.2	19.7	1.06	94.5	80.2
0.28	25.0	21.2	1.08	96.3	81.7
0.30	26.7	22.7	1.10	98.1	83.2
0.32	28.5	24.2	1.12	99.9	84.7
0.34	30.3	25.7	1.14	101.6	86.3
0.36	32.1	27.2	1.16	103.4	87.8
0.38	33.9	28.8	1.18	105.2	89.3
0.40	35.7	30.3	1.20	107.0	90.8
0.42	37.4	31.8	1.22	108.8	92.3
0.44	39.2	33.3	1.24	110.6	93.8
0.46	41.0	34.8	1.26	112.3	95.3
0.48	42.8	36.3	1.28	114.1	96.8
0.50	44.6	37.8	1.30	115.9	98.4
0.52	46.4	39.3	1.32	117.7	99.9
0.54	48.1	40.9	1.34	119.5	101.4
0.56	49.9	42.4	1.36	121.3	102.9
0.58	51.7	43.9	1.38	123.0	104.4
0.60	53.5	45.4	1.40	124.8	105.9
0.62	55.3	46.9	1.42	126.6	107.4
0.64	57.1	48.4	1.44	128.4	108.9
0.66	58.8	49.9	1.46	130.2	110.5
0.68	60.6	51.4	1.48	132.0	112.0
0.70	62.4	53.0	1.50	133.7	113.5
0.72	64.2	54.5	1.52	135.5	115.0
0.74	66.0	56.0	1.54	137.3	116.5
0.76	67.8	57.5	1.56	139.1	118.0
0.78	69.5	59.0	1.58	140.9	119.5
0.80	71.3	60.5	1.60	142.7	121.1

TABLE 5 (cont'd)
Values of Kl/r for F_y = 36 and 50 ksi

λ_c	Kl/r $F_y = 36$	Kl/r $F_y = 50$	λ_c	Kl/r $F_y = 50$
1.62	144.4	122.6	2.42	183.1
1.64	146.2	124.1	2.44	184.6
1.66	148.0	125.6	2.46	186.1
1.68	149.8	127.1	2.48	187.6
1.70	151.6	128.6	2.50	189.1
1.72	153.4	130.1	2.52	190.7
1.74	155.1	131.6	2.54	192.2
1.76	156.9	133.2	2.56	193.7
1.78	158.7	134.7	2.58	195.2
1.80	160.5	136.2	2.60	196.7
1.82	162.3	137.7	2.62	198.2
1.84	164.1	139.2	2.64	199.7
1.86	165.8	140.7		
1.88	167.6	142.2		
1.90	169.4	143.8		
1.92	171.2	145.3		
1.94	173.0	146.8		
1.96	174.8	148.3		
1.98	176.5	149.8		
2.00	178.3	151.3		
2.02	180.1	152.8		
2.04	181.9	154.3		
2.06	183.7	155.9		
2.08	185.5	157.4		
2.10	187.2	158.9		
2.12	189.0	160.4		
2.14	190.8	161.9		
2.16	192.6	163.4		
2.18	194.4	164.9		
2.20	196.2	166.5		
2.22	197.9	168.0		
2.24	199.7	169.5		
2.26		171.0		
2.28		172.5		
2.30		174.0		
2.32		175.5		
2.34		177.0		
2.36		178.6		
2.38		180.1		
2.40		181.6		

Heavy line indicates Kl/r of 200.

TABLE 6
Slenderness Ratios of Elements as a Function of F_y
From Table B5.1

Ratio	F_y (ksi)					
	36	42	46	50	60	65
$65/\sqrt{F_y}$	10.8	10.0	9.6	9.2	8.4	8.1
$76/\sqrt{F_y}$	12.7	11.7	11.2	10.7	9.8	9.4
$95/\sqrt{F_y}$	15.8	14.7	14.0	13.4	12.3	11.8
$106/\sqrt{F_y-16.5}$	24.0	21.0	19.5	18.3	16.1	15.2
$127/\sqrt{F_y}$	21.2	19.6	18.7	18.0	16.4	15.8
$141/\sqrt{F_y-10}$	27.7	24.9	23.5	22.3	19.9	19.0
$190/\sqrt{F_y}$	31.7	29.3	28.0	26.9	24.5	23.6
$238/\sqrt{F_y-10}$	46.7	42.1	39.7	37.6	33.7	32.1
$238/\sqrt{F_y-16.5}$	53.9	47.1	43.8	41.1	36.1	34.2
$253/\sqrt{F_y}$	42.2	39.0	37.3	35.8	32.7	31.4
$317/\sqrt{F_y-10}$	62.2	56.0	52.8	50.1	44.8	42.7
$640/\sqrt{F_y}$	107	98.8	94.4	90.5	82.6	79.4
$970/\sqrt{F_y}$	162	150	143	137	125	120
$1{,}300/F_y$	36.1	31.0	28.3	26.0	21.7	20.0
$2{,}070/F_y$	57.5	49.3	45.0	41.4	34.5	31.8
$3{,}300/F_y$	91.7	78.6	71.7	66.0	55.0	50.8
$8{,}970/F_y$	249	214	195	179	150	138

TABLE 7
Values of C_b
For Use in Chapters F and G

$\frac{M_1}{M_2}$	C_b	$\frac{M_1}{M_2}$	C_b	$\frac{M_1}{M_2}$	C_b
−1.00	1.00	−0.45	1.34	0.10	1.86
−0.95	1.02	−0.40	1.38	0.15	1.91
−0.90	1.05	−0.35	1.42	0.20	1.97
−0.85	1.07	−0.30	1.46	0.25	2.03
−0.80	1.10	−0.25	1.51	0.30	2.09
−0.75	1.13	−0.20	1.55	0.35	2.15
−0.70	1.16	−0.15	1.60	0.40	2.22
−0.65	1.19	−0.10	1.65	0.45	2.28
−0.60	1.23	−0.05	1.70	≥0.47	2.30
−0.55	1.26	0	1.75		
−0.50	1.30	0.05	1.80		

Note 1: $C_b = 1.75 + 1.05(M_1/M_2) + 0.3\,(M_1/M_2)^2 \leq 2.3$
Note 2: M_1/M_2 positive for reverse curvature and negative for single curvature.

TABLE 8
Values of C_m
For Use in Section H1

$\frac{M_1}{M_2}$	C_m	$\frac{M_1}{M_2}$	C_m	$\frac{M_1}{M_2}$	C_m
−1.00	1.00	−0.45	0.78	0.10	0.56
−0.95	0.98	−0.40	0.76	0.15	0.54
−0.90	0.96	−0.35	0.74	0.20	0.52
−0.85	0.94	−0.30	0.72	0.25	0.50
−0.80	0.92	−0.25	0.70	0.30	0.48
−0.75	0.90	−0.20	0.68	0.35	0.46
−0.70	0.88	−0.15	0.66	0.40	0.44
−0.65	0.86	−0.10	0.64	0.45	0.42
−0.60	0.84	−0.05	0.62	0.50	0.40
				0.60	0.36
−0.55	0.82	0	0.60	0.80	0.28
−0.50	0.80	0.05	0.58	1.00	0.20

Note 1: $C_m = 0.6 - 0.4(M_1/M_2)$.
Note 2: M_1/M_2 is positive for reverse curvature and negative for single curvature.

TABLE 9
Values of P_e/A_g
For Use in Section H1 for Steel of Any Yield Stress

$\frac{Kl}{r}$	P_e/A_g (ksi)	$\frac{Kl}{r}$	P_e/A_g (ksi)	$\frac{Kl}{r}$	P_e/A_g (ksi)	$\frac{Kl}{r}$	P_e/A_g (ksi)	$\frac{Kl}{r}$	P_e/A_g (ksi)	$\frac{Kl}{r}$	P_e/A_g (ksi)
21	649.02	51	110.04	81	43.62	111	23.23	141	14.40	171	9.79
22	591.36	52	105.85	82	42.57	112	22.82	142	14.19	172	9.67
23	541.06	53	101.89	83	41.55	113	22.42	143	14.00	173	9.56
24	496.91	54	98.15	84	40.56	114	22.02	144	13.80	174	9.45
25	457.95	55	94.62	85	39.62	115	21.64	145	13.61	175	9.35
26	423.40	56	91.27	86	38.70	116	21.27	146	13.43	176	9.24
27	392.62	57	88.09	87	37.81	117	20.91	147	13.25	177	9.14
28	365.07	58	85.08	88	36.96	118	20.56	148	13.07	178	9.03
29	340.33	59	82.22	89	36.13	119	20.21	149	12.89	179	8.93
30	318.02	60	79.51	90	35.34	120	19.88	150	12.72	180	8.83
31	297.83	61	76.92	91	34.56	121	19.55	151	12.55	181	8.74
32	279.51	62	74.46	92	33.82	122	19.23	152	12.39	182	8.64
33	262.83	63	72.11	93	33.09	123	18.92	153	12.23	183	8.55
34	247.59	64	69.88	94	32.39	124	18.61	154	12.07	184	8.45
35	233.65	65	67.74	95	31.71	125	18.32	155	11.91	185	8.36
36	220.85	66	65.71	96	31.06	126	18.03	156	11.76	186	8.27
37	209.07	67	63.76	97	30.42	127	17.75	157	11.61	187	8.18
38	198.21	68	61.90	98	29.80	128	17.47	158	11.47	188	8.10
39	188.18	69	60.12	99	29.20	129	17.20	159	11.32	189	8.01
40	178.89	70	58.41	100	28.62	130	16.94	160	11.18	190	7.93
41	170.27	71	56.78	101	28.06	131	16.68	161	11.04	191	7.85
42	162.26	72	55.21	102	27.51	132	16.43	162	10.91	192	7.76
43	154.80	73	53.71	103	26.98	133	16.18	163	10.77	193	7.68
44	147.84	74	52.57	104	26.46	134	15.94	164	10.64	194	7.60
45	141.34	75	50.88	105	25.96	135	15.70	165	10.51	195	7.53
46	135.26	76	49.55	106	25.47	136	15.47	166	10.39	196	7.45
47	129.57	77	48.27	107	25.00	137	15.25	167	10.26	197	7.38
48	124.23	78	47.04	108	24.54	138	15.03	168	10.14	198	7.30
49	119.21	79	45.86	109	24.09	139	14.81	169	10.02	199	7.23
50	114.49	80	44.72	110	23.65	140	14.60	170	9.90	200	7.16

Note: $P_e/A_g = \dfrac{\pi^2 E}{(Kl/r)^2}$

TABLE 10-36

$\frac{\phi_v V_n}{A_w}$ (ksi) for Plate Girders by Section F2

For 36 ksi Yield-stress Steel, Tension Field Action Not Included

$\frac{h}{t_w}$	Aspect Ratio a/h: Stiffener Spacing to Web Depth													
	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	Over 3.0
60	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4
70	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4
80	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	18.9	18.2	17.9	16.9
90	19.4	19.4	19.4	19.4	19.4	19.4	19.4	18.5	17.8	17.2	16.8	16.2	15.9	14.7
100	19.4	19.4	19.4	19.4	19.4	19.2	17.6	16.6	16.0	15.5	14.9	13.8	13.2	11.9
110	19.4	19.4	19.4	19.4	18.4	17.4	16.0	14.8	13.7	12.8	12.3	11.4	10.9	9.8
120	19.4	19.4	19.4	18.1	16.9	16.0	14.0	12.5	11.5	10.8	10.3	9.6	9.2	8.3
130	19.4	19.4	18.2	16.7	15.6	14.1	11.9	10.6	9.8	9.2	8.8	8.2	7.8	7.0
140	19.4	18.8	16.9	15.5	13.5	12.1	10.3	9.2	8.4	7.9	7.6	7.0	6.7	6.1
150	19.4	17.6	15.7	13.5	11.8	10.6	8.9	8.0	7.3	6.9	6.6	6.1	5.9	5.3
160	18.9	16.5	14.1	11.9	10.4	9.3	7.9	7.0	6.5	6.1	5.8	5.4		4.6
170	17.8	15.5	12.5	10.5	9.2	8.2	7.0	6.2	5.7	5.4	5.1			4.1
180	16.8	13.9	11.1	9.4	8.2	7.3	6.2	5.5	5.1	4.8	4.6			3.7
200	14.9	11.2	9.0	7.6	6.6	5.9	5.0	4.5	4.1					3.0
220	12.3	9.3	7.5	6.3	5.5	4.9	4.2							2.5
240	10.3	7.8	6.3	5.3	4.6	4.1								2.1
260	8.8	6.6	5.3	4.5	3.9	3.5								1.8
280	7.6	5.7	4.6	3.9										
300	6.6	5.0	4.0											
320	5.8	4.4												

TABLE 10-50

$\dfrac{\phi_v V_n}{A_w}$ (ksi) for Plate Girders by Section F2

For 50 ksi Yield-stress Steel, Tension Field Action Not Included

$\dfrac{h}{t_w}$	Aspect Ratio a/h: Stiffener Spacing to Web Depth													
	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	Over 3.0
60	27.0	27.0	27.0	27.0	27.0	27.0	27.0	27.0	27.0	27.0	27.0	27.0	27.0	26.6
70	27.0	27.0	27.0	27.0	27.0	27.0	27.0	27.0	26.9	26.1	25.5	24.6	24.0	22.8
80	27.0	27.0	27.0	27.0	27.0	27.0	26.0	24.5	23.5	22.8	22.3	21.5	20.6	18.6
90	27.0	27.0	27.0	27.0	26.5	25.1	23.1	21.8	20.4	19.2	18.3	17.0	16.3	14.7
100	27.0	27.0	27.0	25.6	23.9	22.6	20.1	17.9	16.5	15.5	14.9	13.8	13.2	11.9
110	27.0	27.0	25.3	23.2	21.7	19.6	16.6	14.8	13.7	12.8	12.3	11.4	10.9	9.8
120	27.0	25.9	23.2	21.1	18.4	16.5	14.0	12.5	11.5	10.8	10.3	9.6	9.2	8.3
130	27.0	23.9	21.4	18.0	15.7	14.1	11.9	10.6	9.8	9.2	8.8	8.2	7.8	7.0
140	25.5	22.2	18.4	15.5	13.5	12.1	10.3	9.2	8.4	7.9	7.6	7.0	6.7	6.1
150	23.8	19.9	16.1	13.5	11.8	10.6	8.9	8.0	7.3	6.9	6.6	6.1	5.9	5.3
160	22.3	17.5	14.1	11.9	10.4	9.3	7.9	7.0	6.5	6.1	5.8	5.4		4.6
170	20.6	15.5	12.5	10.5	9.2	8.2	7.0	6.2	5.7	5.4	5.1			4.1
180	18.3	13.9	11.1	9.4	8.2	7.3	6.2	5.5	5.1	4.8	4.6			3.7
200	14.9	11.2	9.0	7.6	6.6	5.9	5.0	4.5	4.1					3.0
220	12.3	9.3	7.5	6.3	5.5	4.9	4.2							2.5
240	10.3	7.8	6.3	5.3	4.6	4.1								2.1
260	8.8	6.6	5.3	4.5	3.9	3.5								
280	7.6	5.7	4.6	3.9										

TABLE 11-36

$\frac{\phi_v V_n}{A_w}$ (ksi) for Plate Girders by Appendix G

For 36 ksi Yield-stress Steel, Tension Field Action Included[b]
(*Italic* values indicate gross area, as percent of ($h \times t_w$) required for pairs of intermediate stiffeners of 36 ksi yield-stress steel with $V_u/\phi V_n = 1.0$.)[a]

$\frac{h}{t_w}$	Aspect Ratio a/h: Stiffener Spacing to Web Depth												
	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	Over 3.0[c]
60	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4
70	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4
80	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.1	18.6	18.3	16.9
90	19.4	19.4	19.4	19.4	19.4	19.4	19.4	19.0	18.5	18.2	17.8	17.3	16.8	14.7
100	19.4	19.4	19.4	19.4	19.4	19.3	18.6	18.1	17.6	17.2	16.6	15.6	14.9	11.9
110	19.4	19.4	19.4	19.4	19.1	18.7	17.9	17.2	16.3	15.6	15.1	14.0	13.3	9.8
120	19.4	19.4	19.4	19.0	18.5	18.1	17.0	16.0	15.1	14.4	13.9	12.8	12.0	8.3
130	19.4	19.4	19.1	18.6	18.1	17.4	16.1	15.1	14.2	13.5	12.9	11.8	11.0	7.0
140	19.4	19.3	18.7	18.2	17.4	16.6	15.4	14.4	13.5	12.8	12.2	11.0	10.2	6.1
150	19.4	19.0	18.4	17.5	16.7	16.0	14.8	13.8	12.9	12.2	11.6	10.4	9.6	5.3
160	19.3	18.7	17.9	17.0	16.2	15.5	14.3	13.3	12.4	11.7	11.1	9.9		4.6
170	19.1	18.4	17.4	16.6	15.8	15.1	13.9	12.9	12.0	11.3 *0.3*	10.7 *0.4*			4.1
180	18.9	18.0	17.1	16.2	15.5	14.8	13.6 *0.2*	12.6 *0.7*	11.7 *1.1*	11.0 *1.3*	10.4 *1.5*			3.7
200	18.4	17.3	16.4	15.6 *0.1*	14.9 *0.9*	14.2 *1.4*	13.1 *2.1*	12.0 *2.5*	11.2 *2.8*					3.0
220	17.8	16.9	16.0 *1.1*	15.2 *2.0*	14.5 *2.6*	13.8 *3.0*	12.7 *3.6*							2.5
240	17.4	16.5 *1.5*	15.7 *2.7*	14.9 *3.4*	14.2 *3.9*	13.5 *4.3*								2.1
260	17.1 *1.3*	16.2 *3.0*	15.4 *4.0*	14.6 *4.6*	14.0 *5.0*	13.3 *5.4*								1.8
280	16.8 *2.7*	16.0 *4.2*	15.2 *5.0*	14.4 *5.6*										
300	16.6 *3.9*	15.8 *5.2*	15.0 *5.9*											
320	16.4 *4.9*	15.6 *6.0*												

[a]For area of single-angle and single-plate stiffeners, or when $V_u/\phi V_n < 1.0$, see Formula A-G4-2.
[b]For end-panels and all panels in hybrid and web-tapered plate girders use Table 10-36.
[c]Same as for Table 10-36.
Note: Girders so proportioned that the computed shear is less than that given in right-hand column do not require intermediate stiffeners.

TABLE 11-50

$\dfrac{\phi_v V_n}{A_w}$ (ksi) for Plate Girders by Appendix G

For 50 ksi Yield-stress Steel, Tension Field Action Included[b]
(*Italic* values indicate gross area, as percent of ($h \times t_w$) required for pairs of intermediate stiffeners of 50 ksi yield-stress steel with $V_u/\phi V_n = 1.0$.)[a]

$\frac{h}{t_w}$	Aspect Ratio a/h: Stiffener Spacing to Web Depth													
	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	Over 3.0[c]
60	27.0	27.0	27.0	27.0	27.0	27.0	27.0	27.0	27.0	27.0	27.0	27.0	27.0	26.6
70	27.0	27.0	27.0	27.0	27.0	27.0	27.0	27.0	26.9	26.5	26.1	25.4	24.9	22.8
80	27.0	27.0	27.0	27.0	27.0	27.0	26.5	25.8	25.1	24.6	24.1	23.3	22.4	18.6
90	27.0	27.0	27.0	27.0	26.8	26.3	25.3	24.4	23.4	22.5	21.7	20.2	19.2	14.7
100	27.0	27.0	27.0	26.5	25.9	25.3	24.0	22.5	21.4	20.4	19.6	18.0	17.0	11.9
110	27.0	27.0	26.5	25.8	25.1	24.2	22.4	21.0	19.8	18.8	18.0	16.4	15.3	9.8
120	27.0	26.7	25.9	25.1	24.0	23.0	21.2	19.8	18.6	17.6	16.8	15.2	14.1	8.3
130	27.0	26.2	25.4	24.1	23.0	22.0	20.3	18.9	17.7	16.7	15.9	14.2	13.1	7.0
140	26.7	25.8	24.5	23.3	22.2	21.3	19.6	18.2	17.0	16.0	15.1	13.5	12.3	6.1
150	26.3	25.2	23.9	22.7	21.6	20.7	19.0	17.6	16.4	15.4	14.5	12.9	11.7	5.3
160	26.0	24.6	23.3	22.2	21.1	20.2	18.5	17.1	15.9 *0.2*	14.9 *0.4*	14.0 *0.5*	12.4 *0.8*		4.6
170	25.6	24.1	22.8	21.7	20.7	19.8	18.1 *0.5*	16.7 *1.0*	15.5 *1.2*	14.5 *1.4*	13.6 *1.6*			4.1
180	25.1	23.7	22.4	21.3	20.3 *0.4*	19.4 *0.9*	17.8 *1.5*	16.4 *1.9*	15.2 *2.2*	14.2 *2.3*	13.3 *2.5*			3.7
200	24.3	23.0	21.8 *1.0*	20.8 *1.8*	19.8 *2.3*	18.9 *2.7*	17.3 *3.2*	15.9 *3.5*	14.7 *3.7*					3.0
220	23.7	22.5 *1.7*	21.4 *2.7*	20.4 *3.3*	19.4 *3.8*	18.5 *4.1*	16.9 *4.5*							2.5
240	23.2 *1.8*	22.1 *3.2*	21.0 *4.0*	20.0 *4.6*	19.1 *4.9*	18.2 *5.2*								2.1
260	23.0 *3.2*	21.8 *4.4*	20.8 *5.1*	19.8 *5.6*	18.8 *5.9*	18.0 *6.1*								
280	22.7 *4.4*	21.6 *5.4*	20.6 *6.0*	19.6 *6.4*										

[a]For area of single-angle and single-plate stiffeners, or when $V_u/\phi V_n < 1.0$, see Formula A-G4-2.
[b]For end-panels and all panels in hybrid and web-tapered plate girders use Table 10-50.
[c]Same as for Table 10-50.
Note: Girders so proportioned that the computed shear is less than that given in right-hand column do not require intermediate stiffeners.

TABLE 12
Nominal Horizontal Shear Load for One Connector Q_n, kips[a]
From Formulas I5-1 and I5-2

Connector[b]	Specified Compressive Strength of Concrete, f_c', ksi[d]		
	3.0	3.5	4.0
½-in. dia. × 2-in. hooked or headed stud	9.4	10.5	11.6
⅝-in. dia. × 2½-in. hooked or headed stud	14.6	16.4	18.1
¾-in. dia. × 3-in. hooked or headed stud	21.0	23.6	26.1
⅞-in. dia. × 3½-in. hooked or headed stud	28.6	32.1	35.5
Channel C3 × 4.1	10.2 L_c[c]	11.5 L_c[c]	12.7 L_c[c]
Channel C4 × 5.4	11.1 L_c[c]	12.4 L_c[c]	13.8 L_c[c]
Channel C5 × 6.7	11.9 L_c[c]	13.3 L_c[c]	14.7 L_c[c]

[a]Applicable only to concrete made with ASTM C33 aggregates.
[b]The design horizontal loads tabulated may also be used for studs longer than shown.
[c]L_c = length of channel, inches.
[d]$F_u > 0.5(f_c'w)^{.75}$, w = 145 lbs./cu.ft.

TABLE 13
Coefficients for Use with Concrete Made with C330 Aggregates to Adjust Values from Table 12 for Lightweight Concrete

Specified Compressive Strength of Concrete (f_c')	Air Dry Unit Weight of Concrete, pcf						
	90	95	100	105	110	115	120
≤ 4.0 ksi	0.73	0.76	0.78	0.81	0.83	0.86	0.88
≥ 5.0 ksi	0.82	0.85	0.87	0.91	0.93	0.96	0.99

Commentary

on the Load and Resistance Factor Design Specification for Structural Steel Buildings (September 1, 1986)

INTRODUCTION

This Commentary provides information on the basis and limitations of various provisions of the LRFD Specification, so that designers, fabricators and erectors (users) can make more efficient use of the Specification. The Commentary and Specification documents do not attempt to anticipate and/or set forth all the questions or possible problems that may be encountered, or situations in which special consideration and engineering judgment should be exercised in using the documents. Such a recitation would make the documents unduly lengthy and cumbersome. WARNING is given that AISC assumes that the users of its documents are competent in their fields of endeavor and are informed on current developments and findings related to their fields.

CHAPTER A.
GENERAL PROVISIONS

A1. SCOPE

Load and Resistance Factor Design (LRFD) is an improved approach to the design of structural steel for buildings. It involves explicit consideration of limit states, multiple load factors and resistance factors, and implicit probabilistic determination of reliability. The designation LRFD reflects the concept of factoring both loads and resistance. This type of factoring differs from Part 1 of the allowable stress design (ASD) 1978 AISC Specification,[1] where only the resistance is divided by a factor of safety (to obtain allowable stress) and from Part 2, where only the loads are multiplied by a common load factor. The LRFD method was devised to offer the designer greater flexibility, more rationality and possible overall economy.

The format of using resistance factors and multiple load factors is not new, as several such design codes are in effect (the ACI-318 Strength Design for Reinforced Concrete[2] and the AASHTO Load Factor Design for Bridges).[3] Nor should the new LRFD method give designs radically different from the older methods, since it was tuned, or "calibrated," to typical representative designs of the earlier methods. The principal new ingredient is the use of a probabilistic mathematical model in the development of the load and resistance factors, which made it possible to give proper weight to the degree of accuracy with which the various loads and resistances can be determined. Also, it provides a rational methodology for transference of test results into design provisions. A more rational design procedure leading to more uniform reliability is the practical result.

A2. LIMITS OF APPLICABILITY

The AISC *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings* has evolved through numerous versions from the 1st Edition, published June 1, 1923. Each succeeding edition has been based upon past successful usage, advances in the state of knowledge and changes in engineering design practice. The data included has been developed to provide a uniform practice in the design of steel-framed buildings.

The intention of the new LRFD Specification, as with the allowable stress design AISC Specification, is to cover the many everyday design criteria required for routine design office usage. It is not intended to cover the infrequently encountered problems within the full range of structural design practice, because to provide such definitive provisions covering all possible cases and their complexities would diminish the Specification's usefulness for routine design office use.

The LRFD Specification is the result of the deliberations of a committee of structural engineers with wide experience and high professional standing, representing a wide geographical distribution throughout the United States. The membership of the committee is made up of approximately equal numbers representing design engineers in private practice, engineers involved in research and teaching and engineers employed by steel fabricating companies.

In order to avoid reference to proprietary steels which may have limited availability, only those steels which can be identified by ASTM specifications are listed as approved under the Specification. However, some steels covered by ASTM specification, but subject to more costly manufacturing and inspection techniques than deemed essential for structures covered by this Specification, are not listed, even though they may provide all of the necessary characteristics of less expensive steels which are listed. Approval of such steels is left to the owner's representative.

As used throughout the Specification, the term *structural steel* refers exclusively to those items enumerated in Sect. 2 of the AISC *Code of Standard Practice for Steel Buildings and Bridges*, and nothing contained herein is intended as a specification for design of items not specifically enumerated in that code, such as skylights, fire escapes, etc. For the design of cold-formed steel structural members, whose profiles contain rounded corners and slender flat elements, the provisions of the American Iron and Steel Institute *Specification for the Design of Cold-Formed Steel Structural Members* are recommended.

The reader is cautioned that independent professional judgment must be exercised when data or recommendations set forth in the Specification and Commentary are applied. The publication of the material contained herein is not intended as a representation or warranty on the part of the American Institute of Steel Construction, Inc.—or any other person named herein—that this information is suitable for general or particular use, or freedom from infringement of any patent or patents. Anyone making use of this information assumes all liability arising from such use. The design of structures is within the scope of expertise of a competent licensed professional for the application of principles to a particular structure.

2. Types of Construction

The provisions for these types of construction have been revised to provide for a truer recognition of the actual degree of connection restraint in the structural design. All connections possess an amount of restraint. Depending on the amount of restraint offered by the connection they are classified as either Type FR or PR. This new classification renames the Type I connection of the 1978 AISC Specification to Type FR and includes both Type II and Type III of that Specification under a new, more general classification of Type PR.

Just as in the allowable stress design (ASD) provisions, construction utilizing Type FR connections may be designed in LRFD using either elastic or plastic analysis provided the appropriate Specification provisions are satisfied.

For Type PR construction which uses the "simple framing" approach, the restraint of the connection is ignored, provided the given conditions are met. This is no change from the ASD provisions. Where there is evidence of the actual moment rotation capability of a given type of connection, the use of designs incorporating the connection restraint is permitted just as in ASD. The designer should, when incorporating connection restraint into the design, take into account the reduced connection stiffness on the stability of the structure and its effect on the magnitude of second order effects.

A3. MATERIAL

The grades of structural steel approved for use under the LRFD Specification, covered by ASTM standard specifications, extend to a yield stress of 100 ksi. Some of these ASTM standards specify a minimum yield point, while others specify a minimum yield strength. The term "yield stress" is used in the Specification as a generic term to denote either the yield point or the yield strength.

It is important to be aware of limitations of availability that may exist for some combinations of strength and size. Not all structural section sizes are included in the various material specifications. For example, the 60 ksi yield strength steel in the A572 specification includes plate only up to 1¼ in. in thickness. Another limitation on availability is that even when a product is included in the specifications, it may be only infrequently produced by the mills. Specifying these products may result in procurement delays or require ordering large quantities directly from the producing mills. Consequently, it is prudent to check availability before completing the details of a design.

The direction parallel to the direction of rolling is the direction of principal interest in the design of steel structures. Hence, yield stress as determined by standard tensile test is the principal mechanical property recognized in the selection of the steels approved for use under the Specification. It must be recognized that other mechanical and physical properties of rolled steel, such as anisotropy, ductility, notch toughness, formability, corrosion resistance, etc., may also be important to the satisfactory performance of a structure.

It is not possible to incorporate in the Commentary adequate information to impart full understanding of all factors which might merit consideration in the selection and specification of materials for unique or especially demanding applications. In such a situation the user of the Specification is advised to make use of reference material contained in the literature on the specific properties of concern and to specify supplementary material production or quality requirements as provided for in ASTM material specifications. One such case is the design of highly restrained welded connections.[90] Rolled steel is anisotropic, especially insofar as ductility is concerned; therefore, weld contraction strains in the region of highly restrained welded connections may exceed the capabilities of the material if special attention is not given to material selection, details, workmanship and inspection.

Another special situation is that of fracture control design for certain types of service conditions.[3] The relatively warm temperatures of steel in buildings, the essentially static strain rates, the stress intensity and the number of cycles of full design stress make the probability of fracture in building structures extremely remote. Good design details which incorporate joint geometry that avoids severe stress concentrations and good workmanship are generally the most effective means of providing fracture-resistant construction. However, for especially demanding service conditions such as low temperatures with impact loading, the specification of steels with superior notch toughness may be warranted.

The ASTM standard for A307 bolts covers two grades of fasteners. Either grade may be used under the LRFD Specification; however, it should be noted that Gr. B is intended for pipe flange bolting and Gr. A is the quality long in use for structural applications.

When specifying filler metal and/or flux by AWS designation, the applicable standard specifications should be carefully reviewed to assure a complete understanding of the designation reference. This is necessary because the AWS designation systems are not consistent. For example, in the case of electrodes for shielded metal arc welding (AWS A5.1), the first two or three digits indicate the nominal tensile strength classification, in ksi, of the weld metal and the final two digits indicate the type of coating; however, in the case of mild steel electrodes for submerged arc welding (AWS A5.17), the first one or two digits times 10 indicate the nominal tensile strength classification, while the final digit or digits times 10 indicate the testing temperature in degrees F, for weld metal impact tests. In the case of low-alloy steel covered arc welding electrodes (AWS A5.5), certain portions of the designation indicate a requirement for stress relief, while others indicate no stress relief requirement.

4. Anchor Bolts and Threaded Rods

New criteria on anchor bolts and threaded rods have been included in the LRFD Specification. Since there is a limit on the maximum available length of A325 and A490, the use of these bolts for anchor bolts with design lengths longer than the maximum available lengths has presented problems in the past. The inclusion of A687 material in this Specification allows the use of higher strength material for bolts longer than A325 and A490 bolts. The designer should be aware that pretensioning anchor bolts is not recommended due to relaxation and stress corrosion after pretensioning.

A new provision for threads has been included. The designer should specify the appropriate thread and SAE fit for threaded rods used as load-carrying members.

A4. LOADS AND LOAD COMBINATIONS

1. Loads, Load Factors and Load Combinations

The load factors and load combinations given in Sect. A4.1 were developed to be used with the recommended minimum loads given in ANSI A58.1 *Minimum Design Loads for Buildings and Other Structures*.[6] The load factors and load combinations are developed in Ref. 5. The target reliability indices underlying the load factors are β = 3.0 for combinations with gravity loads only (dead, snow and live loads), 2.5 for combinations with wind included and 1.75 for combinations with earthquake loads. See Commentary A5.3 for definition of β.

The load factors and load combinations recognize that when several loads act in combination with the dead load, e.g., dead plus live plus wind loads, only one of these takes on its maximum lifetime value, while the other load is at its "arbitrary point-in-time value," i.e., at a value which can be expected to be on the structure at any time. For example, under dead, live and wind loads the following combinations are appropriate:

$$\gamma_D D + \gamma_L L \qquad \text{(C-A4-1)}$$

$$\gamma_D D + \gamma_{L_a} L_a + \gamma_W W \qquad \text{(C-A4-2)}$$

$$\gamma_D D + \gamma_L L + \gamma_{W_a} W_a \qquad \text{(C-A4-3)}$$

where γ is the appropriate load factor as designated by the subscript symbol. Subscript a refers to an "arbitrary point-in-time" value.

The mean value of arbitrary point-in-time live load L_a is on the order of 0.24 to 0.4 times the mean maximum lifetime live load L for many occupancies, but its dispersion is far greater. The arbitrary point-in-time wind load W_a, acting in conjunction with the maximum lifetime live load, is the maximum daily wind. It turns out that $\gamma_{W_a} W_a$ is a negligible quantity so only two load combinations remain:

$$1.2\,D + 1.6\,L \qquad \text{(C-A4-4)}$$

$$1.2\,D + 0.5\,L + 1.3\,W \qquad \text{(C-A4-5)}$$

The load factor 0.5 assigned to L in the second equation reflects the statistical properties of L_a, but to avoid having to calculate yet another load, it is reduced so it can be combined with the maximum lifetime wind load.

The nominal loads D, L, W, E and S are the code loads or the loads given in ANSI A58.1-1982.[6] The dead and live loads in Ref. 6 are essentially identical to ANSI A58.1-1972, but there have been substantial changes in the manner of determining wind, snow and earthquake loads. These changes are discussed in the appendix to Ref. 6. Only one item will be mentioned. The treatment of the live load reduction has changed to the form

$$L = L_o\,(0.25 + 15/\sqrt{A_I}) \leq L_o \qquad \text{(C-A4-6)}$$

where L is the nominal live load, L_o is the basic live load assigned to the type of occupancy and A_I is an influence area which is equal to the tributary area for a two-way floor system, twice the tributary area for beams and four times the tributary area for columns.

2. Impact

A mass of the total moving load (wheel load) is used as the basis for impact loads on crane runway girders, because maximum impact load results when cranes travel while supporting lifted loads.

The increase in load, in recognition of random impacts, is not required to be applied to supporting columns because the impact load effects (increase in eccentricities or increases in out-of-straightness) will not develop or will be negligible during the short duration of impact. For additional information on crane girder design criteria see AISE Technical Report No. 13.

A5. DESIGN BASIS

1. Required Strength at Factored Loads

LRFD permits the use of both elastic and plastic structural analyses. LRFD provisions result in essentially the same methodology for, and end product of, plastic design as Part 2 of the 1978 AISC Specification,[1] except that the LRFD provisions tend to be more liberal reflecting added experience and the results of further research. The 10% redistribution permitted is carried forward from Part 1 of the 1978 AISC Specification.[1]

2. Limit States

A limit state is a condition which represents the limit of structural usefulness. Limit states may be dictated by functional requirements, such as maximum deflections or drift; they may be conceptual, such as plastic hinge or mechanism formation; or they may represent the actual collapse of the whole or part of the structure, such as fracture or instability. Design criteria insure that a limit state is violated only with an acceptably small probability by selecting the load and resistance factors and nominal load and resistance values which will never be exceeded under the design assumptions.

Two kinds of limit states apply for structures: limit states of strength which define safety against the extreme loads during the intended life of the structure, and limit states of serviceability which define the functional requirements. The LRFD Specification, like other structural codes, focuses on the limit states of strength because of the overriding considerations of public safety for the life, limb and property of human beings. This does not mean that limit states of serviceability are not important to the designer, who must equally insure functional performance and economy of design.

However, these latter considerations permit more exercise of judgment on the part of designers. Minimum considerations of public safety, on the other hand, are not matters of individual judgment and, therefore, specifications dwell more on the limit states of strength than on the limit states of serviceability.

Limit states of strength vary from member to member, and several limit states may apply to a given member. The following limit states of strength are the most common: onset of yielding, formation of a plastic hinge, formation of a plastic mechanism, overall frame or member instability, lateral-torsional buckling, local buckling, tensile fracture, development of fatigue cracks, deflection instability, alternating plasticity and excessive deformation.

The most common serviceability limit states include unacceptable elastic deflections and drift, unacceptable vibrations and permanent deformations.

3. Design for Strength

The general format of the LRFD Specification is given by the formula:

$$\Sigma \gamma_i Q_i \leq \phi R_n \tag{C-A5-1}$$

where

Σ = summation
i = type of load, i.e., dead load, live load, wind, etc.
Q_i = nominal load effect
γ_i = load factor corresponding to Q_i
$\Sigma \gamma_i Q_i$ = required resistance
R_n = nominal resistance
ϕ = resistance factor corresponding to R_n
ϕR_n = design strength

The left side of Formula C-A5-1 represents the required resistance which is computed by structural analysis based upon assumed loads and the right side of Formula C-A5-1 represents a limiting structural capacity provided by the selected members. In LRFD, the designer compares the effect of factored loads to the strength actually provided. The term design strength refers to the resistance or strength ϕR_n that must be provided by the selected member. The load factors γ and the resistance factor ϕ reflect the fact that loads, load effects (the computed forces and moments in the structural elements) and the resistances can be determined only to imperfect degrees of accuracy. The resistance factor ϕ is equal to or less than 1.0 because there is always a chance for the actual resistance to be less than the nominal value R_n computed by the formulas given in Chaps. D through K. Similarly, the load factors γ reflect the fact that the actual load effects may deviate from the nominal values of Q_i computed from the specified nominal loads. These factors account for unavoidable inaccuracies in the theory, variations in the material properties and dimensions and uncertainties in the determination of loads. They provide a margin of reliability to account for unexpected loads. They do not account for gross error or negligence.

The LRFD Specification is based on (1) a probabilistic model,[4,5] (2) a calibration of the new criteria to the 1978 AISC Specification,[1] and (3) the evaluation of the resulting criteria by judgment and past experience aided by comparative design office studies of representative structures.

The following is a brief probabilistic basis for LRFD.[4,5] The load effects Q and the resistance factor R are assumed to be statistically independent random variables. In

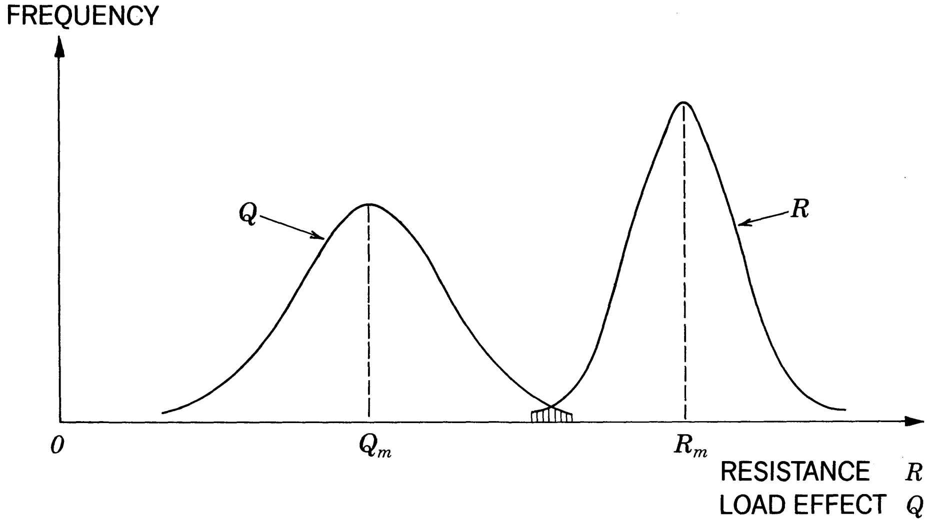


Fig. C-A5.1. Frequency distribution of load effect Q *and resistance* R

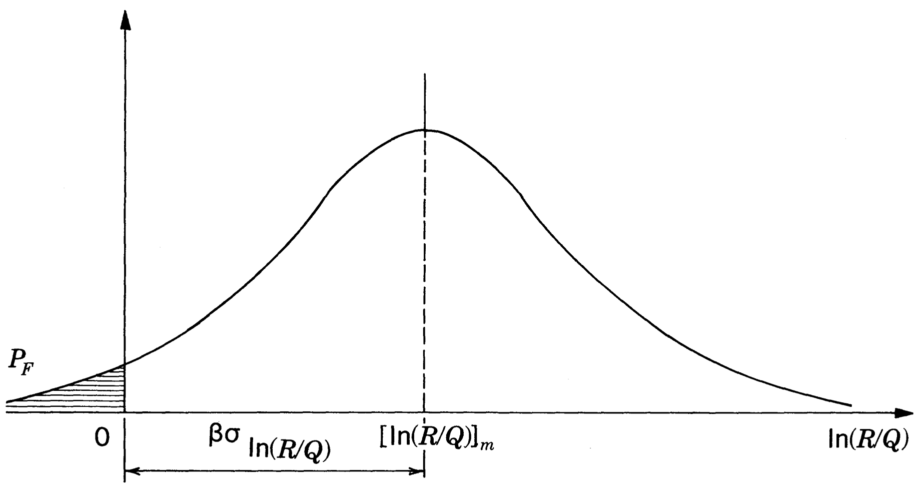


Fig. C-A5.2. Definition of reliability index

Fig. C-A5.1, frequency distributions for Q and R are portrayed as separate curves on a common plot for a hypothetical case. As long as the resistance R is greater than (to the right of) the effects of the loads Q, a margin of safety for the particular limit state exists. However, because Q and R are random variables, there is some small probability that R may be less than Q, ($R < Q$). This is portrayed by the small shaded area where the distribution curves cross.

An equivalent situation may be represented as in Fig. C-A5.2. If the expression $R < Q$ is divided by Q and the result expressed logarithmically, the result will be a single frequency distribution curve combining the uncertainties of both R and Q. The probability of attaining a limit state ($R < Q$) is equal to the probability that $\ln(R/Q) < 0$ and is represented by the shaded area in the diagram.

The shaded area may be reduced and thus reliability increased in either of two ways: (1) by moving the mean of $\ln(R/Q)$ to the right, or (2) by reducing the spread of the curve for a given position of the mean relative to the origin. A convenient way of combining these two approaches is by defining the position of the mean using the standard deviation of the curve, $\ln(R/Q)$, as the unit of measure. Thus, the distance from the origin to the mean is measured as the number of standard deviations of the function $\ln(R/Q)$. As shown in Fig. C-A5.2, this is stated as β times $\sigma_{\ln R/Q}$, the standard deviation of $\ln R/Q$. The factor β therefore is called the "reliability index."

If the actual distribution shape of $\ln(R/Q)$ were known, and if an acceptable value of the probability of reaching the limit state could be agreed upon, one could establish a completely probability-based set of design criteria. Unfortunately, this much information frequently is not known. The distribution shape of each of the many variables (material, loads, etc.) has an influence on the shape of the distribution of $\ln(R/Q)$. Often only the means and the standard deviations of the many variables involved in the makeup of the resistance and the load effect can be estimated. However, this information is enough to build an approximate design criterion which is independent of the knowledge of the distribution, by stipulating the following design condition:

$$\beta\sigma_{\ln(R/Q)} \cong \beta\sqrt{V_R^2 + V_Q^2} \leq \lambda\nu(R/Q)_m \cong \ln(R_m/Q_m) \qquad \text{(C-A5-2)}$$

In this formula, the standard deviation has been replaced by the approximation $\sqrt{V_R^2 + V_Q^2}$, where $V_R = \sigma_R/R_m$ and $V_Q = \sigma_Q/Q_m$ (σ_R and σ_Q are the standard deviations, R_m and Q_m are the mean values, V_R and V_Q are the coefficients of variation, respectively, of the resistance R and the load effect Q). For structural elements and the usual loadings R_m, Q_m, and the coefficients of variation, V_R and V_Q, can be estimated, so a calculation of

$$\beta = \frac{\ln(R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}} \qquad \text{(C-A5-3)}$$

will give a comparative value of the measure of reliability of a structure or component.

The description of the determination of β as given above is a simple way of defining the probabilistic method used in the development of LRFD. A more refined method, which can accommodate more complex design situations (such as the beam-column interaction equation) and include probabilistic distributions other than the lognormal distribution used to derive Formula C-A5-3, has been developed since the publication of Ref. 4 and is fully described in Ref. 5. This latter method has been used in the development of the recommended load factors (see Sect. A4). The two methods give essentially the same β values for most steel structural members and connections.

Statistical properties (mean values and coefficients of variations) are presented for the basic material properties and for steel beams, columns, composite beams, plate girders, beam-columns and connection elements in a series of eight articles in the September 1978 issue of the *Journal of the Structural Division of ASCE* (Vol. 104, ST9). The corresponding load statistics are given in Ref. 5. Based on these statistics, the values of β inherent in the 1978 AISC Specification[1] were evaluated under different load combinations (live/dead, wind/dead, etc.), and for various tributary areas for typical members (beams, columns, beam-columns, structural components, etc.). As can be expected, there was a considerable variation in the range of β values.

Examination of the many β values which were computed revealed certain trends. For example, typical beams have β values on the order of 3, and for typical connectors β was on the order of 4 to 5. The reliability index β for load combinations involving wind and earthquake tended to be lower.

One of the features of the probability-based method used in the development of LRFD is that the variations of β can be reduced by specifying several "target" β values, and selecting multiple load and resistance factors to meet these targets.

The following targets were selected: (1) under dead plus live and/or snow loading, $\beta = 3.0$ for members and $\beta = 4.5$ for connections; (2) under dead plus live plus wind loading, $\beta = 2.5$ for members; and (3) under dead plus live plus earthquake loading, $\beta = 1.75$ for members. The larger value of $\beta = 4.5$ for connectors reflects the fact that connections are expected to be stronger than the members they connect. The lower β-values for load combinations involving wind or earthquake loading are consistent with previous specifications.

Based on the target values of β given above, common load factors for various structural materials (steel, reinforced concrete, etc.) were developed in Ref. 5. Computer methods as well as charts are also given in Ref. 5 for the use of specification writers to determine the resistance factors ϕ. These factors can also be approximately determined by the following:

$$\phi = (R_m / R_n) \exp(-0.55 \, \beta \, V_R) \qquad \text{(C-A5-4)*}$$

where

R_m = mean resistance
R_n = nominal resistance according to the formulas in Chaps. D through K
V_R = coefficient of variation of the resistance

4. Design for Serviceability and Other Considerations

Nominally, serviceability should be checked at the unfactored loads. For combinations of gravity and wind or seismic loads some additional reduction factor may be warranted.

*Note that $\exp(x)$ is identical to the more familiar e^x.

CHAPTER B. DESIGN REQUIREMENTS

B3. EFFECTIVE NET AREA

Section B3 deals with the effect of shear lag. The inclusion of welded members acknowledges that shear lag is also a factor in determining the effective area of welded connections where the welds are so distributed as to directly connect some, but not all, of the elements of a tension member. However, since welds are applied to the unreduced cross-sectional area, the reduction coefficient U is applied to the gross area A_g. With this modification the values of U are the same as for similar shapes connected by bolts and rivets except that: (1) the provisions for members having only two fasteners per line in the direction of stress have no application to welded connections; and (2) tests[7] have shown that flat plates, or bars axially loaded in tension and connected only by longitudinal fillet welds, may fail prematurely by shear lag at their corners if the welds are separated by too great a distance. Therefore, the values of U are specified unless the member is designed on the basis of effective net area as discussed below.

As the length of a connection ℓ is increased the intensity of shear lag is diminished. The concept can be expressed empirically as:

$$U = 1 - \bar{x}/\ell$$

Munse and Chesson have shown,[43,92,93] using this expression to compute an effective net area, that with few exceptions, the estimated strength of some 1,000 test specimens correlated with observed test results within a scatterband of $\pm$ 10%. For any given profile and connected elements, $\bar{x}$ is a fixed geometric property. Length ℓ, however, is dependent upon the number of fasteners or length of weld required to develop the given tensile force, and this in turn is dependent upon the mechanical properties of the member and the capacity of the fasteners or weld used. The values of U, given as reduction coefficients in Sect. B3, are reasonably lower bounds for the profile types and connection means described, based upon the use of the above expression.

B4. STABILITY

The stability of structures must be considered from the standpoint of the structures as a whole, including not only the compression members, but also the beams, bracing system and connections. The stability of individual elements must also be provided. Considerable attention has been given to this subject in the technical literature, and various methods of analysis are available to assure stability. The SSRC *Guide to Design Criteria for Metal Compression Members*[11] devotes several chapters to the stability of different types of members considered as individual elements, and then considers the effects of individual elements on the stability of the structure as a whole.

B5. LOCAL BUCKLING

For the purposes of this Specification, steel sections are divided into compact sections, noncompact sections and sections with slender compression elements. Compact sections are capable of developing a *fully plastic* stress distribution and possess rotation capacity of approximately 3 before the onset of local buckling.[14] Noncompact sections can develop the yield stress in compression elements before local buckling occurs, but will not resist inelastic local buckling at the strain levels required for a fully plastic stress distribution. Slender compression elements buckle elastically before the yield stress is achieved.

The dividing line between compact and noncompact sections is the limiting width-thickness ratio λ_p. For a section to be compact, all of its compression elements must have width-thickness ratios smaller than the limiting λ_p.

A greater inelastic rotation capacity than provided by the limiting values λ_p given in Table B5.1 may be required for some structures in areas of high seismicity. It has been suggested that in order to develop a ductility of from 3 to 5 in a structural member, ductility factors for elements would have to lie in the range of 5 to 15. Thus, in this case, it is prudent to provide for an inelastic rotation of 7 to 9 times the elastic rotation.[8] In order to provide for this rotation capacity the limits λ_p for local flange and web buckling would be as shown in Table C-B5.1.[9]

Another limiting width-thickness ratio is λ_r, representing the distinction between noncompact sections and sections with slender compression elements. As long as the width-thickness ratio of a compression element does not exceed the limiting value λ_r, local elastic buckling will not govern its strength. However, for those cases where the width-thickness ratios exceed λ_r, elastic buckling strength must be considered. A design procedure for such slender compression elements, based on elastic buckling of plates, is given in Appendix B5.3. An exception is plate girders with slender webs. Such plate girders are capable of developing postbuckling strength in excess of the elastic buckling load. A design procedure for plate girders including tension field action is given in Appendix G.

The values of the limiting ratios λ_p and λ_r specified in Table B5.1 were obtained from Sects. 1.9 and 2.7 of Ref. 1 and from Table 2.3.3.3 of Ref. 9, except that: (1) $\lambda_p = 65/\sqrt{F_y}$, limited in Ref. 9 to indeterminate beams when moments are determined by elastic analysis and to determinate beams, was adopted for all conditions on the basis of Ref. 14; and (2) $\lambda_p = 1{,}300/F_y$ for circular hollow sections was obtained from Ref. 10.

The high shape factor for circular hollow sections makes it impractical to use the same slenderness limits to define the regions of behavior for different types of loading. In Table B5.1, the values of λ_p for a compact shape that can achieve the plastic moment, and λ_r for bending, are based on an analysis of test data from several projects involving the bending of pipes in a region of constant moment.[79,81] The same analysis produced the equation for the inelastic moment capacity in Table A-F1.1 in Appendix F1.7. However, a more restrictive value of λ_p is required to prevent inelastic local buckling from limiting the plastic hinge rotation capacity needed to develop a mechanism in a circular hollow beam section.[10]

The values of λ_r for axial compression and for bending are both based on test data. The former value has been used in building specifications since 1968.[80] Appendices B5.3 and F1.7 also limit the diameter-to-thickness ratio for any circular section to $13{,}000/F_y$. Beyond this, the local buckling strength decreases rapidly, making it impractical to use these sections in building construction.

Following the SSRC recommendations[11] and the approach used for other shapes

TABLE C-B5.1
Limiting Width-thickness Ratios for Compression Elements

Description of Element	Width-thickness Ratio	Limiting Width-thickness Ratios λ_p Non-seismic	Limiting Width-thickness Ratios λ_p Seismic
Flanges of I-shaped sections (including hybrid sections) and channels in flexure[a]	b/t	$65/\sqrt{F_y}$	$52/\sqrt{F_y}$
Webs in combined flexural and axial compression	h_c/t_w	For $P_n/\phi_b P_y \leqq 0.125$	
		$\frac{640}{\sqrt{F_y}}\left(1-\frac{2.75P}{\phi_b P_y}\right)$	$\frac{520}{\sqrt{F_y}}\left(1-\frac{1.54P}{\phi_b P_y}\right)$
		For $P_n/\phi_b P_y > 0.125$	
		$\frac{191}{\sqrt{F_u}}\left(2.33-\frac{P}{\phi_b P_y}\right) \geq \frac{253}{\sqrt{F_y}}$	$\frac{152}{\sqrt{F_y}}\left(2.89-\frac{P}{\phi_b P_y}\right)$

[a]For hybrid beams use F_{yf} in place of F_y.

with slender compression elements, a Q-factor is used for circular sections to account for an interaction between local and column buckling. The Q-factor is the ratio between the local buckling stress and the yield stress. The local buckling stress for the circular section is taken from the inelastic AISI criteria[80] and is based on tests conducted on fabricated and manufactured cylinders. More recent tests on fabricated cylinders[81] confirm that this equation is conservative.

The definitions of the width and thickness of compression elements were taken from Sect. 1.9 of the 1978 AISC Specification[1] with minor modifications extending their applicability to sections formed by bending and to unsymmetrical and hybrid sections.

B7. LIMITING SLENDERNESS RATIOS

Chapters D and E provide reliable criteria for resistance of axially loaded members based on theory and confirmed by test for all significant parameters including slenderness. The advisory upper limits on slenderness contained in B7 are based on professional judgement and practical considerations of economics, ease of handling, and care required to minimize inadvertent damage during fabrication, transport and erection. Out-of-straightness within reasonable tolerances does not affect the strength of tension members, and the effect of out-of-straightness within specified tolerances on the strength of compression members is accounted for in formulas for resistance. Applied tension tends to reduce whereas compression tends to amplify out-of-straightness. Therefore, more liberal criteria are suggested for tension members, including those subject to small compressive force resulting from transient loads such as earthquake and wind. For members with slenderness ratios greater than 200, these compressive forces correspond to stresses less than 2.6 ksi.

CHAPTER C.
FRAMES AND OTHER STRUCTURES

C1. SECOND ORDER EFFECTS

While resistance to wind and seismic loading can be provided in certain buildings by means of concrete or masonry shear walls, which also provide for overall frame stability at factored gravity loading, other building frames must provide this resistance by acting alone. This resistance can be achieved in several ways, e.g., by a system of bracing, by a moment-resisting frame or by any combination of lateral force-resisting elements.

For frames under combined gravity and lateral loads, drift (horizontal deflection caused by applied loads) occurs at the start of loading. At a given value of the applied loads, the frame has a definite amount of drift Δ. In unbraced frames, significant additional secondary bending moments, known as the $P\Delta$ moments, may be developed in the columns and beams of the lateral load-resisting systems in each story. P is the total gravity load above the story and Δ is the story drift. As the applied load increases, the $P\Delta$ moments also increase. Therefore, the $P\Delta$ effect often should be accounted for in frame design. Similarly, in braced frames, increases in axial forces occur in the members of the bracing systems; however, such effects are usually less significant. The designer should consider these effects for all types of frames and determine if they are significant for his particular case. When considering frame instability effects ($P\Delta$) in the design of these frames, the equations in Chap. H offer the designer a direct method for including such considerations in calculations of required strength moments. The designer also has the option to compute second order effects directly. Since $P\Delta$ effects can cause frame drifts to be larger than those calculated by ignoring them, $P\Delta$ effects should also be included in the drift analysis when they are significant.

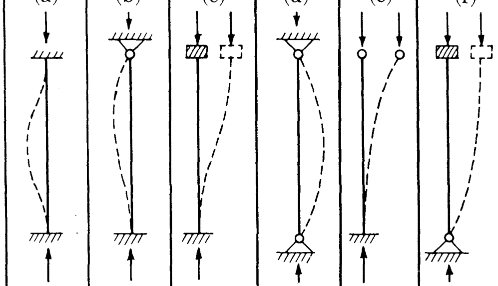
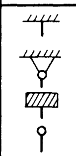
In plastically designed unbraced frames the limit of 0.75 P_y on column axial loads has also been retained to help insure proper stability.

C2. FRAME STABILITY

The stability of structures must be considered from the standpoint of the structure as a whole, including not only the compression members, but also the beams, bracing system and connections. The stability of individual elements must also be provided. Considerable attention has been given in the technical literature to this subject, and various methods of analysis are available to assure stability. The SSRC *Guide to Design Criteria for Metal Compression Members*[11] devotes several chapters to the stability of different types of members considered as individual elements, and then considers the effects of individual elements on the stability of the structure as a whole.

The effective length concept is one method for estimating the interaction effects of the total frame on a compression element being considered. This concept uses K-factors to equate the strength of a framed compression element of length L to an equivalent pin-ended member of length KL subject to axial load only. Other rational methods are available for evaluating the stability of frames subject to gravity and side loading and individual compression members subject to axial load and moments. However, the effective length concept is the only tool currently available for handling several cases which occur in practically all structures, and it is an essential part of many analysis procedures. Although the concept is completely valid for ideal structures, its practical implementation involves several assumptions of idealized conditions which will be mentioned later.

Table C-C2.1

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code		Rotation fixed and translation fixed Rotation free and translation fixed Rotation fixed and translation free Rotation free and translation free				

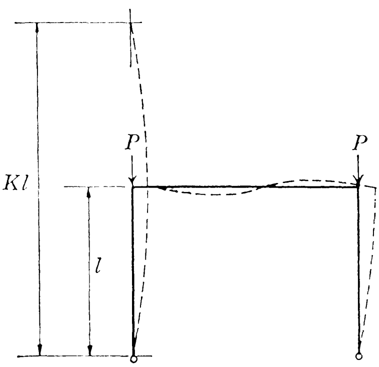


Figure C-C2.1

Two conditions, opposite in their effect upon column strength under axial loading, must be considered. If enough axial load is applied to the columns in an unbraced frame dependent entirely on its own bending stiffness for resistance to lateral deflection of the tops of the columns with respect to their bases (see Fig. C-C2.1), the effective length of these columns will exceed the actual length. On the other hand, if the same frame were braced to resist such lateral movement, the effective length would be less than the actual length, due to the restraint (resistance to joint rotation) provided by the bracing or other lateral support. The ratio K, effective column length to actual unbraced length, may be greater or less than 1.0.

The theoretical K-values for six idealized conditions in which joint rotation and translation are either fully realized or nonexistent are tabulated in Table C-C2.1. Also shown are suggested design values recommended by the Structural Stability Research Council (formerly the Column Research Council) for use when these conditions are approximated in actual design. In general, these suggested values are slightly higher than their theoretical equivalents, since joint fixity is seldom fully realized.

If the column base in Case f of Table C-C2.1 were truly pinned, K would actually exceed 2.0 for a frame such as that pictured in Fig. C-C2.1, because the flexibility of the horizontal member would prevent realization of full fixity at the top of the column. On the other hand, it has been shown[51] that the restraining influence of foundations, even where these footings are designed only for vertical load, can be very substantial in the case of flat-ended column base details with ordinary anchorage. For this condition, a design K-value of 1.5 would generally be conservative in Case f.

While in some cases the existence of masonry walls provides enough lateral support for their building frames to control lateral deflection, the increasing use of light curtain wall construction and wide column spacing for high-rise structures not provided with a positive system of diagonal bracing can create a situation where only the bending stiffness of the frame itself provides this support.

In this case the effective length factor K for an unbraced length of column L is dependent upon the amount of bending stiffness provided by the other inplane members entering the joint at each end of the unbraced segment. If the combined stiffness provided by the beams is sufficiently small, relative to that of the unbraced column segments, KL could exceed two or more story heights.*

Several rational methods are available to estimate the effective length of the columns in an unbraced frame with sufficient accuracy. These range from simple interpolation between the idealized cases shown in Table C-C2.1 to very complex analytical procedures. Once a trial selection of framing members has been made, the use of the alignment chart in Fig. C-C2.2 affords a fairly rapid method for determining adequate K-values.

However, it should be noted that this alignment chart is based upon assumptions of idealized conditions which seldom exist in real structures.[11] These assumptions are as follows:

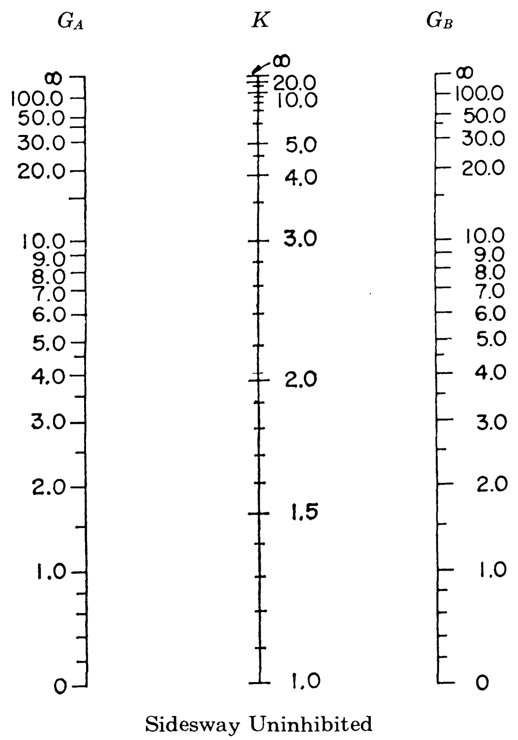
1. Behavior is purely elastic.
2. All members have constant cross section.
3. All joints are rigid.
4. For braced frames, rotations at opposite ends of beams are equal in magnitude, producing single curvature bending.
5. For unbraced frames, rotations at opposite ends of the restraining beams are equal in magnitude, producing reverse curvature bending.
6. The stiffness parameters $L\sqrt{P/EI}$ of all columns are equal.
7. Joint restraint is distributed to the column above and below the joint in proportion to I/L of the two columns.
8. All columns buckle simultaneously.

Where the actual conditions differ from these assumptions, unrealistic designs may result. There are design procedures available[52,53] which may be used in the calculation of G for use in Fig. C-C2.2 to give results more truly representative of conditions in real structures.

It is worth mentioning here that frames which use PR connections violate the condition that all joints are rigid, and special attention should be paid to calculation of a proper G for these instances.

In frames which depend upon their own bending stiffness for stability, the amplified moments are accounted for in the design of columns by means of the

*Ref. 50, pp. 260–265.



The subscripts A and B refer to the joints at the two ends of the column section being considered. G is defined as

$$G = \frac{\sum \frac{I_c}{L_c}}{\sum \frac{I_g}{L_g}}$$

in which Σ indicates a summation of all members rigidly connected to that joint and lying in the plane in which buckling of the column is being considered. I_c is the moment of inertia and L_c the unsupported length of a column section, and I_g is the moment of inertia and L_g the unsupported length of a girder or other restraining member. I_c and I_g are taken about axes perpendicular to the plane of buckling being considered.

For column ends supported by but not rigidly connected to a footing or foundation, G is theoretically infinity, but, unless actually designed as a true friction free pin, may be taken as "10" for practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as 1.0. Smaller values may be used if justified by analysis.

Alignment Chart for Effective Length of Columns in Continuous Frames

Figure C-C2.2

interaction formulas of Sect. H1. However, moments are also induced in the beams which restrain the columns; thus, consideration must be given to the amplification of those portions of the beam moments that are increased when the frame drifts. The effect may be particularly important in frames in which the contribution to individual beam moments from story shears becomes small as a result of distribution to many bays, but in which the $P\Delta$ moment in individual columns and beams is not diminished and becomes dominant.

If roof decks or floor slabs, anchored to shear walls or vertical plane bracing systems, are counted upon to provide lateral support for individual columns in a building frame, due consideration must be given to their stiffness when functioning as a horizontal diaphragm.[59]

While translation of the joints in the plane of a truss is inhibited and, due to end restraint, the effective length of compression members might therefore be assumed to be less than the distance between panel points, it is usual practice to take K as equal to 1.0,[11] since, if all members of the truss reached their ultimate load capacity simultaneously, the restraints at the ends of the compression members would disappear or, at least, be greatly reduced.

CHAPTER D.
TENSION MEMBERS

D1. DESIGN TENSILE STRENGTH

Due to strain hardening, a ductile steel bar loaded in axial tension can resist, without fracture, a force greater than the product of its gross area and its coupon yield stress. However, excessive elongation of a tension member due to uncontrolled yielding of its gross area not only marks the limit of its usefulness, but can precipitate failure of the structural system of which it is a part. On the other hand, depending upon the scale of reduction of gross area and the mechanical properties of the steel, the member can fail by fracture of the net area at a load smaller than required to yield the gross area. Hence, general yielding of the gross area and fracture of the net area both constitute failure limit states. The relative values of ϕ_t given for yielding and fracture reflect the same basic difference in factor of safety as between design of members and design of connections in the 1978 AISC Specification, Part 1.[1]

The part of the member occupied by the net area at fastener holes has a negligible length relative to the total length of the member. As a result, the strain hardening condition is quickly reached and yielding of the net area at fastener holes does not constitute a limit state of practical significance.

D2. BUILT-UP MEMBERS

The slenderness ratio L/r of tension members other than rods, tubes or straps should preferably not exceed the limiting values of 240. This slenderness limit recommended for tension members is not essential to the structural integrity of such members; they merely assure a degree of stiffness such that undesirable lateral movement ("slapping" or vibration) will be unlikely.

See Commentary Sect. E4.

D3. EYEBARS AND PIN-CONNECTED MEMBERS

Forged eyebars have generally been replaced by pin-connected plates or eyebars thermally cut from plates. Provisions for the proportioning of eyebars contained in the LRFD Specification are based upon standards evolved from long experience with forged eyebars. Through extensive destructive testing, eyebars have been found to provide balanced designs when they are thermally cut instead of forged. The somewhat more conservative rules for pin-connected members of nonuniform cross section and those not having enlarged "circular" heads are likewise based on the results of experimental research.[4] For greater clarity, the provisions relating to pin plate design have been extensively reworded and updated.

Somewhat stockier proportions are provided for eyebars and pin-connected members fabricated from steel having a yield stress greater than 70 ksi, in order to eliminate any possibility of their "dishing" under the higher working stress for which they may be designed.

CHAPTER E. COLUMNS AND OTHER COMPRESSION MEMBERS

E1. EFFECTIVE LENGTH AND SLENDERNESS LIMITATIONS

1. Effective Length

The Commentary on Sect. C2 regarding frame stability and effective length factors applies here. Further analytic methods, formulas, charts and references for the determination of effective length are provided in Chap. 15 of the SSRC *Guide to Design Criteria for Metal Compression Members*.[11]

2. Plastic Analysis

The limitation on λ_c is essentially the same as that for L/r in Sect. 2.4 of the 1978 AISC Specification.[1]

E2. DESIGN COMPRESSIVE STRENGTH*

Formulas E2-2 and E2-3 are based on a reasonable conversion of research data into design equations. Conversion of the allowable stress design (ASD) equations based on the Structural Stability Research Council (SSRC) (formerly the Column Research Council)[11] was found to be cumbersome for two reasons. The first was the nature of the ASD variable safety factor. Secondly, the difference in philosophical origins of the two design procedures requires an assumption of a live load-to-dead load ratio (L/D).

Since all L/D ratios could not be considered, a value of approximately 1.1 at λ equal to 1.0 was used to calibrate the exponential equation for columns with the lower range of λ against the appropriate ASD provision. The coefficient with the Euler equation was obtained by equating the two equations at the common λ of 1.5.

Formulas E2-2 and E2-3 are essentially the same as column-strength curve 2P of the 4th edition SSRC *Guide* for an out-of-straightness criterion of $L/1500$.[81,99]

It should be noted that this set of column equations has a range of reliability (β)-values. At low and high column slenderness, β-values exceeding 3.0 and 3.3 respectively are obtained compared to β of 2.60 at L/D of 1.1. This is considered satisfactory, since the limits of out-of-straightness combined with residual stress have not been clearly established. Furthermore, there has been no history of unacceptable behavior of columns designed using the ASD procedure. This includes cases with L/D ratios greater than 1.1.

Formulas E2-2 and E2-3 can be restated in terms of the more familiar slenderness $K\ell/r$. First, Formula E2-2 is expressed in exponential form,

$$F_{cr} = [\exp(-0.419\lambda_c^2)]F_y \qquad \text{(C-E2-1)}$$

*For tapered members, also see Commentary Sect. D2.

Note that exp(x) is identical to e^x. Substitution of λ_c according to Formula E2-4 gives,

for $\frac{K\ell}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$

$$F_{cr} = \left\{\exp\left[-0.0424\frac{F_y}{E}\left(\frac{K\ell}{r}\right)^2\right]\right\}F_y \quad \text{(C-E2-2)}$$

for $\frac{K\ell}{r} > 4.71\sqrt{\frac{E}{F_y}}$

$$F_{cr} = \frac{0.877\pi^2 E}{\left(\frac{K\ell}{r}\right)^2} \quad \text{(C-E2-3)}$$

The design strength of columns and other compression members of 36 and 50 ksi structural steels is included in Tables 3-36 and 3-50 of the LRFD Specification for the convenience of the designer.

E3. FLEXURAL-TORSIONAL BUCKLING

Torsional buckling of symmetric shapes and flexural-torsional buckling of unsymmetric shapes are failure modes usually not considered in the design of hot-rolled columns. They generally do not govern or the critical load differs very little from the weak axis planar buckling load. Such buckling loads may, however, control the capacity of symmetric columns made from relatively thin plate elements and of unsymmetric columns. Design equations for determining the strength of such columns are given in Appendix E3.

E4. BUILT-UP MEMBERS

Requirements for detailing and design of built-up members, which cannot be stated in terms of calculated stress, are based upon judgment and experience.

The longitudinal spacing of connectors connecting components of built-up compression members must be such that the slenderness ratio L/r of individual shapes does not exceed the slenderness ratio of the entire member. Additional requirements are imposed for built-up members consisting of angles. However, these minimum requirements do not necessarily ensure that the effective slenderness ratio of the built-up member is equal to that for the built-up member acting as a single unit. Appendix E4 gives formulas for modified slenderness ratios that are based on research and take into account the effect of shear deformation in the connectors.[94] The connectors must be designed to resist the shear forces which develop in the buckled member. The shear stresses are highest where the slope of the buckled shape is maximum.[50]

Maximum fastener spacing less than that required for strength may be needed to ensure a close fit over the entire faying surface of components in continuous contact. Specific requirements are given for weathering steel members exposed to atmospheric corrosion.[91]

The provisions governing the proportioning of perforated cover plates are based upon extensive experimental research.[95]

CHAPTER F.
BEAMS AND OTHER FLEXURAL MEMBERS

F1. DESIGN FOR FLEXURE

1. Unbraced Length for Plastic Analysis

In the 1978 AISC Specification,[1] Part 2, the unbraced length of a beam which permits the attainment of plastic moments and sufficient rotation capacity to redistribute moments is given by two formulas which depend on the moment ratio at the ends of the unbraced length. One length was permitted for $M_1/M_p < -0.5$ (almost uniform moment), and a substantially larger length for $M_1/M_p > -0.5$. These two formulas are replaced by (F1-1) which provides for a continuous function between unbraced length and end moment ratio so there is no abrupt change for a slight change in moment ratio near -0.5. At $M_1/M_p = -1.0$ (uniform moment) the maximum unbraced length is almost the same as the 1978 Specification. There is a substantial increase in unbraced length for positive moment ratios (reverse curvature) because the yielding is confined to zones close to the brace points.[14] Formulas F1-1 and F1-2 assume that the moment diagram within the unbraced length next to plastic hinge locations is reasonably linear. For nonlinear diagrams between braces, judgement should be used in choosing a representative ratio.

Formulas F1-1 and F1-2 were developed to provide rotations capacities of at least 3.0, which are sufficient for most applications.[14] When inelastic rotations of 7 to 9 are deemed appropriate in areas of high seismicity, as discussed in Commentary Sect. B5, Formula F1-1 would become:[9]

$$L_{pd} = \frac{146\ r_y}{\sqrt{F_y}} \qquad \text{(C-F1-1)}$$

3. Compact Section Members with $L_b \leq L_r$

The basic relationship between nominal moment M_n and unbraced length L_b is shown in Fig. C-F1.1 for a compact section with $C_b = 1.0$. There are four principal zones defined on the basic curve by L_{pd}, L_p and L_r. Formula F1-4 defines the maximum unbraced length L_p to reach M_p with uniform moment. Elastic lateral-torsional buckling will occur when the unbraced length is greater than L_r given by Formula F1-6. Formula F1-3 defines the inelastic lateral-torsional buckling as a straight line between the defined limits L_p and L_r. Buckling strength in the elastic region $L_b > L_r$ is given by Formula F1-13 for I-shaped members.

For other moment diagrams, the lateral buckling strength is obtained by multiplying the basic strength by C_b as shown in Fig. C-F1.1. The maximum M_n, however, is limited to M_p. Note that L_p given by Formula F1-4 is merely a definition which has physical meaning when $C_b = 1.0$. For C_b greater than 1.0, larger unbraced lengths are permitted to reach M_p as shown by the curve for $C_b = 2.3$. For design, this length could be calculated by setting Formula F1-3 equal to M_p and solving this equation for L_b using the desired C_b value.

For nonlinear moment diagrams between brace points, especially where the largest moment is not at the ends, C_b larger than 1.0 may be warranted. See Ref. 11 for a summary of such cases.

The elastic strength of hybrid beams is identical to homogeneous beams. The strength advantage of hybrid sections becomes evident only in the inelastic and plastic slenderness ranges.

4. Compact Section Members with $L_b > L_r$

The equation given in the Specification assumes that the loading is applied along the beam centroidal axis. If the load is placed on the top flange and is not braced, there is a tipping effect that reduces the critical moment; conversely, if the load is suspended from the bottom flange and is not braced, there is a stabilizing effect which increases the critical moment.[81] For unbraced top flange loading, the reduced critical moment may be conservatively approximated by setting the warping buckling factor X_2 to zero.

5. Tees and Double-angle Beams

The lateral-torsional buckling strength of singly symmetric tee beams is given by a fairly complex formula.[11] This buckling formula has been simplified considerably in Ref. 15 and Formula F1-15 is based on this work.

7. Nominal Flexural Strength of Other Sections

Formulae for the nominal strength of various types of sections are given in Appendix F1.

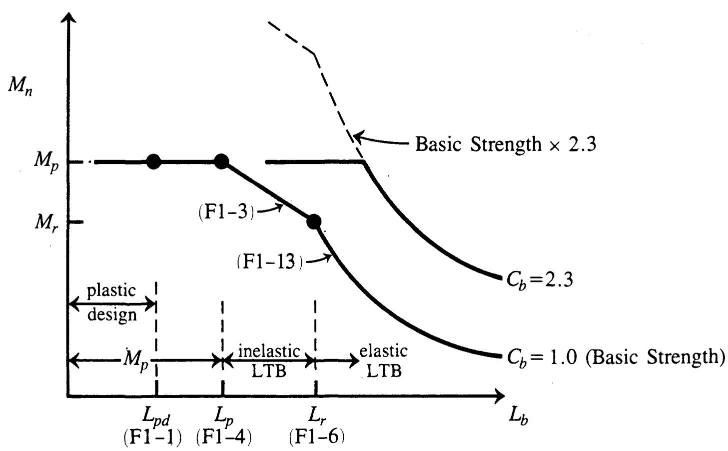


Fig. C-F1.1. Nominal moment as a function of unbraced length and moment gradient.

F2. DESIGN FOR SHEAR

For webs with $h/t_w \leq 187\sqrt{k/F_{yw}}$, the nominal shear strength V_n is based on shear yielding of the web, Formula F2-1. This h/t_w limit was determined by setting the critical stress causing shear buckling F_{cr} equal to the yield point of the web F_{yw} in Formula 35 of Ref. 16. When $h/t_w > 187\sqrt{k/F_{yw}}$, the web shear strength is based on buckling. Basler[17] suggested taking the proportional limit as 80% of the yield stress of the web. This corresponds to $h/t_w = (187/0.8)(\sqrt{k/F_{yw}})$. Thus, when $h/t_w > 234(\sqrt{k/F_{yw}})$, the web strength is determined from the elastic buckling stress given by Formula 6 of Ref. 16:

$$F_{cr} = \frac{\pi^2 Ek}{12(1 - \nu^2)(h/t_w)^2} \tag{C-F2-1}$$

The nominal shear strength, given by Formula F2-3, was obtained by multiplying F_{cr} by the web area and using $E = 29{,}000$ ksi and $\nu = 0.3$. A straight line transition, Formula F2-2, is used between the limits $187(\sqrt{k/F_{yw}})$ and $234(\sqrt{k/F_{yw}})$.

The shear strength of flexural members follows the approach used in the 1978 AISC Specification,[1] except for two simplifications. First, the expression for the plate buckling coefficient k has been simplified; it corresponds to that given in the AASHTO *Standard Specification for Highway Bridges*.[3] The earlier expression for k was a curve fit to the exact expression; the new recommendation is just as accurate. Second, the alternate method (tension field action) for web shear strength which utilized post buckling strength has been placed in Appendix G because the AISC Specification Advisory Committee recommends that only one method appear in the main body of the Specification with alternate methods given in the Appendix. When designing plate girders, thicker unstiffened webs will frequently be less costly than lighter stiffened web designs because of the additional fabrication. If a stiffened girder design has economic advantages, the tension field method in Appendix G will require fewer stiffeners.

The formulas in this section were established assuming monotonically increasing loads. If a flexural member is subjected to load reversals causing cyclic yielding over large portions of a web, such as may occur during a major earthquake, special design considerations may apply.[18]

CHAPTER H. MEMBERS UNDER TORSION AND COMBINED FORCES

H1. SYMMETRIC MEMBERS SUBJECT TO BENDING AND AXIAL FORCE

The new Formulas H1-1a and H1-1b are simplifications and clarifications of Formulas 1.6-1a and 1.6-1b used in the AISC Specification[1] since 1961. Previously, both equations had to be checked, Formula 1.6-1a being the check for stability and Formula 1.6-1b being the check for strength. In the new formulation the applicable equation is governed by the value of the first term, $P_u/\phi P_n$. For bending about one axis only, the equations have the form shown in Fig. C-H1.1.

The first term $P_u/\phi P_n$ has the same significance as the axial load term f_a/F_a in Formula 1.6-1a of the 1978 AISC Specification. This means that P_n must be based on the largest effective slenderness ratio $K\ell/r$. In the development of Formulas H1-1a and H1-1b, a number of alternative formulations were compared to the exact inelastic solutions of 82 sidesway cases reported in Ref. 61. In particular, the possibility of using $K\ell/r$ as the actual column length ($K = 1$) in determining P_n, combined with an elastic second order moment M_u, was studied. In those cases where the true P_n based on $K\ell/r$, with $K = 1.0$, was in the inelastic range, the errors proved to be unacceptably large without the additional check that $P_u \leq \phi_c P_n$, P_n being based on effective length. Although deviations from exact solutions were reduced they still remained high.

In summary, it is not possible to formulate a safe general interaction equation for compression without considering effective length directly (or indirectly by a second equation). Therefore, the requirement that the nominal compressive strength P_n be

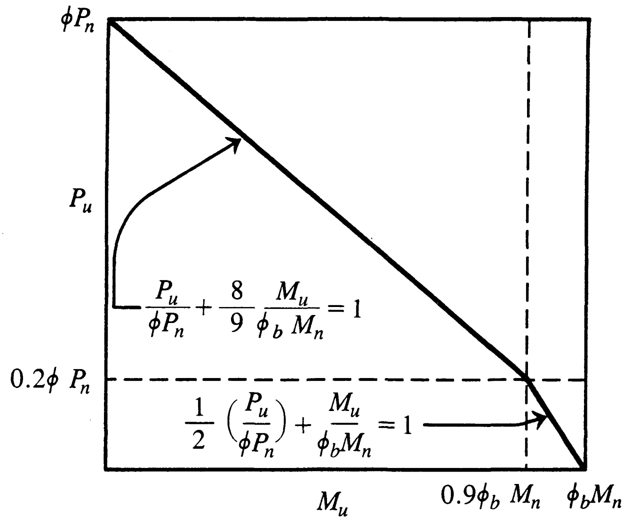


Figure C-H1.1

based on the effective length KL in the general interaction equation is continued in the LRFD Specification as it has been in the AISC Specification since 1961. It is not intended these provisions be applicable to limit nonlinear secondary flexure that might be encountered in large amplitude earthquake stability design.[28]

An important benefit which follows from including the correct value of P_n in Formulas H1-1a and H1-1b is that the factor B_2 may be based on first order sidesway deflection. Alternatively, the product $B_2 M_{lt}$ may be directly determined by a $P\Delta$ analysis as outlined in Ref. 62.

The newly defined term M_u is the maximum moment in a member, including the magnifying effect of compressive axial loads. In the general case, a member may have first order moments not associated with sidesway which are multiplied by B_1, and first order moments produced by forces causing sidesway which are multiplied by B_2. Unlike the 1978 AISC Specification,[1] B_1 is never less than one in the LRFD Specification.

In frames subject to sidesway, earlier Specifications required that all moments be multiplied by the factor $0.85/(1 - f_a/F'_e)$. This often unnecessarily increased gravity moments not associated with sidesway. The new factor B_2 applies only to moments caused by forces producing sidesway and is calculated for an entire story. In building frames designed to limit Δ_{oh}/L to a predetermined value, the factor B_2 may be found in advance of designing individual members. Drift limits may also be set for design of various categories of buildings so that the effect of secondary bending can be insignificant.[27,28] Alternative formulations of $B_2 M_{lt}$ may be made at the designer's option.[24,25,26,27] It is conservative to use the B_2 factor with the total of the sway and no sway moments, as in the 1978 AISC Specification.

The two kinds of first order moment M_{nt} and M_{lt} may both occur in sidesway frames from gravity loads. M_{nt} may be defined as a moment developed in a member with frame sidesway prevented. If a significant restraining force is necessary to prevent sidesway of an unsymmetrical structure (or an unsymmetrically loaded symmetrical structure), the moments induced by releasing the restraining force will be M_{lt} moments, to be multiplied by B_2. In most reasonably symmetric frames, this effect will be trivial. If such a moment $B_2 M_{lt}$ is added algebraically to the $B_1 M_{nt}$ moment developed with sidesway prevented, a fairly accurate value of M_{lt} will result. Of course end moments produced in sidesway frames by lateral loads from wind or earthquake will always be M_{lt} moments to be multiplied by B_2.

For compression members in braced frames, Formulas H1-1a and H1-1b (which is a continuous function) are similar in application to the previous Formula 1.61-1a. B_1 is determined from C_m values which are unchanged from the 1978 AISC Specification except that the 0.4 limit has been removed in Formula H1-4. A significant change, however, is that B_1 is never less than 1. When $C_m = 1$ for a compression member loaded between its supports the factors of 8/9 and ½ make the new equations more liberal than Formula 1.6-1a. For $C_m \leq 1$ (for members with unequal end moments) the new equations will be slightly more conservative than the 1978 AISC Specification for a very slender member with low C_m. For the entire range of l/r and C_m, the equations compare very closely to exact inelastic solutions of braced members.

For braced frames, $\phi_c P_n$ will always be based on $K \leq 1$, and B_1 will always be determined from P_e based on $K \leq 1$. The actual length of members is used in the structural analysis. In braced and unbraced frames, P_n is governed by the maximum slenderness ratio regardless of the plane of bending. P_e on the other hand, is always governed by the slenderness ratio in the plane of bending. Thus, when flexure is about the strong axis only, two different values of slenderness ratio may be involved in solving a given problem.

When bending occurs about both the x- and y-axes, the required flexural strength calculated about each axis is adjusted by the value of C_m and P_e corresponding to the distribution of moment and the slenderness ratio in its plane of bending, and is then taken as a fraction of the design bending strength, $\phi_b M_n$, about that axis, with due regard to the unbraced length of compression flange where this is a factor.

Formulas H1-3 and H1-4 approximate the maximum second order moments in compression members with no relative joint translation and no transverse loads between the ends of the member. This approximation is compared to an exact solution[97] in Fig. C-H1.2. For single curvature, Formula H1-4 is slightly unconservative, for a zero end moment it is almost exact; and for double curvature it is conservative. The 1978 AISC Specification limits $C_m \geq 0.4$ which corresponds to a M_1/M_2 ratio of 0.5. However, Fig. C-H1.2 shows that if, for example, $M_1/M_2 = 0.8$, the $C_m = 0.28$ is already very conservative, so the limit has been removed. The limit was originally adopted from Ref. 29, which was intended to apply to lateral-torsional buckling not second order in plane bending strength. The AISC Specifications, both in the 1978 and LRFD, use a modification factor C_b as given in Formula F1-3 for lateral-torsional buckling. C_b, which is limited to 2.3, is approximately the inverse of C_m as presented in Ref. 29 with a 0.4 limit. In Ref. 94 it was pointed out that Formula H1-4 could be used for in plane second order moments if the 0.4 limit was eliminated. Unfortunately, Ref. 29 was misinterpreted and a lateral-torsional buckling solution was used for an in plane second order analysis. This oversight has now been corrected.

For beam columns with transverse loadings, the second order moment can be approximated by using the following equation

$$C_m = 1 + \psi P_u / P_e$$

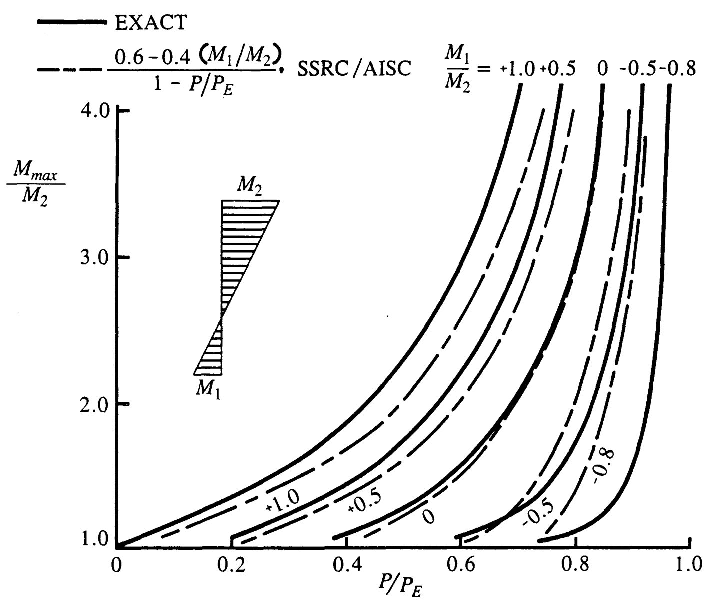
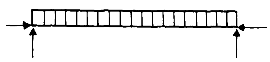
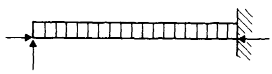
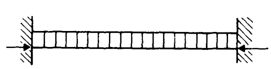
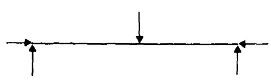
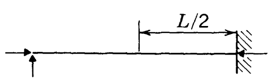
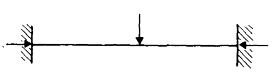


Figure C-H1.2

TABLE C-H1.1
Amplification Factors ψ and C_m

Case	ψ	C_m
	0	1.0
	−0.4	$1 - 0.4\frac{f_a}{F'_e}$
	−0.4	$1 - 0.4\frac{f_a}{F'_e}$
	−0.2	$1 - 0.2\frac{f_a}{F'_e}$
	−0.3	$1 - 0.3\frac{f_a}{F'_e}$
	−0.2	$1 - 0.2\frac{f_a}{F'_e}$

For simply supported members

$$\psi = \frac{\pi^2 \delta_o EI}{M_o L^2} - 1$$

where

δ_o = maximum deflection due to transverse loading, in.
M_o = maximum factored design moment between supports due to transverse loading, kip-in.

For restrained ends some limiting cases[98] are given in Table C-H1.1 together with two cases of simply supported beam-columns. These values of C_m are always used with the maximum moment in the member. For the restrained end cases, the values of B_1 will be most accurate if values of $K < 1.0$ corresponding to the end boundary conditions are used in calculating P_e. In lieu of using the equations above, C_m = 1.0

can be used conservatively for transversely loaded members with unrestrained ends and 0.85 for restrained ends.

If, as in the case of a derrick boom, such a beam-column is subject to transverse (gravity) load and a calculable amount of end moment, the value δ_o should include the deflection between supports produced by this moment.

The interaction equations in Appendix H3 have been recommended for biaxially loaded H and wide flange shapes in Refs. 11 and 23. These equations which can be used only in braced frames represent a considerable liberalization over the provisions given in Sect. H1; it is, therefore, also necessary to check yielding under service loads, using the appropriate load and resistance factors for the serviceability limit state in Formula H1-1a or H1-1b with $M_{ux} = S_x F_y$ and $M_{uy} = S_y F_y$.

H2. UNSYMMETRIC MEMBERS AND MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE AND/OR AXIAL FORCE

This section deals with types of cross sections and loadings not covered in Sect. H1, especially where torsion is a consideration. For such cases it is recommended to perform an elastic analysis based on the theoretical numerical methods available from the literature for the determination of the maximum normal and shear stresses, or for the elastic buckling stresses. In the buckling calculations an equivalent slenderness parameter is determined for use in Formulas E2-2 or E3-3, as follows:

$$\lambda_e = \sqrt{F_y / F_e}$$

where F_e is the elastic buckling stress determined from a stability analysis. This procedure is similar to that of Appendix E3.

CHAPTER I. COMPOSITE MEMBERS

I1. DESIGN ASSUMPTIONS

Force Determination

Loads applied to an unshored beam before the concrete has hardened are resisted by the steel section alone, and only loads applied after the concrete has hardened are considered as resisted by the composite section. It is usually assumed for design purposes that concrete has hardened when it attains 75% of its design strength. In beams properly shored during construction, all loads may be assumed as resisted by the composite cross-section. Loads applied to a continuous composite beam with shear connectors throughout its length, after the slab was cracked in the negative moment region, are resisted in that region by the steel section and by properly anchored longitudinal slab reinforcement.

For purposes of plastic analysis all loads are considered resisted by the composite cross section, since a fully plastic strength is reached only after considerable yielding at the locations of plastic hinges.

Elastic Analysis

The use of constant stiffness in elastic analyses of continuous beams is analogous to the practice in reinforced concrete design.

Plastic Analysis

The requirement of the plastic strength of cross section for plastically analyzed beams means that, for composite beams with shear connectors, plastic analysis may be used only when the steel section in the positive moment region has a compact web, i.e., $h_c/t_w \leq 640/\sqrt{F_{yf}}$, and in the negative moment region the steel section is compact, as required for steel beams alone. No compactness limitations are placed on encased beams, but plastic analysis is permitted only if the direct contribution of concrete to the strength of sections is neglected; the concrete is relied upon only to prevent buckling.

Plastic Stress Distribution for Positive Moment

Plastic stress distributions are described in the Commentary in Sect. I3, along with a discussion of the composite participation of slab reinforcement.

Plastic Stress Distribution for Negative Moment

Plastic stress distributions are described in the Commentary in Sect. I3.

Elastic Stress Distribution

The strain distribution at any cross section of a composite beam is related to slip between the structural steel and concrete elements. Prior to slip, strain in both steel and concrete is proportional to the distance from the neutral axis for the elastic transformed section. After slip, the strain distribution is discontinuous, with a jump at the top of the steel shape. The strains in steel and concrete are proportional to distance from separate neutral axes, one for steel and the other for concrete.

Fully Composite Beam

Either the tensile yield strength of the steel section or the compressive strength of the concrete slab governs the maximum flexural strength of a fully composite beam subjected to a positive moment. The tensile yield strength of the longitudinal reinforcing bars in the slab governs the maximum flexural strength of a fully composite beam subjected to a negative moment. When shear connectors are provided in sufficient numbers to fully develop this maximum flexural strength, any slip that occurs prior to yielding is minor and has negligible influence both on stresses and stiffness.

Partially Composite Beam

The effects of slip on elastic properties of a partially composite beam can be significant and should be accounted for in calculations of deflections and stresses at service loads. Approximate elastic properties of partially composite beams are given in the Commentary in Sect. I3.*

Concrete-encased Beam

When the dimensions of a concrete slab supported on steel beams are such that the slab can effectively serve as the flange of a composite T-beam, and the concrete and steel are adequately tied together so as to act as a unit, the beam can be proportioned on the assumption of composite action.

Two cases are recognized: fully encased steel beams, which depend upon natural bond for interaction with the concrete, and those with mechanical anchorage to the slab (shear connectors), which do not have to be encased.

I2. COMPRESSION MEMBERS

1. Limitations

a. The lower limit of 4% on the cross-sectional area of structural steel differentiates between composite and reinforced concrete columns. If the area is less than 4%, a column with a structural steel core should be designed as a reinforced concrete column.
b. The specified minimum quantity of transverse and longitudinal reinforcement in the encasement should be adequate to prevent severe spalling of the surface concrete during fires.
c. Very little of the supporting test data involved concrete strengths in excess of 6 ksi, even though the cylinder strength for one group of four columns was 9.6

*For simplified design methods, see Ref. 37.

ksi. Normal weight concrete is believed to have been used in all tests. Thus, the upper limit of concrete strength is specified as 8 ksi for normal weight concrete. A lower limit of 3 ksi is specified for normal weight concrete and 4 ksi for lightweight concrete to encourage the use of good quality, yet readily available, grades of structural concrete.

d. Encased steel shapes and longitudinal reinforcing bars are restrained from buckling as long as the concrete remains sound. A limit strain of 0.0018, at which unconfined concrete remains unspalled and stable, serves analytically to define a failure condition for composite cross sections under uniform axial strain. The limit strain of 0.0018 corresponds approximately to 55 ksi.

e. The specified minimum wall thicknesses are identical to those in the 1983 ACI Building Code.[2] Their purpose is to prevent buckling of the steel pipe or tubing before yielding.

2. Design Strength

The procedure adopted for the design of axially loaded composite columns is described in detail in Ref. 34, which utilized the equation for the strength of a short column, derived in Ref. 35, and the same reductions for slenderness as those specified for steel columns in Sect. E2. The design follows the same path as the design of steel columns, except that the yield stress of structural steel, the modulus of elasticity of steel and the radius of gyration of the steel section are modified to account for the effect of concrete and of longitudinal reinforcing bars. A detailed explanation of the origin of these modifications may be found in Ref. 35. Reference 34 includes comparisons of the design procedure with tests of 48 axially loaded stub columns, 96 tests of concrete-filled pipes or tubing, and 26 tests of concrete-encased steel shapes. The mean ratio of the test failure loads to the predicted strengths is 1.18 for all 170 tests, and the corresponding coefficient of variation is 0.19.

3. Columns with Multiple Steel Shapes

This limitation is based on Australian research reported in Ref. 76, which demonstrated that after hardening of concrete the composite column will respond to loading as a unit even without lacing, tie plates or batten plates connecting the individual steel sections.

4. Load Transfer

To avoid overstressing either the structural steel section or the concrete at connections, a transfer of load to concrete by direct bearing is required.

When a supporting concrete area is wider on all sides than the loaded area, the maximum design strength of concrete is specified by Ref. 2 as $1.7\phi_B f'_c A_b$ where $\phi_B = 0.7$ is the strength reduction factor in bearing on concrete and A_B is the loaded area. Because the AISC LRFD Specification is based on the lower ANSI A58 load factors, $\phi_B = 0.60$ in the AISC LRFD Specification. The portion of the design load of an axially loaded column P_n resisted by the concrete may be expressed as $(c_2 f'_c A_c / A_s F_{my})\phi_B P_n$.

Accordingly,

$$A_B \geq \frac{\phi_B}{\phi_B} \frac{c_2}{1.7} \frac{A_c}{A_s} \frac{P_n}{F_{my}} = \frac{c_2}{1.7} \frac{A_c}{A_s} \frac{P_n}{F_{my}} \qquad \text{(C-I2-1)}$$

I3. FLEXURAL MEMBERS

1. Effective Width

LRFD provisions for effective width omit any limit based on slab thickness, in accord with both theoretical and experimental studies, as well as current composite beam codes in other countries.[36] The same effective width rules apply to composite beams with a slab on either one side or both sides of the beam. To simplify design, effective width is based on the full span, c.-to-c. of supports, for both simple and continuous beams.

2. Strength of Beams with Shear Connectors

This section applies to simple and continuous composite beams with shear connectors, constructed with or without temporary shores.

Positive Flexural Design Strength

Flexural strength of a composite beam in the positive moment region may be limited by the plastic strength of the steel section, the concrete slab or shear connectors. In addition, web buckling may limit flexural strength if the web is slender and a significantly large portion of the web is in compression.

According to Table B5.1, local web buckling does not reduce the plastic strength of a bare steel beam if the beam depth-to-web thickness ratio is not larger than $640/\sqrt{F_y}$. In the absence of web buckling research on composite beams, the same ratio is conservatively applied to composite beams. Furthermore, for more slender webs, the LRFD Specification conservatively adopts first yield as the flexural strength limit. In this case, stresses on the steel section from permanent loads applied to unshored beams before the concrete has hardened must be superimposed on stresses on the composite section from loads applied to the beams after hardening of concrete. In this superposition, all permanent loads should be multiplied by the dead load factor and all live loads should be multiplied by the live load factor. For shored beams, all loads may be assumed as resisted by the composite section.

When first yield is the flexural strength limit, the elastic transformed section is used to calculate stresses on the composite section. The modular ratio $n = E/E_c$ used to determine the transformed section depends on the specified unit weight and strength of concrete. Note that this procedure for compact beams differs from the requirements of Sect. 1.11.2.2 of the 1978 AISC Specification.[1]

Plastic Stress Distribution for Positive Moment

When flexural strength is determined from the plastic stress distribution shown in Fig. C-I3.1, compression force C in the concrete slab is the smallest of:

$$C = A_{sw}F_{yw} + 2A_{sf}F_{yf} \tag{C-I3-1}$$

$$C = 0.85 f'_c A_c \tag{C-I3-2}$$

$$C = \Sigma Q_n \tag{C-I3-3}$$

For a non-hybrid steel section, Formula C-I3-1 becomes $C = A_s F_y$

where

f'_c = specified compressive strength of concrete, ksi
A_c = area of concrete slab within effective width, in.^{2}
A_s = area of steel cross section, in.^{2}
A_{sw} = area of steel web, in.^{2}
A_{sf} = area of steel flange, in.^{2}
F_y = minimum specified yield stress of steel, ksi
F_{yw} = minimum specified yield stress of web steel, ksi
F_{yf} = minimum specified yield stress of flange steel, ksi
ΣQ_n = sum of nominal strengths of shear connectors between the point of maximum positive moment and the point of zero moment to either side, kips

Longitudinal slab reinforcement makes a negligible contribution to the compression force, except when Formula C-I3-2 governs. In this case, the area of longitudinal reinforcement within the effective width of the concrete slab times the yield stress of the reinforcement may be added in determining C.

The depth of the compression block is

$$a = \frac{C}{0.85\, f'_c b} \qquad \text{(C-I3-4)}$$

where

b = effective width of concrete slab, in.

A fully composite beam corresponds to the case of C governed by the yield strength of the steel beam or the compressive strength of the concrete slab, as in Formulas C-I3-1 or C-I3-2. The number and strength of shear connectors govern C for a partially composite beam as in Formula C-I3-3.

The plastic stress distribution may have the plastic neutral axis (PNA) in the web, in the top flange of the steel section or in the slab, depending on the value of C. These alternatives are shown in Fig. C-I3.1.

For convenient reference, plastic moments for composite beams using a symmetrical non-hybrid steel shape are presented below. The more general case using an unsymmetrical hybrid steel shape is treated in the *Washington University Report No. 45*.[9]

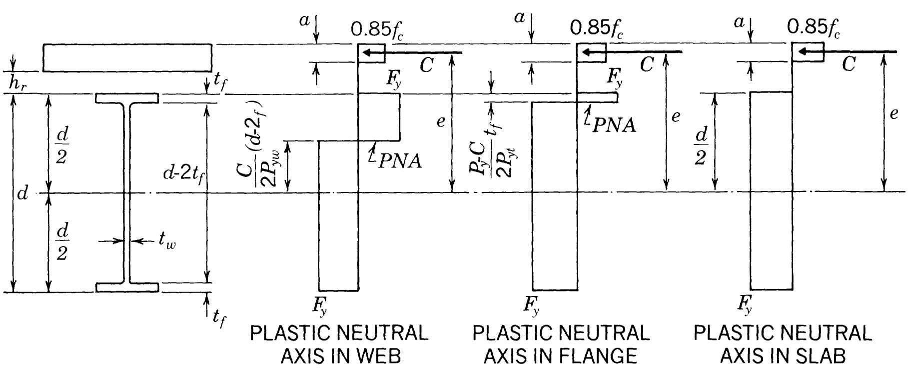


Fig. C-I3.1. Plastic stress distribution for positive moment in composite beams

It is convenient to introduce the following notation for symmetrical steel shapes:

Let

$P_{yw} = (d - 2t_f)\, t_w F_y$ = web yield force, kips
$P_{yf} = 0.5\,(A_s F_y - P_{yw})$ = flange yield force, kips
$P_y = P_{yw} + 2P_{yf}$ = steel section yield force, kips
$M_{pw} = 0.25\, P_{yw}\,(d - 2t_f)$ = web plastic moment, kip-in.
$M_{pf} = P_{yf}\,(d - t_f)$ = flange plastic moment, kip-in.
$M_p = M_{pw} + M_{pf}$ = steel section plastic moment, kip-in.

If the compression force C is less than the web yield force, i.e., $C \leq P_{yw}$, the PNA is in the web and the nominal plastic moment of the composite beam is

$$M_n = M_p - (C/P_{yw})^2\, M_{pw} + Ce \qquad \text{(C-I3-5)}$$

where e = distance from center of steel section to the center of the compression stress block in the slab.
$= 0.5d + h_r + t_c - 0.5a$

The upper limit is reached when $C = P_{yw}$. Then, the PNA is at the top of the web and the nominal plastic moment of the composite beam is

$$M_n = M_{pf} + P_{yw} e \qquad \text{(C-I3-6)}$$

If the compression force C is larger than the web yield force, but smaller than the steel section yield force, i.e., $P_{yw} \leq C \leq P_y$, the PNA is in the steel flange and the nominal plastic moment of the composite beam is

$$M_n = 0.5(P_y - C)\left[d - \left(\frac{P_y - C}{2P_{yf}}\right)t_f\right] + Ce \qquad \text{(C-I3-7)}$$

At the lower limit $C = P_{yw}$, Formula C-I3-7 reduces to Formula C-I3-6 for the PNA at the top of the web. At the upper limit $C = P_y$, the entire steel section is at tension yield and the nominal plastic moment of the composite beam is

$$M_n = P_y e \qquad \text{(C-I3-8)}$$

Formula C-I3-8 also applies when compression force is less than $0.85 f'_c A_c$, i.e., when the neutral axis is in the slab.*

Approximate Elastic Properties of Partially Composite Beams

Elastic calculations for stress and deflection of partially composite beams should include the effects of slip.

The effective moment of inertia I_{eff} for a partially composite beam is approximated by

$$I_{eff} = I_s + \sqrt{(\Sigma Q_n / C_f)}\,(I_{tr} - I_s) \qquad \text{(C-I3-9)}$$

where

I_s = moment of inertia for the structural steel section, in.^{4}
I_{tr} = moment of inertia for the fully composite uncracked transformed section, in.^{4}

*For a simplified design method, see Ref. 37.

ΣQ_n = strength of shear connectors between points of maximum and zero moment, kips

C_f = compression force in concrete slab for fully composite beam; smaller of Formulas C-I3-1 and C-I3-2, kips

The effective section modulus S_{eff}, referred to the tension flange of the steel section for a partially composite beam, is approximated by

$$S_{eff} = S_s + \sqrt{(\Sigma Q_n / C_f)}(S_{tr} - S_s) \qquad \text{(C-I3-10)}$$

where

S_s = section modulus for the structural steel section, referred to the tension flange, in.^{3}

S_{tr} = section modulus for the fully composite uncracked transformed section, referred to the tension flange of the steel section, in.^{3}

Formulas C-I3-9 and C-I3-10 should not be used for ratios $\Sigma Q_n / C_f$ less than 0.25. This restriction is to prevent excessive slip, as well as substantial loss in beam stiffness. Studies indicate that Formulas C-I3-9 and C-I3-10 adequately reflect the reduction in strength and beam stiffness, respectively, when fewer connectors are used than required for full composite action.[38]

Negative Flexural Design Strength

The flexural strength in the negative moment region is the strength of the steel beam alone or the plastic strength of the composite section made up of the longitudinal slab reinforcement and the steel section.

Plastic Stress Distribution for Negative Moment

When an adequately braced compact steel section and adequately developed longitudinal reinforcing bars act compositely in the negative moment region, the nominal flexural strength is determined from the plastic stress distributions as shown in Fig. C-I3.2. The tensile force T in the reinforcing bars is the smaller of:

$$T = A_r F_{yr} \qquad \text{(C-I3-11)}$$

$$T = \Sigma Q_n \qquad \text{(C-I3-12)}$$

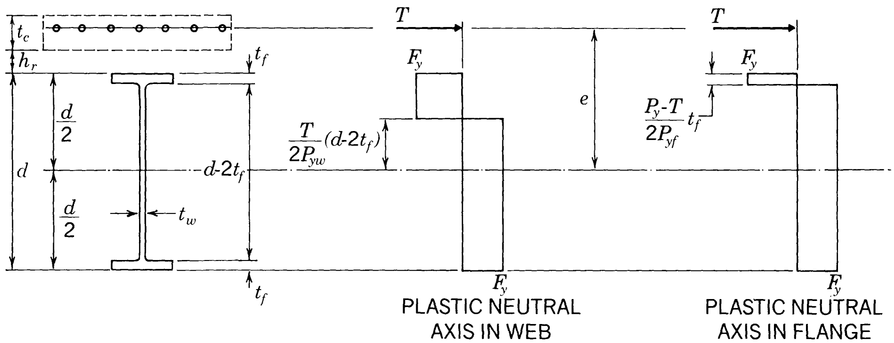


Fig. C-I3.2. Plastic stress distribution for negative moment

where

A_r = area of properly developed slab reinforcement parallel to the steel beam and within the effective width of the slab, in.^{2}

F_{yr} = specified yield stress of the slab reinforcement, ksi

ΣQ_n = sum of the nominal strengths of shear connectors between the points of maximum negative moment and zero moment to either side, kips

A third theoretical limit on T is the product of the area and yield stress of the steel section. However, this limit is redundant in view of practical limitations on slab reinforcement and shear connectors.

The plastic bending strength formulas for positive moment, Formulas C-I3-5 and C-I3-7, can be applied to the negative moment case by substituting T for C and by defining e as the distance from the center of steel section to the centroid of longitudinal slab reinforcement.

In determining web slenderness ratio λ_p, P is replaced with axial tensile force T obtained as the smaller of values given by Formulas C-I3-11 and C-I3-12.

3. Strength of Concrete-encased Beams

Tests of concrete-encased beams demonstrated that (1) the encasement drastically reduces the possibility of lateral-torsional instability and prevents local buckling of the encased steel, (2) the restrictions imposed on the encasement practically prevent bond failure prior to first yielding of the steel section, and (3) bond failure does not necessarily limit the moment capacity of an encased steel beam.[36] Accordingly, LRFD Specification permits two alternate design methods: one based on the first yield in the tension flange of the composite section and the other based on the plastic moment capacity of the steel beam alone. No limitations are placed on the slenderness of either the composite beam or the elements of the steel section, since the encasement effectively inhibits both local and lateral buckling.

In the method based on first yield, stresses on the steel section from permanent loads applied to unshored beams before the concrete has hardened must be superimposed on stresses on the composite section from loads applied to the beams after hardening of concrete. In this superposition, all permanent loads should be multiplied by the dead load factor and all live loads should be multiplied by the live load factor. For shored beams, all loads may be assumed as resisted by the composite section. Complete interaction (no slip) between the concrete and steel is assumed.

The contribution of concrete to the strength of the composite section is ordinarily larger in the positive moment regions than in the negative moment regions. Accordingly, the design based on composite section is more advantageous in the regions of positive moments.

4. Strength During Construction

When temporary shores are not used during construction, the steel beam alone must resist all loads applied before the concrete has hardened enough to provide composite action. Unshored beam deflection caused by wet concrete tends to increase slab thickness and dead load. For longer spans this may lead to instability analogous to roof ponding. An excessive increase of slab thickness may be avoided by beam camber.

When forms are not attached to the top flange, lateral bracing of the steel beam during construction may not be continuous and the unbraced length may control flexural strength, as defined in Sect. F1.

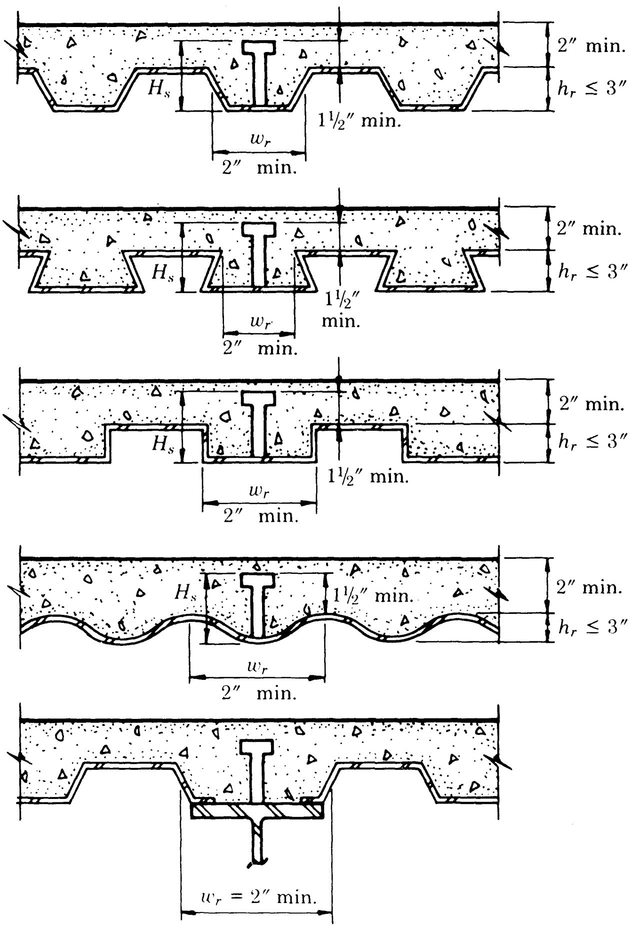


Figure C-I3.3

The LRFD Specification does not include special requirements for a margin against yield during construction. According to Sect. F1, maximum factored moment during construction is 0.90 $F_y Z$, where $F_y Z$ is the plastic moment. (0.90 $F_y Z \approx 0.90 \times 1.1 F_y S$) This is equivalent to approximately the yield moment, $F_y S$. Hence, required flexural strength during construction prevents moment in excess of the yield moment.

Load factors for construction loads should be determined for individual projects according to local conditions, with the factors listed in Sect. A4 as a guide. Once the concrete has hardened, slab weight becomes a permanent dead load and the dead load factor applies to any load combinations.

5. Formed Steel Deck

Figure C-I3.3 is a graphic presentation of the terminology used in Sect. I3.5.

When studs are used on beams with formed steel deck, they may be welded directly through the deck or through prepunched or cut-in-place holes in the deck. The usual procedure is to install studs by welding directly through the deck; however, when the deck thickness is greater than 16 gage for single thickness, or 18 gage for each sheet of double thickness, or when the total thickness of galvanized coating is greater than 1.25 ounces/sq. ft, special precautions and procedures recommended by the stud manufacturer should be followed.

The design rules for composite construction with formed steel deck are based upon a study[38] at Lehigh University of the then available test results. The limiting parameters listed in Sect. I3.5 were established to keep composite construction with formed steel deck within the available research data.

Seventeen full size composite beams with concrete slab on formed steel deck were tested at Lehigh University and the results supplemented by the results of 58 tests performed elsewhere. The range of stud and steel deck dimensions encompassed by the 75 tests were limited to:

1. Stud dimensions: ¾ in.-dia. × 3.00 to 7.00 in.
2. Rib width: 1.94 in. to 7.25 in.
3. Rib height: 0.88 in. to 3.00 in.
4. Ratio w_r/h_r: 1.30 to 3.33
5. Ratio H_s/h_r: 1.50 to 3.41
6. Number of studs in any one rib: 1, 2 or 3

The strength of stud connectors installed in the ribs of concrete slabs on formed steel deck with the ribs oriented perpendicular to the steel beam is reasonably estimated by the strength of stud connectors in flat soffit composite slabs multiplied by values computed from Formula I3-1.

For the case where ribs run parallel to the beam, limited testing[38] has shown that shear connection is not significantly affected by the ribs. However, for narrow ribs, where the ratio w_r/h_r is less than 1.5, a shear stud reduction factor, Formula I3-2, has been suggested in view of lack of test data.

The Lehigh study[38] also indicated that Formula C-I3-10 for effective section modulus and Formula C-I3-9 for effective moment of inertia were valid for composite construction with formed steel deck.

When metal deck includes units for carrying electrical wiring, crossover headers are commonly installed over the cellular deck perpendicular to the ribs. They create trenches which completely or partially replace sections of the concrete slab above the deck. These trenches, running parallel to or transverse to a composite beam, may reduce the effectiveness of the concrete flange. Without special provisions to replace the concrete displaced by the trench, the trench should be considered as a complete structural discontinuity in the concrete flange.

When trenches are parallel to the composite beam, the effective flange width should be determined from the known position of the trench.

Trenches oriented transverse to composite beams should, if possible, be located in areas of low bending moment and the full required number of studs should be placed between the trench and the point of maximum positive moment. Where the trench cannot be located in an area of low moment, the beam should be designed as non-composite.

6. Design Shear Strength

A conservative approach to vertical shear provisions for composite beams is adopted by assigning all shear to the steel section web. This neglects any concrete slab contribution and serves to simplify the design.

I4. COMBINED COMPRESSION AND FLEXURE

The procedure adopted for the design of beam-columns is described and supported by comparisons with test data in Ref. 34. The basic approach is identical to that specified for steel columns in Sect. H1.

The nominal axial strength of a beam-column is obtained from Sect. I2.2, while the nominal flexural strength is determined from the plastic stress distribution on the composite section. An approximate formula for this plastic moment resistance of a composite column is given in Ref. 34:

$$M_n = M_p = ZF_y + \frac{1}{3}(h_2 - 2c_r)A_r F_{yr} + \left(\frac{h_2}{2} - \frac{A_w F_y}{1.7 f_c' h_1}\right) A_w F_y \qquad \text{(C-I4-1)}$$

where

A_w = web area of encased steel shape; for concrete-filled tubes, $A_w = 0$, in.^{2}
Z = plastic section modulus of the steel section, in.^{3}
c_r = average of distance from compression face to longitudinal reinforcement in that face and distance from tension face to longitudinal reinforcement in that face, in.
h_1 = width of composite cross section perpendicular to the plane of bending, in.
h_2 = width of composite cross section parallel to the plane of bending, in.

The supporting comparisons with beam-column tests included 48 concrete-filled pipes or tubing and 44 concrete-encased steel shapes.[34] The overall mean test-to-prediction ratio was 1.23 and the coefficient of variation 0.21.

The last paragraph in Sect. I4 provides a transition from beam-columns to beams. It involves bond between the steel section and concrete. Section I3 for beams requires either shear connectors or full, properly reinforced encasement of the steel section. Furthermore, even with full encasement, it is assumed that bond is capable of developing only the moment at first yielding in the steel of the composite section. No test data are available on the loss of bond in composite beam-columns. However, consideration of tensile cracking of concrete suggests $P_u / \phi_b P_n = 0.3$ as a conservative limit. It is assumed that when $P_u / \phi_b P_n$ is less than 0.3, the nominal flexural strength is reduced below that indicated by plastic stress distribution on the composite cross section unless the transfer of shear from the concrete to the steel is provided for by shear connectors.

I5. SHEAR CONNECTORS

1. Materials

Tests[39] have shown that fully composite beams with concrete meeting the requirements of Part 3, Chapter 4, "Concrete Quality," of Ref. 2, made with ASTM C33 or rotary-kiln produced C330 aggregates, develop their full flexural capacity.

2. Horizontal Shear Force

Composite beams in which the longitudinal spacing of shear connectors has been varied according to the intensity of statical shear, and duplicate beams where the number of connectors were uniformly spaced, have exhibited the same ultimate strength and the same amount of deflection at normal working loads. Only a slight deformation in the concrete and the more heavily stressed connectors is needed to redistribute the horizontal shear to other less heavily stressed connectors. The important consideration is that the total number of connectors be sufficient to develop the shear V_h on either side of the point of maximum moment. The provisions of the LRFD Specification are based upon this concept of composite action.

In computing the design flexural strength at points of maximum negative bending, reinforcement parallel to the steel beam and lying within the effective width of slab may be included, provided such reinforcement is properly anchored beyond the region of negative moment. However, enough shear connectors are required to transfer, from the slab to the steel beam, the ultimate tensile force in the reinforcement.

3. Strength of Stud Shear Connectors

Studies have defined stud shear connector strength in terms of normal weight and lightweight aggregate concretes as a function of both concrete modulus of elasticity and concrete strength as given by Formula I5-1.

Formula I5-1, obtained from Ref. 39, corresponds to Tables 1.11.4 and 4A in Sect. 1.11.4 of the 1978 AISC Specification.[1] Note that an upper bound on stud shear strength is the product of the cross-sectional area of the stud times its ultimate tensile strength.

The LRFD Specification does not specify a resistance factor for shear connector strength. The resistance factor for the flexural strength of a composite beam accounts for all sources of variability, including those associated with the shear connectors.

4. Strength of Channel Shear Connectors

Formula I5-2 is a modified form of the formula for the strength of channel connectors developed by Slutter and Driscoll.[40] The modification has extended its use to lightweight concrete.

6. Shear Connector Placement and Spacing

As in Part 1 of the 1978 AISC Specification,[1] uniform spacing of shear connectors is permitted except in the presence of heavy concentrated loads. The second sentence in the first paragraph of Sect. I5.6 serves the same purpose as Formula 1.11-7 in Sect. 1.11.4 of Ref. 1.

When stud shear connectors are installed on beams with formed steel deck, concrete cover at the sides of studs adjacent to sides of steel ribs is not critical. Tests have shown that studs installed as close as is permitted to accomplish welding of studs does not reduce the composite beam capacity.

Studs not located directly over the web of a beam tend to tear out of a thin flange before attaining their full shear-resisting capacity. To guard against this contingency, the size of a stud not located over the beam web is limited to 2½ times the flange thickness.[87]

The minimum spacing of connectors along the length of the beam, in both flat soffit concrete slabs and when ribs of formed steel deck are parallel to the beam, is 6 diameters; this spacing reflects development of shear planes in the concrete slab.[39] Since most test data are based on the minimum transverse spacing of 4 diameters, this transverse spacing was set as the minimum permitted. If the steel beam flange is narrow, this spacing requirement may be achieved by staggering the studs with a minimum transverse spacing of 3 diameters between the staggered row of studs. The reduction in connector capacity in the ribs of formed steel decks is provided by the factor $0.85/\sqrt{N_r}$, which accounts for the reduced capacity of multiple connectors, including the effect of spacing. When deck ribs are parallel to the beam and the design

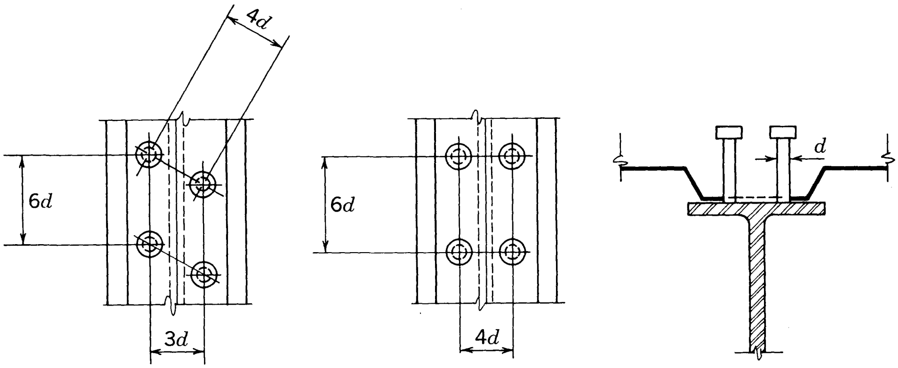


Fig. C-I5.1. Connector arrangements

requires more studs than can be placed in the rib, the deck may be split so that adequate spacing is available for stud installation. Figure C-I5.1 shows possible connector arrangements.

I6. SPECIAL CASES

This section is a modified version of Sect. 1.11.6 of the 1978 AISC Specification.[1] The phrase "and details of construction" was added in recognition of the fact that different types of shear connectors may require different spacing and other detailing than stud and channel connectors.

CHAPTER J.
CONNECTIONS JOINTS AND FASTENERS

J1. GENERAL PROVISIONS

6. Placement of Welds and Bolts

Slight eccentricities between the gravity axis of single and double angle members and the center of gravity of their connecting rivets or bolts have long been ignored as having negligible effect on the static strength of such members. Tests[41] have shown that similar practice is warranted in the case of welded members in statically loaded structures.

However, the fatigue life of eccentrically loaded welded angles has been shown to be very short.[42] Notches at the roots of fillet welds are harmful when reverse stresses are normal to the axis of the weld, as could occur when axial cyclic loading is applied to angles with end welds not balanced about the neutral axis. Accordingly, balanced welds are indicated when such members are subjected to cyclic loading (see Fig. C-J1.1).

7. Bolts in Combination with Welds

Welds will not share the load equally with mechanical fasteners in bearing-type connections. Before ultimate loading occurs, the fastener will slip and the weld will carry an indeterminately larger share of the load.

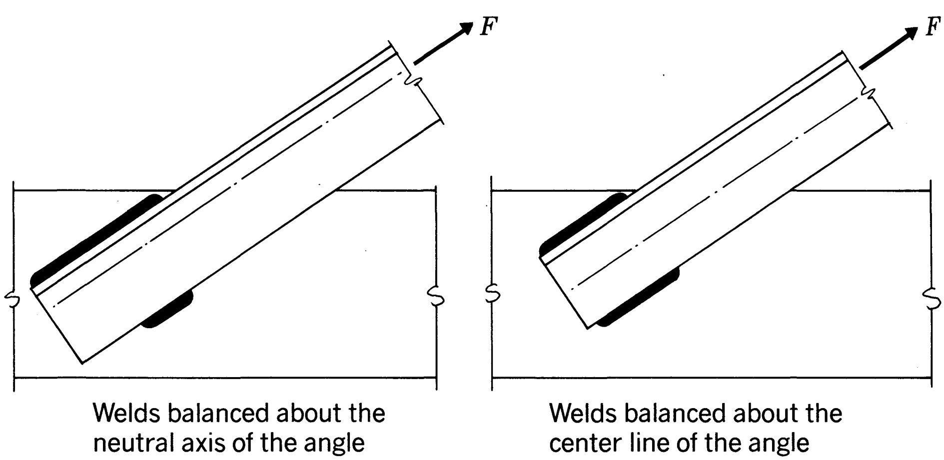


Fig. C-J1.1. Welds at end connection

Accordingly, the sharing of load between welds and A307 bolts or high-strength bolts in a bearing-type connection is not recommended. For similar reasons, A307 bolts and rivets should not be assumed to share loads in a single group of fasteners.

For high-strength bolts in slip-resistant connections to share the load with welds it is advisable to properly torque the bolt before the weld is made. Were the weld to be placed first, angular distortion from the heat of the weld might prevent the faying action required for development of the slip-resistant force. When the bolts are properly torqued before the weld is made, the slip-resistant bolts and the weld may be assumed to share the load on a common shear plane.[43] The heat of welding near bolts will not alter the mechanical properties of the bolt.

In making alterations to existing structures, it is assumed that whatever slip is likely to occur in high-strength bolted bearing-type connections or riveted connections will have already taken place. Hence, in such cases the use of welding to resist all stresses, other than those produced by existing dead load present at the time of making the alteration, is permitted.

It should be noted that combination of fasteners as defined herein does not refer to connections such as shear plates for beam-to-column connections which are welded to the column and bolted to the beam flange or web[43] and other comparable connections.

8. High-strength Bolts in Combination with Rivets

When high-strength bolts are used in combination with rivets, the ductility of the rivets permits the direct addition of the capacity of both fastener groups.

J2. WELDS

1. Groove Welds

The engineer preparing contract design drawings cannot specify the depth of groove without knowing the welding process and the position of welding. Accordingly, only the effective throat for partial-penetration groove welds should be specified on design drawings, allowing the fabricator to produce this effective throat with his own choice of welding process and position.

The weld reinforcement is not used in determining the effective throat thickness of a groove weld (see Table J2.1).

2. Fillet Welds

a. Effective Area

The effective throat of a fillet weld is based upon the root of the joint and the face of the diagrammatic weld, hence this definition gives no credit for weld penetration or reinforcement at the weld face. If the fillet weld is made by the submerged arc welding process, some credit for penetration is made. If the leg size of the resulting fillet weld does not exceed ⅜-in., then 0.11 in. is added to the theoretical throat. This increased weld throat is allowed because the submerged arc process produces deep penetration welds of consistent quality. However, it is necessary to run a short length of fillet weld to be assured that this increased penetration is obtained.

In practice, this is usually done initially by cross-sectioning the runoff plates of the joint. Once this is done, no further testing is required, as long as the welding procedure is not changed.

b. Limitations

Table J2.5 provides a minimum size of fillet weld for a given thickness of the thicker part joined.

The requirements are not based upon strength considerations, but upon the quench effect of thick material on small welds. Very rapid cooling of weld metal may result in a loss of ductility. Further, the restraint to weld metal shrinkage provided by thick material may result in weld cracking. Because a 5⁄16-in. fillet weld is the largest that can be deposited in a single pass by manual process, 5⁄16-in. applies to all material 3⁄4-in. and greater in thickness, but minimum preheat and interpass temperature are required by AWS D1.1.* Both the design engineer and the shop welder must be governed by the requirements.

Table J2.4 gives the minimum effective throat of a partial-penetration groove weld. Notice that Table J2.4 for partial-penetration groove welds goes up to a plate thickness of over 6 in. and a minimum weld throat of 5⁄8-in., whereas, for fillet welds, Table J2.5 goes up to a plate thickness of over 3⁄4-in. and a minimum leg size of fillet weld of only 5⁄16-in. The additional thickness for partial-penetration welds is to provide for reasonable proportionality between weld and material thickness.

For plates of 1⁄4-in. or more in thickness, it is necessary that the inspector be able to identify the edge of the plate to position the weld gage. This is assured if the weld is kept back at least 1⁄16-in. from this edge, as shown in Fig. C-J2.1.

Where longitudinal fillet welds are used alone in a connection (see Fig. C-J2.4), Sect. J2.2b requires the length of each weld to be at least equal to the width of the connecting material, because of shear lag.[44] (See Fig. C-J2.2.) By providing a minimum lap of five times the thickness of the thinner part of a lap joint, the resulting rotation of the joint when pulled will not be excessive, as shown in Fig. C-J2.3.

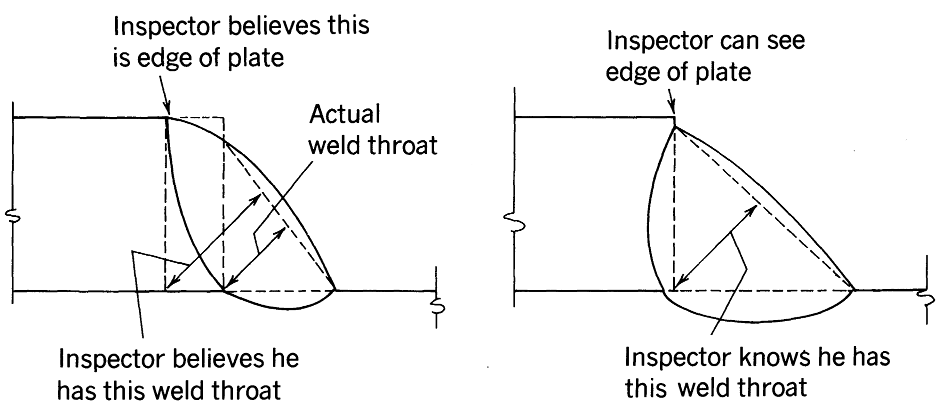


Fig. C-J2.1. Identification of plate edge

*See Table J2.5.

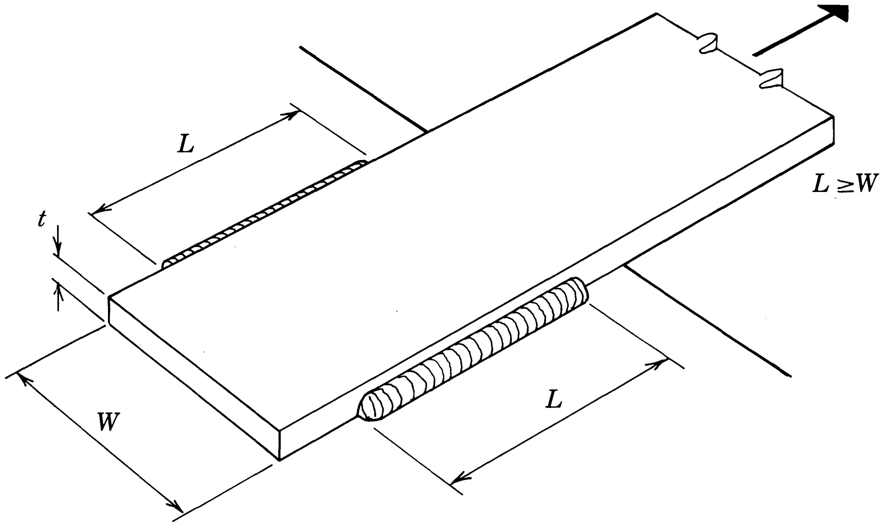


Fig. C-J2.2. Longitudinal fillet welds

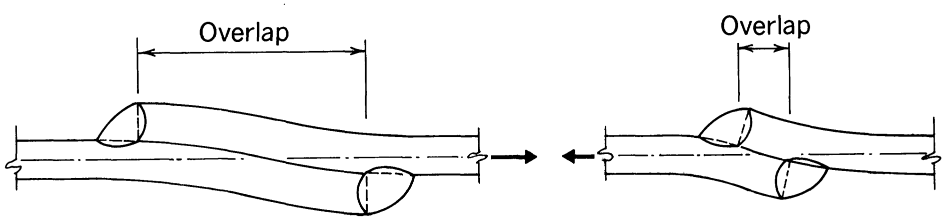


Fig. C-J2.3. Minimum lap

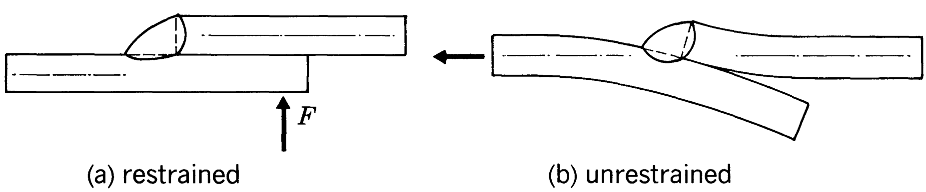


Fig. C-J2.4. Restraint of lap joints

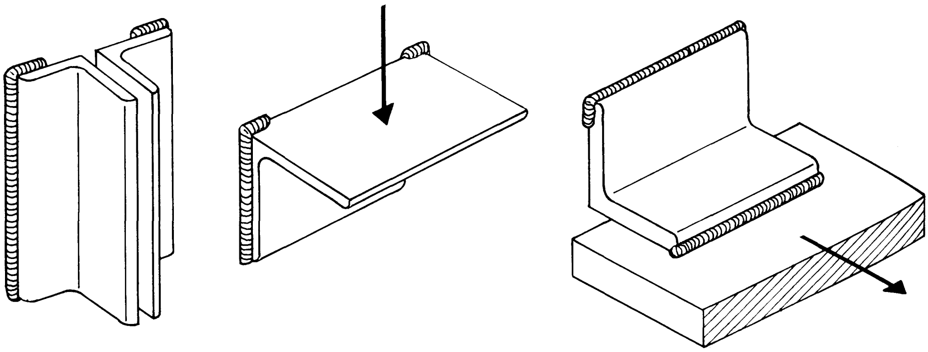


Fig. C-J2.5. Return welds

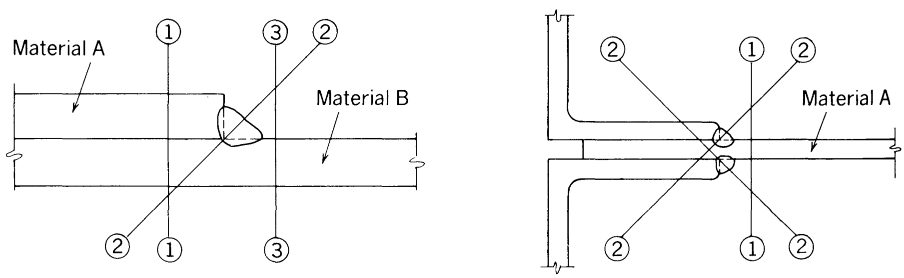


Fig. C-J2.6. Shear planes for fillet welds loaded in longitudinal shear

Fillet welded lap joints under tension tend to open and apply a tearing action at the root of the weld as shown in Fig. C-J2.4b, unless restrained by a force F as shown in Fig. C-J2.4a.

While end returns (see Fig. C-J2.5) do not substantially increase the strength of the connection, they do increase the carrying capacity of the structure by providing more ultimate plastic movement of the connection before failure. The end return delays the initial tearing of the weld in the connection.

4. Design Strength

The strength of welds is governed by the strength of either the base material or the deposited weld metal. Table J2.3 contains the resistance factors and nominal weld strengths, as well as a number of limitations. It is analogous to Table 1.5.3 in the 1978 AISC Specification.[1]

It should be noted that in Table J2.3 the nominal strength of fillet welds is determined from the effective throat area, whereas the strength of the connected parts is governed by their respective thicknesses. Figure C-J2.6 illustrates the shear planes for fillet welds and base material:

a. Plane 1-1, in which the resistance is governed by the shear strength for material A

b. Plane 2-2, in which the resistance is governed by the shear strength of the weld metal
c. Plane 3-3 in which the resistance is governed by the shear strength of material B

The resistance of the welded joint is the lowest of the resistances calculated in each plane of shear transfer. Note that planes 1-1 and 3-3 are positioned away from the fusion areas between the weld and the base material. Tests have demonstrated that the stress on this fusion area is not critical in determining the shear strength of fillet welds.*

5. Combination of Welds

This method of adding weld capacities does not apply to a welded joint using a partial-penetration single bevel groove weld with a superimposed fillet weld. In this case, the effective throat of the combined joint must be determined and the design capacity based upon this throat area.

J3. BOLTS, THREADED PARTS AND RIVETS

1. High-strength Bolts

In general, the use of high-strength bolts is required to conform to the provisions of the *Specification for Structural Joints Using ASTM A325 or A490 Bolts*[68] as approved by the Research Council on Structural Connections.

Occasionally the need arises for the use of high-strength bolts of diameters in excess of those available for A325 and A490 bolts, as for example, anchor bolts for fastening machine castings. For this situation Sect. A3.3 permits the use of A449 bolts.

3. Design Tension or Shear Strength

Tension loading of fasteners is usually accompanied by some bending due to the deformation of the connected parts. Hence, the resistance factor ϕ, by which R_n is multiplied to obtain the design tensile strength of fasteners, is relatively low. The nominal strength values in Table J3.2 were obtained from the formula

$$R_n = 0.75\, A_b F_u \tag{C-J3-1}$$

While the formula was developed for bolted connections (Ref. 43, p. 68), it was also conservatively applied to threaded parts and to rivets. The nominal strength of A307 bolts was discounted by 5 ksi.

In connections consisting of only a few fasteners, the effects of strain on the shear in bearing fasteners is negligible.[43,45] In longer joints, the differential strain produces an uneven distribution between fasteners (those near the end taking a disproportionate part of the total load), so that the maximum strength per fastener is reduced. The 1978 AISC Specification[1] permits connections up to 50 in. in length without a reduction in maximum shear stress. With this in mind the resistance factor ϕ for shear in bearing-type connections has been adjusted to accommodate the same range of connections.

The values of nominal strength in Table J3.2 were obtained from the formula

$$R_n / mA_b = 0.60 F_u \tag{C-J3-2}$$

*F. R. Preece, "AWS-AISC Fillet Weld Study—Longitudinal and Transverse Shear Tests," Testing Engineers, Inc., Los Angeles, May 31, 1968.

when threads are excluded from the shear planes and

$$R_n / mA_b = 0.45F_u \qquad \text{(C-J3-3)}$$

when threads are not excluded from the shear plane (Ref. 43, p. 68), where m is the number of shear planes. While developed for bolted connections,[43] the formulas were also conservatively applied to threaded parts and rivets. The value given for A307 bolts was obtained from Formula C-J3-3 but is specified for all cases regardless of the position of threads. For A325 bolts, no distinction is made between small and large diameters, even though the minimum tensile strength F_u is lower for bolts with diameters in excess of 1 in. It was felt that such a refinement of design was not justified, particularly in view of the low resistance factor ϕ and other compensating factors.

4. Combined Tension and Shear in Bearing-type Connections

Tests have shown that the strength of bearing fasteners subject to combined shear and tension resulting from externally applied forces can be closely defined by an ellipse.[43] Such a curve can be replaced, with only minor deviations, by three straight lines as shown in Fig. C-J3.1. This latter representation offers the advantage that no modification of either type stress is required in the presence of fairly large magnitudes of other types. This linear representation was adopted for Table J3.3, giving a limiting tensile stress F_t as a function of the shearing stress f_v for bearing-type connections.

5. High-strength Bolts in Slip-critical Joints

The onset of slipping in a high-strength bolted, slip-critical connection is not an indication that maximum capacity of the connection has been reached. Its occurrence may be only a serviceability limit state. However, poor fatigue performance due to joint slippage may occur in slip-critical connections subjected to repeated loads producing large stress reversal. If slip occurs in slip-critical connections with oversized or slotted holes, the rigid body movement could introduce second order effects that would reduce the maximum load capacity of the structure. Slip of slip-critical connections is likely to occur at approximately 1.4 to 1.5 times the service loads.

While the possibility of a slip-critical connection slipping into bearing under anticipated service conditions is small, such connections must comply with the provisions of Sect. J3.6 in order to prevent connection failure at the maximum load condition.

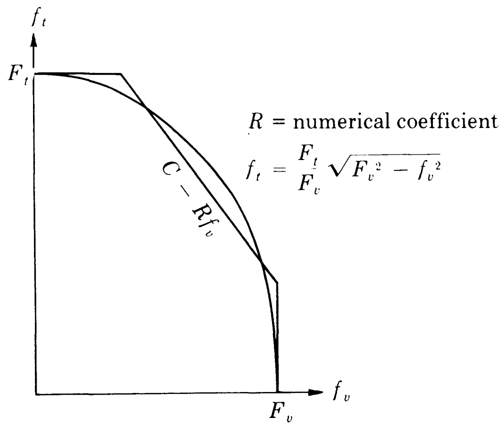


Figure C-J3.1

6. Bearing Strength at Bolt Holes

The recommended bearing stress on pins is not the same as for bolts.

Bearing values are not provided as a protection to the fastener, because it needs no such protection. Therefore, the same bearing value applies to joints assembled by bolts, regardless of fastener shear strength or the presence or absence of threads in the bearing area.

Recent tests[96] have demonstrated that hole elongation greater than 0.25 in. will begin to develop as the bearing stress is increased beyond the values given in Formulas J3-1a and J3-1b, especially if it is combined with high tensile stress on the net section, even though rupture does not occur. Formula J3-1d considers the effect of hole ovalization.

7. Size and Use of Holes

To provide some latitude for adjustment in plumbing up a frame during erection, three types of enlarged size holes are permitted, subject to the approval of the designer. The nominal maximum sizes of these holes are given in Table J3.5.

The use of these oversize holes is restricted to connections assembled with bolts and is subject to the provisions of Sects. J3.9 and J3.10.

8. Long Grips

Provisions requiring a decrease in calculated stress for A307 bolts having long grips (by arbitrarily increasing the required number in proportion to the grip length) are not required for high-strength bolts. Tests[46] have demonstrated that the ultimate shearing strength of high-strength bolts having a grip of 8 or 9 diameters is no less than that of similar bolts with much shorter grips.

9. Minimum Spacing

The *maximum* factored strength R_n at a bolt or rivet hole in bearing requires that the distance between the centerline of the first fastener and the edge of a plate toward which the force is directed should not be less than $1\frac{1}{2}d$, where d is the fastener diameter.[43] By similar reasoning the distance measured in the line of force, from the centerline of any fastener to the nearest edge of an adjacent hole, should not be less than $3d$, to insure maximum design strength in bearing. Plotting of numerous test results indicates that the critical bearing strength is directly proportional to the above defined distances up to a maximum value of $3d$, above which no additional bearing strength is achieved.[43] By adding $d/2$, Formula J3-2 defines the maximum pitch in terms of bearing capacity for standard holes. Table J3.6 lists the increments that must be added to adjust the spacing upward to compensate for an increase in hole dimension parallel to the line of force.

10. Minimum Edge Distance

Critical bearing stress is a function of the material tensile strength, the spacing of fasteners and the distance from the edge of the part to the center line of the nearest fastener. Tests have shown* that a linear relationship exists between the ratio of

*Ref. 43, pp. 109–112, 137.

critical bearing stress to tensile strength (of the connected material) and the ratio of fastener spacing (in the line of force) to fastener diameter. The following equation affords a good lower bound to published test data for single-fastener connections with standard holes, and is conservative for adequately spaced multi-fastener connections:

$$\frac{F_{pcr}}{F_u} = \frac{\ell_e}{d} \tag{C-J3-4}$$

where

F_{pcr} = critical bearing stress, ksi
F_u = tensile strength of the connected material, ksi
ℓ_e = distance, along a line of transmitted force, from the center of a fastener to the nearest edge of an adjacent fastener or to the free edge of a connected part (in the direction of stress), in.
d = diameter of a fastener, in.

The above equation, modified by a resistance factor $\phi = 0.75$, is the basis for Formulas J3-2 and J3-3 in the LRFD Specification.

The provisions of Sect. J3.9 are concerned with ℓ_e as hole spacing, whereas Sect. J3.10 is concerned with ℓ_e as edge distance in the direction of stress. Section J3.6 establishes a maximum bearing strength. Spacing and/or edge distance may be increased to provide for a required bearing strength, or bearing force may be reduced to satisfy a spacing and/or edge distance limitation.

It has long been known that the critical bearing stress of a single fastener connection is more dependent upon a given edge distance than multi-fastener connections.[63] For this reason, longer edge distances (in the direction of force) are required for connections with one fastener in the line of transmitted force than required for those having two or more.

11. Maximum Edge Distance and Spacing

Limiting the edge distance to not more than 12 times the thickness of an outside connected part, but not more than 6 in., is intended to provide for the exclusion of moisture in the event of paint failure, thus preventing corrosion between the parts which might accumulate and force these parts to separate. More restrictive limitations are required for connected parts of unpainted steel exposed to atmospheric corrosion.

J4. DESIGN SHEAR RUPTURE STRENGTH

Tests[47] on coped beams indicated that a tearing failure mode can occur along the perimeter of the bolt holes as shown in Fig. C-J5.1. This block shear mode combines tensile strength on one plane and shear strength on a perpendicular plane. The 1978 AISC Specification[1] adopted an analytical model which sums the tensile net area strength and shear net area fracture strength to predict the block shear strength. The shear fracture stress is taken as $0.6F_u$. The failure path is defined by the center lines of the bolt holes. The block shear failure mode is not limited to the coped ends of beams. Other examples are shown in Fig. C-J5.2. The block shear failure mode should also be checked around the periphery of welded connections.

Based on more recent tests,[101,102] the LRFD Specification has adopted a more conservative model to predict block shear strength. The previous model[1] summed the fracture strengths on two perpendicular planes which implies that ultimate fracture strength on both planes occur simultaneously. If fracture occurs first on one plane, the

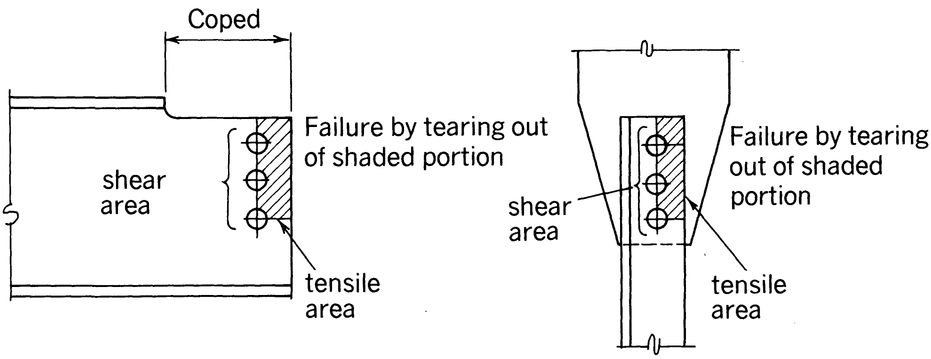


Fig. C-J5.1. Failure surface for tearing mode

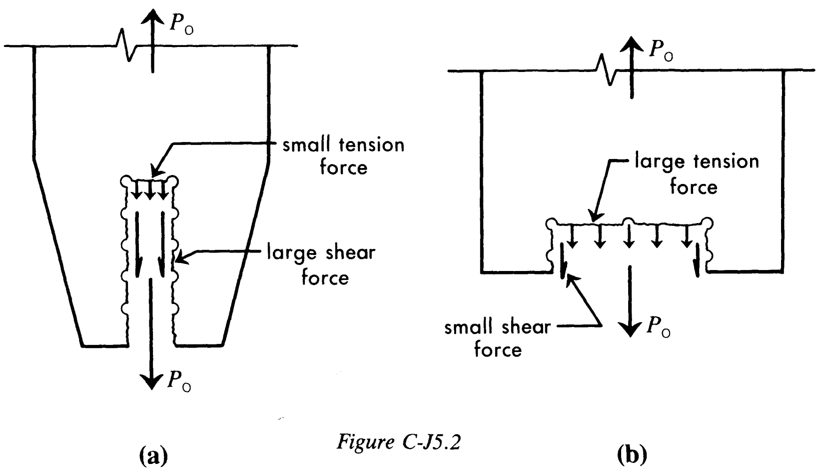


Figure C-J5.2

strength is lost and the total force must be supported by the perpendicular plane. Test results suggest that it is reasonable to add the yield strength on one plane to the fracture strength of the perpendicular plane. Therefore, two possible block shear strengths can be calculated; fracture strength F_u on the net tensile section along with shear yielding $0.6\,F_y$ on the gross section on the shear plane(s) or fracture $0.6F_u$ on the net shear area(s) combined with yielding F_y on the gross tensile area as given by the following formulas:

$$\phi[0.6\,F_y A_{vg} + F_u A_{nt}] \qquad \text{(C-J4-1)}$$

$$\phi[0.6\,F_u A_{ns} + F_y A_{tg}] \qquad \text{(C-J4-2)}$$

where

ϕ = 0.75
A_{vg} = gross area subjected to shear, in.^{2}
A_{tg} = gross area subjected to tension, in.^{2}
A_{ns} = net area subjected to shear, in.^{2}
A_{nt} = net area subjected to tension, in.^{2}

These formulas are consistent with the philosophy in Chap. D for tension members, where gross area is used for the limit state of yielding and net area is used for fracture. The controlling equation is one that produces the *larger* force. This can be explained by the two extreme examples given in Fig. C-J5.2. In Case a, the total force is resisted primarily by shear, therefore shear fracture, not shear yielding, should control the block shear tearing mode; therefore, use Formula C-J4-2. For Case b, block shear cannot occur until the tension area fractures as given by Formula C-J4-1. If Formula C-J4-2 (shear fracture on the small area and yielding on the large tension area) is checked for Case b, a smaller P_o will result. In fact, as the shear area gets smaller and approaches zero, the use of Formula C-J4-2 for Case b would give a block shear strength based totally on *yielding* of the gross tensile area. Block shear is a fracture or tearing phenomenon not a yielding limit state. Therefore, the proper formula to use is the one in which the fracture term is larger than the yield term. When it is not obvious which failure plane fractures, it is easier just to use the larger of the two formulas.

J5. CONNECTING ELEMENTS

2. Design Strength of Connecting Elements

Tests have shown that yield will occur on the gross section area before the tensile capacity of the net section is reached, if the ratio $A_n/A_g \leq 0.85$.[43] Since the length of connecting elements is small compared to the member length, inelastic deformation of the gross section is limited. Hence, the effective net area A_n of the connecting element is limited to $0.85 A_g$ in recognition of the limited inelastic deformation and to provide a reverse capacity.

J6. FILLERS

The practice of securing fillers by means of additional fasteners, so that they are in effect an integral part of a shear-connected component, is not required where a connection is designed to be a slip-critical connection using high-strength bolts. In such connections, the resistance to slip between filler and either connected part is comparable to that which would exist between the connected parts if no fill were present.

Filler plates may be used in lap joints of welded connections that splice parts of different thickness, or where there may be an offset in the joint.

J8. BEARING STRENGTH

The LRFD Specification provisions for bearing on milled surfaces, Sect. J8.1, and on expansion rollers and rockers, Sect. J8.2, are equivalent to those provisions in the 1978 AISC Specification,[1] Sects. 1.5.1.5.1 and 1.5.1.5.2, respectively. The specific formulas were transformed from allowable stress to LRFD format by multiplying the numerical constants 0.9 and 0.66 by 1.67, resulting in 1.5 and 1.1 for their LRFD equivalents.

As used throughout the LRFD Specification, the terms "milled surface," "milled" or "milling" are intended to include surfaces which have been accurately sawed or finished to a true plane by any suitable means.

J9. COLUMN BASES AND BEARING ON CONCRETE

The formulas for resistance of concrete in bearing are the same as ACI 318-83, except that AISC formulas are used with $\phi = 0.60$ while ACI uses $\phi = 0.70$. The difference is because ACI specifies larger load factors than the ANSI load factors specified by AISC.

CHAPTER K.
STRENGTH DESIGN CONSIDERATIONS

K1. WEBS AND FLANGES WITH CONCENTRATED FORCES

The LRFD Specification separates web strength requirements into distinct categories representing different limit state criteria, i.e., local flange bending (Sect. K1.2), local web yielding (Sect. K1.3), web crippling (Sect. K1.4), sidesway web buckling (Sect. K1.5) and column buckling of the web (Sect. K1.6). The following is a discussion of each criterion with respect to a concentrated load applied at an interior location; the same argument would be valid for an exterior reaction load except as modified by the appropriate equations. An interior location is defined as being at least a distance equal to the member's depth from the end of the member.

2. Local Flange Bending

Where a tension force is applied through a plate welded to a flange, that flange must be sufficiently rigid not to deform and cause an area of high stress concentration in the weld in line with the web. This is handled in the 1978 AISC Specification[1] by Formula 1.15-3 which has been retained in the LRFD Specification, in a modified form, as Formula K1-1.

3. Local Web Yielding

The web strength criteria have been established to limit the stress in the web of a member into which a force is being transmitted. It should matter little whether the member receiving the force is a beam or a column; however, the reference material[9,1] upon which the LRFD Specification is based did make such a distinction. For beams, a 2:1 stress gradient through the flange was used, whereas the gradient was 2½:1 through column flanges. In Sect. K1.3, the 2½:1 gradient is used for both.

4. Web Crippling

The expression for resistance to web crippling at a concentrated load is a departure from previous specifications.[30,31,32,64] Formulas K1-4 and K1-5 are based on research by Roberts.[88]

5. Sidesway Web Buckling

The sidesway web buckling criteria were developed after observing several unexpected failures in beams tested at the University of Texas-Austin.[33] In these tests the compression flanges were braced at the concentrated load, the web was squeezed into compression and the tension flange buckled (see Fig. C-K1.1).

Sidesway web buckling will not occur in the following cases:

For flanges restrained against rotation:

$$\frac{d_c/t_w}{\ell/b_f} > 2.3$$

For flange rotation not restrained:

$$\frac{d_c/t_w}{\ell/b_f} > 1.7$$

Sidesway web buckling can also be prevented by the proper design of lateral bracing or stiffeners at the load point. It is suggested that local bracing at both flanges be designed for 1% of the concentrated load applied to that point. Stiffeners must extend from the load point through at least one-half the girder depth. In addition, the pair of stiffeners should be designed to carry the full load. If flange rotation is permitted at the loaded flange, stiffeners will not be effective.

7. Compression Members with Web Panels Subject to High Shear

In rigid beam-to-column connections, the column web stresses are often quite high and may require a pair of diagonal stiffeners or a web doubler plate. Referring to Fig. C-K1.2, a minimum web thickness not requiring reinforcement is computed as follows:

The factored column web strength is ϕR_v, where

$$\phi = 0.90$$
$$R_v = 0.7\, F_{yc} d_c t_w$$

The maximum beam forces are assumed to act through the flanges as a couple with a moment arm of $0.95d_b$. Noting that the factored resistance must be greater than the factored load, the following inequality results:

$$(0.9)\,(0.7)\,F_{yc} d_c t_w \geq \frac{12}{0.95d_b}(M_1 + M_2) - V_u$$

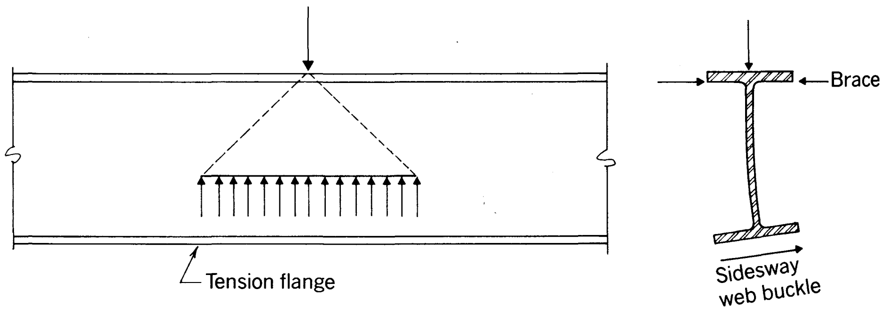


Fig. C-K1.1. Sidesway web buckling

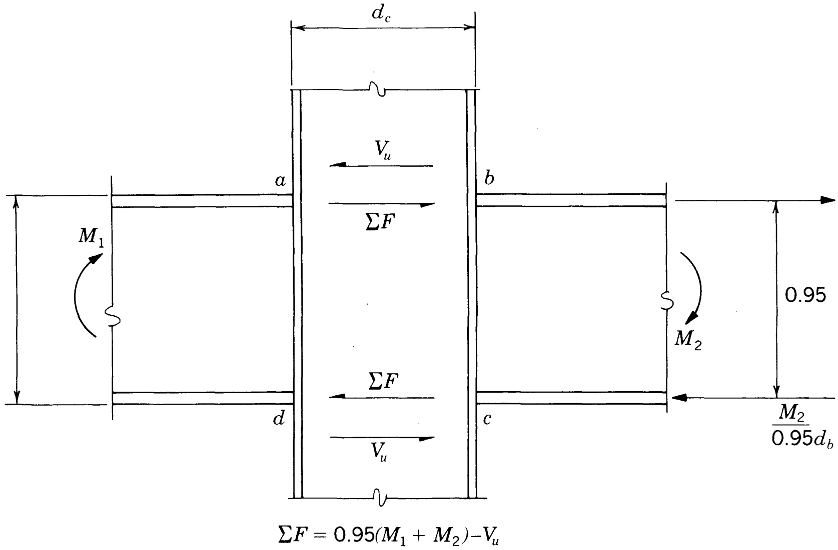


Fig. C-K1.2. Equilibrium of forces on web

Solving the above equation for t_w gives:

$$t_w = \frac{20.1(M_1 + M_2) - 1.6V_u d_b}{F_{yc} A_{bc}} = t_{min}$$

where

M_1, M_2 = factored beam moments, kip-in.
V_u = factored column shear, kips
F_{yc} = yield strength of the column web, ksi
A_{bc} = planar area bounded by the points *abcd* in Fig. C-K1.2, in.^{2}

For webs subject to high shear in combination with axial loads exceeding $0.75P_n$, Formula K1-10 was developed to give a straight line transition from maximum shear at 0.75 axial load to maximum axial load at 0.70 shear; see Fig. C-K1.3.

K2. PONDING

As used in the LRFD Specification, *ponding* refers to the retention of water due solely to the deflection of flat roof framing. The amount of this water is dependent upon the flexibility of the framing. Lacking sufficient framing stiffness, its accumulated weight can result in collapse of the roof if a strength evaluation is not made (see ANSI A58.1).

Representing the deflected shape of the primary and critical secondary member as a half-sine wave, the weight and distribution of the ponded water can be estimated

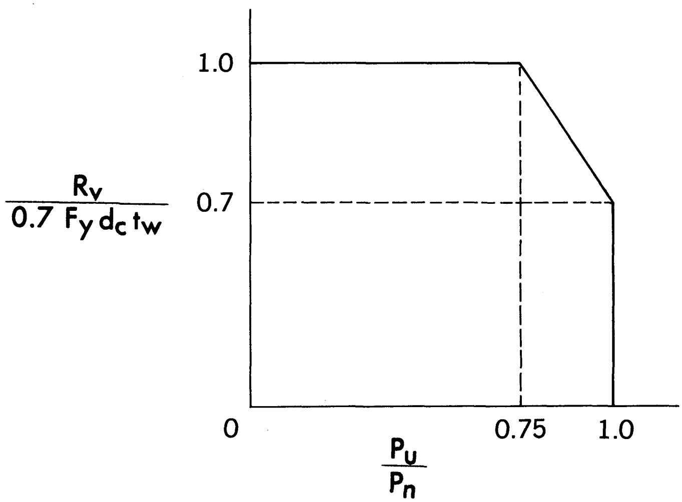


Fig. C-K1.3. Interaction of shear and axial force

and, from this, the contribution that the deflection each of these members makes to the total ponding deflection can be expressed:[65]

For the primary member:

$$\Delta_w = \frac{\alpha_p \Delta_o 1 + 0.25\pi\alpha_s + 0.25\pi\ \rho(1 + \alpha_s)}{1 - 0.25\pi\alpha_p \alpha_s}$$

For the secondary member:

$$\delta_w = \frac{\alpha_s \delta_o 1 + \dfrac{\pi^3}{32}\alpha_p + \dfrac{\pi^2}{8\rho}(1 + \alpha_p) + 0.185\alpha_s \alpha_p}{1 - 0.25\pi\alpha_p \alpha_s}$$

In these expressions Δ_o and δ_o are, respectively, the primary and secondary beam deflections due to loading present at the initiation of ponding, $\alpha_p = C_p/(1 - C_p)$, $\alpha_s = C_s/(1 - C_s)$ and $\rho = \delta_o/\Delta_o = C_s/C_p$.

Using the above expressions for Δ_w and δ_w, the ratios Δ_w/Δ_o and δ_w/δ_o can be computed for any given combination of primary and secondary beam framing using, respectively, the computed value of parameters C_p and C_s defined in the LRFD Specification.

Even on the basis of unlimited elastic behavior, it is seen that the ponding deflections would become infinitely large unless

$$\left(\frac{C_p}{1 - C_p}\right)\left(\frac{C_s}{1 - C_s}\right) < \frac{4}{\pi}$$

Since elastic behavior is not unlimited, the effective bending strength available in each member to resist the stress caused by ponding action is restricted to the difference between the yield stress of the member and the stress f_o produced by the total load supported by it before consideration of ponding is included.

Note that elastic deflection is directly proportional to stress. The admissible amount of ponding in either the primary or critical (midspan) secondary member, in terms of the applicable ratio Δ_w/Δ_o or δ_w/δ_o, can be represented as $(0.8F_y - f_o)/f_o$. Substituting this expression for Δ_w/Δ_o and δ_w/δ_o, and combining with the foregoing expressions for Δ_w and δ_w, the relationship between critical values for C_p and C_s and the available elastic bending strength to resist ponding is obtained. The curves presented in Figs. C-K2.1 and C-K2.2 are based upon this relationship. They constitute a design aid for use when a more exact determination of required flat roof framing stiffness is needed than given by the LRFD Specification provision that $C_p + 0.9C_s \leq 0.25$.

Given any combination of primary and secondary framing, the stress index is computed as

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_p \text{ for the primary member}$$

$$U_s = \left(\frac{0.8F_y - f_o}{f_o}\right)_s \text{ for the secondary member}$$

where f_o, in each case, is the computed bending stress, ksi, in the member due to the supported loading, neglecting ponding effect. Depending upon geographic location, this loading should include such amount of snow as might also be present, although ponding failures have occurred more frequently during torrential summer rains when the rate of precipitation exceeded the rate of drainage runoff and the resulting hydraulic gradient over large roof areas caused substantial accumulation of water some distance from the eaves.

Given the size, spacing and span of a tentatively selected combination of primary and secondary beams, for example, one may enter Fig. C-K2.1 at the level of the computed stress index U_p, determined for the primary beam; move horizontally to the computed C_s value of the secondary beams; then move downward to the abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility constant read from this latter scale is more than the value of C_p computed for the given primary member; if not, a stiffer primary or secondary beam, or combination of both, is required.

If the roof framing consists of a series of equally-spaced wall-bearing beams, they would be considered as secondary members, supported on an infinitely stiff primary member. For this case, one would use Fig. C-K2.2. The limiting value of C_s would be determined by the intercept of a horizontal line representing the U_s value and the curve for $C_p = 0$.

The ponding deflection contributed by a metal deck is usually such a small part of the total ponding deflection of a roof panel that it is sufficient merely to limit its moment of inertia (per foot of width normal to its span) to 0.000025 times the fourth power of its span length, as provided in the LRFD Specification. However, the stability against ponding of a roof consisting of a metal roof deck of relatively slender depth-span ratio, spanning between beams supported directly on columns, may need

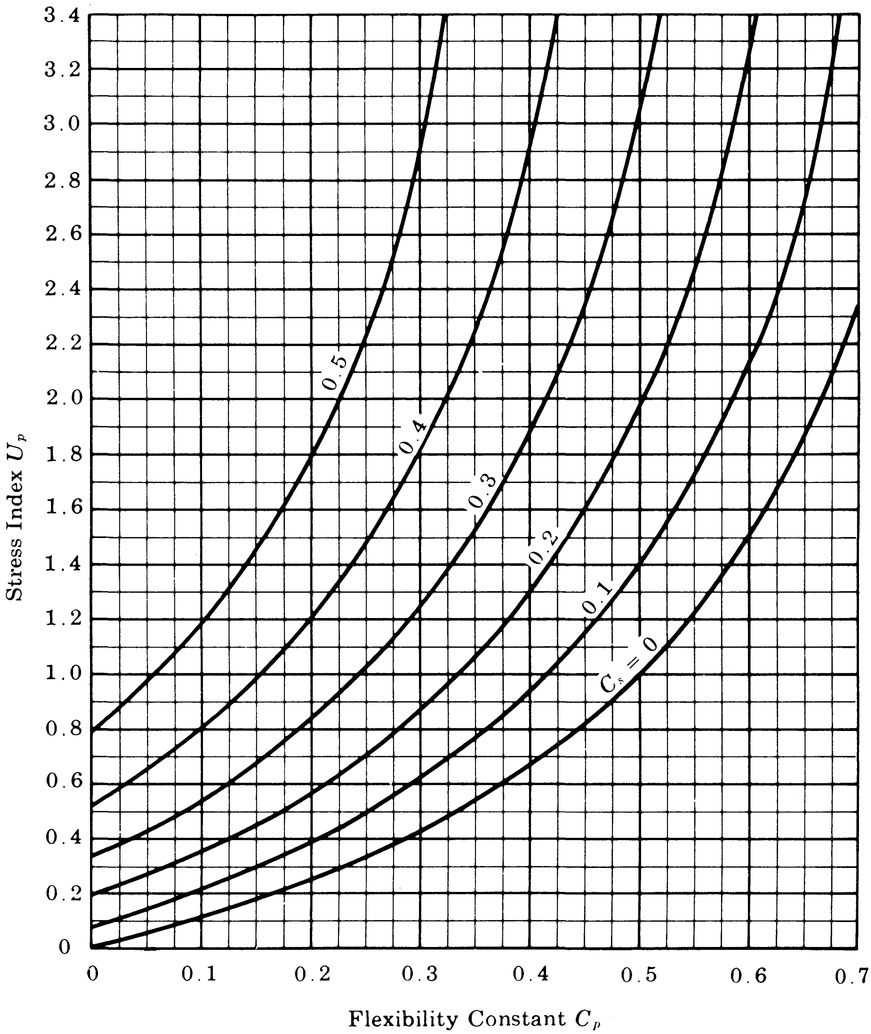
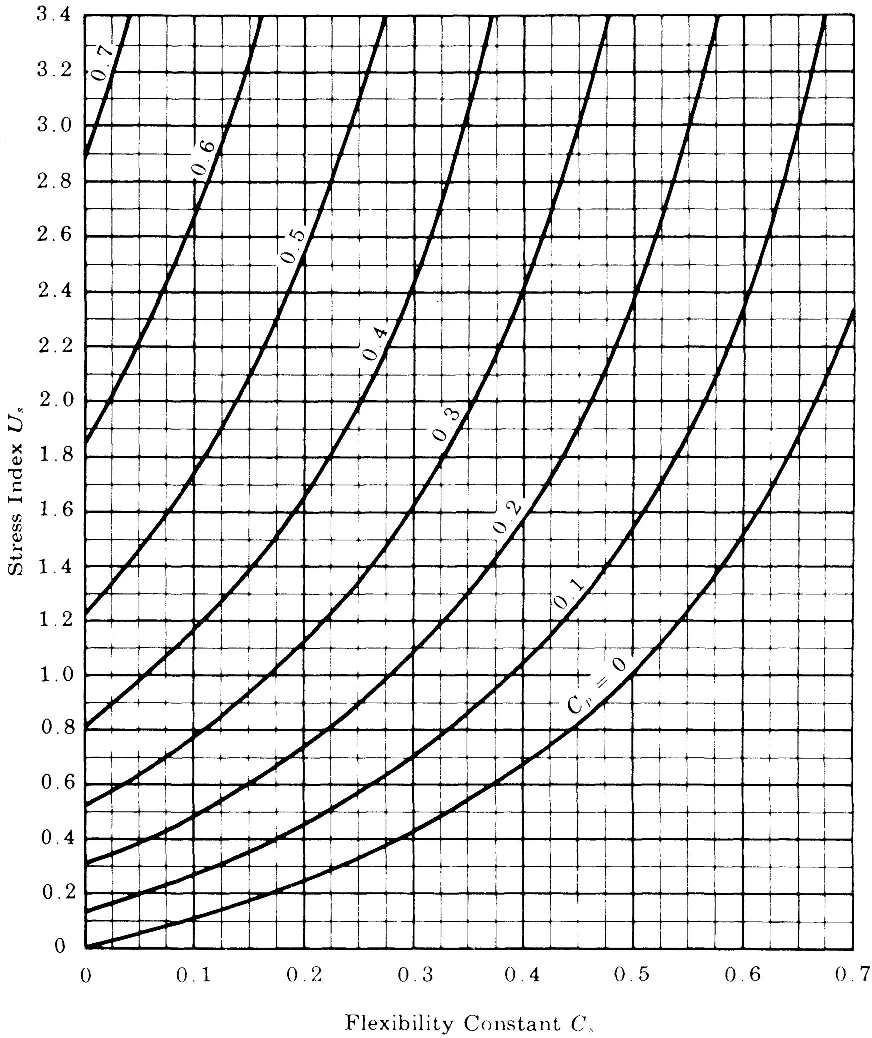


Figure C-K2.1

to be checked. This can be done using Fig. C-K2.1 or C-K2.2 with the following computed values:

U_p = stress index for the supporting beam
U_s = stress index for the roof deck
C_p = flexibility constant for the supporting beams
C_s = flexibility constant for one foot width of the roof deck (S = 1.0)

Since the shear rigidity of their web system is less than that of a solid plate, the moment of inertia of steel joists and trusses should be taken as somewhat less than that of their chords.



3.4
3.2
3.0
2.8
2.6
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
Stress Index U_s
0.7
0.6
0.5
0.4
0.3
0.2
0.1
$C_p = 0$
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
Flexibility Constant C_s

Figure C-K2.2

CHAPTER L.
SERVICEABILITY DESIGN CONSIDERATIONS

Serviceability criteria are formulated to insure that disruptions of the functional use or damage to the structure during its normal everyday use are rare. While malfunctions may not result in the collapse of a structure or in loss of life or injury, they can seriously impair the usefulness of the structure and lead to costly repairs. Concern with serviceability is important because of the increasing use of high-strength materials in design and the resulting relatively flexible structures.

There are essentially three types of structural behavior which may impair serviceability:

1. Excessive local damage (local yielding, buckling, slip or cracking) that may require excessive maintenance or lead to corrosion.
2. Excessive deflection or rotation that may affect the appearance, function or drainage of the structure, or may cause damage to nonstructural components and their attachments.
3. Excessive vibrations induced by wind or transient live loads which affect the comfort of occupants of the structure or the operation of mechanical equipment.

In Part 1 of the 1978 AISC Specification,[1] the problem of local damage is taken care of by the factor of safety built into the allowable stress, while excessive deflection and vibration are controlled, either directly or indirectly, by specifying limiting deflections, lateral drifts and maximum span-depth ratios. In the past, these rules have led to satisfactory structural performance, with perhaps the exception of large open floor areas without partitions. In LRFD the serviceability checks should consider the appropriate loads, the response of the structure and the reaction of the occupants to the structural response.

Examples of loads that may require consideration in serviceability checking include permanent live loads, wind and earthquake; effects of human activities such as walking, dancing, etc.; temperature fluctuations; and vibrations induced by traffic near the building or by the operation of mechanical equipment within the building.

Serviceability checks are concerned with adequate performance under the appropriate load conditions. The response of the structure can usually be assumed to be elastic. However, some structural elements may have to be examined with respect to their long-term behavior under load.

It is difficult to specify limiting values of structural performance based on serviceability considerations because these depend to a great extent on the type of structure, its intended use and subjective physiological reaction. For example, acceptable structural motion in a hospital clearly would be much less than in an ordinary industrial building. It should be noted that humans perceive levels of structural motion that are far less than motions that would cause any structural damage. Serviceability limits must be determined through careful consideration by the designer and client.

L1. CAMBER

The engineer should consider camber when deflections at the appropriate load level present a serviceability problem.

L2. EXPANSION AND CONTRACTION

As in the case of deflections, the satisfactory control of expansion cannot be reduced to a few simple rules, but must depend largely upon the good judgment of qualified engineers.

The problem is more serious in buildings with masonry walls than with prefabricated units. Complete divorcement of the framing, at widely spaced expansion joints, is generally more satisfactory than more frequently located devices dependent upon the sliding of parts in bearing, and usually less expensive than rocker or roller expansion bearings.

Creep and shrinkage of concrete and yielding of steel are among the causes, other than temperature, for dimensional changes.

L3. DEFLECTIONS, VIBRATION AND DRIFT

1. Deflections

Excessive transverse deflections or lateral drift may lead to permanent damage to building elements, separation of cladding or loss of weathertightness, damaging transfer of load to non-load-supporting elements, disruption of operation of building service systems, objectionable changes in appearance of portions of the buildings and discomfort to occupants.

The LRFD Specification does not provide specific limiting deflections for individual members or structural assemblies. Such limits would depend on the function of the structure.[36,77,78] Provisions that limit deflections to a percentage of span may not be adequate for certain long-span floor systems; a limit on maximum deflection that is independent of span length may also be necessary to minimize the possibility of damage to adjoining or connecting nonstructural elements.

2. Vibration

The increasing use of high-strength materials and efficient structural schemes leads to longer spans and more flexible floor systems. Even though the use of a deflection limit related to span length generally precluded vibration problems in the past, some floor systems may require explicit consideration of the dynamic, as well as the static, characteristics of the floor system.

The dynamic response of structures or structural assemblies may be difficult to analyze because of difficulties in defining the actual mass, stiffness and damping characteristics. Moreover, different load sources cause varying responses. For example, a steel beam-concrete slab floor system may respond to live loading as a non-composite system, but to transient excitation from human activity as an orthotropic composite plate. Nonstructural partitions, cladding and built-in furniture significantly increase the stiffness and damping of the structure and frequently eliminate potential vibration problems. The damping can also depend on the amplitude of excitation.

The general objective in minimizing problems associated with excessive structural motion is to limit accelerations, velocities and displacements to levels that would not be disturbing to the building occupants. Generally, occupants of a building find sustained vibrations more objectionable than transient vibrations.

The levels of peak acceleration that people find annoying depend on frequency of response. Thresholds of annoyance for transient vibrations are somewhat higher and depend on the amount of damping in the floor system. These levels depend on the individual and his activity at the time of excitation.[36, 48, 77, 78, 103, 104]

The most effective way to reduce effects of continuous vibrations is through vibration isolation devices. Care should be taken to avoid resonance, where the frequency of steady-state excitation is close to the fundamental frequency of the system. Transient vibrations are reduced most effectively by increasing the damping in the structural assembly. Mechanical equipment which can produce objectionable vibrations in any portion of a structure should be adequately isolated to reduce the transmission of such vibrations to critical elements of the structure.

3. Drift

The LRFD Specification does not provide specific limiting values for lateral drift. If a drift analysis is desired, the stiffening effect of non-load-supporting elements such as partitions and infilled walls may be included in the analysis of drift if substantiating information regarding their effect on response is available.

Some irrecoverable inelastic deformations may occur at given load levels in certain types of construction, e.g., flexural members where the shape factor Z/S is in excess of 1.5. The effect of such deformations may be negligible or serious, depending on the function of the structure, and should be considered by the designer on a case by case basis.

The deformation limits should apply to structural assemblies as a whole. Reasonable tolerance should also be provided for creep. Where load cycling occurs, consideration should be given to the possibility of increases in residual deformation that may lead to incremental failure.

L5. CORROSION

Steel members may deteriorate in particular service environments. This deterioration may appear either in external corrosion, which would be visible upon inspection, or in undetected changes in the material that would reduce its load-carrying capacity. The designer should recognize these problems by either factoring a specific amount of damage tolerance into his design or providing adequate protection systems (e.g., coatings, cathodic protection) and/or planned maintenance programs so that such problems do not occur.

CHAPTER M. FABRICATION, ERECTION AND QUALITY CONTROL

M2. FABRICATION

1. Cambering, Curving and Straightening

The use of heat for straightening or cambering members is permitted for A514 steel, as it is for other steels. However, the maximum temperature permitted is 1,100°F for A514 steel, as contrasted with 1,200°F for other steels.

The cambering of flexural members when required by contract documents, is accomplished in various ways. In the case of trusses and girders, the desired curvature can be built in during assembly of the component parts. Within limits, rolled beams can be cold-cambered at the producing mill.

The local application of heat has come into common use as a means of straightening or cambering beams and girders. The method depends upon an ultimate shortening of the heat-affected zones. A number of such zones, on the side of the member that would be subject to compression during cold-cambering or "gagging," are heated enough to be "upset" by the restraint provided by surrounding unheated areas. Shortening takes place upon cooling.

While the final curvature or camber can be controlled by these methods, it must be realized that some deviation, due to workmanship error and permanent change due to handling, is inevitable.

5. Bolted Construction

In the past, it has been required to tighten to a specified tension all ASTM A325 and A490 bolts in both slip-critical and bearing-type connections. The requirement was changed in 1985 to permit some bearing-type connections to be only tightened to a snug-tight condition.

To qualify as a snug-tight bearing connection, the bolts cannot be subject to tension loads, slip can be permitted and loosening or fatigue due to vibration or load fluctuations are not design considerations.

It is suggested that snug-tight bearing-type connections be used in applications when A307 bolts would be permitted. The 1978 Specification Sect. 1.15.12 serves as a guide to these applications.

This section provides rules for the use of oversized and slotted holes paralleling the provisions which have been in the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*[68] since 1972, extended to include A307 bolts which are outside the scope of the high-strength bolt specifications.

M3. SHOP PAINTING

The surface condition of steel framing disclosed by the demolition of long-standing buildings has been found to be unchanged from the time of its erection, except at isolated spots where leakage may have occurred. Even in the presence of leakage, the shop coat is of minor influence.[69]

The LRFD Specification does not define the type of paint to be used when a shop coat is required. Conditions of exposure and individual preference with regard to finish paint are factors which bear on the selection of the proper primer. Hence, a single formulation would not suffice. For a comprehensive treatment of the subject, see Ref. 70.

M4. ERECTION

4. Fit of Column Compression Joints

Tests at the University of California-Berkeley[71] on spliced full-size columns with joints that had been intentionally milled out-of-square, relative to either strong or weak axis, demonstrated that their load-carrying capacity was the same as that for a similar unspliced column. In the tests, gaps of 1/16-in. were not shimmed; gaps of 1/4-in. were shimmed with non-tapered mild steel shims. Minimum size partial-penetration welds were used in all tests. No tests were performed on specimens with gaps greater than 1/4-in.

APPENDIX E. COLUMNS AND OTHER COMPRESSION MEMBERS

E3. FLEXURAL-TORSIONAL BUCKLING

A possible mode of buckling of columns is torsional buckling for symmetric shapes and flexural-torsional buckling for unsymmetric shapes. These modes are usually not considered in design for the hot-rolled columns because they generally do not govern, or the critical load differs very little from the weak-axis planar buckling load. Such a buckling mode may, however, control the capacity of columns made from plate elements which are relatively thin, and for unsymmetric columns. Formulas for determining the flexural-torsional elastic buckling loads of such columns are derived in texts on structural stability (Refs. 49, 50, 72, for example). Since these equations for flexural-torsional buckling apply only to elastic buckling, they must be modified for inelastic buckling when $F_{cr} > 0.5\ F_y$. This is accomplished through the use of the equivalent slenderness factor $\lambda_e = \sqrt{F_y / F_e}$.

APPENDIX F. BEAMS AND OTHER FLEXURAL MEMBERS

F1. DESIGN FOR FLEXURE

7. Nominal Flexural Strength of Other Sections

Three limit states must be investigated to determine the moment capacity of flexural members: lateral-torsional buckling (LTB), local buckling of the compression flange (FLB) and local buckling of the web (WLB). These limit states depend, respectively, on the beam slenderness ratio L_b / r_y, the width-thickness ratio b/t of the compression flange and the width-thickness ratio h_c / t_w of the web. For convenience, all three measures of slenderness are denoted by λ.

Variations in M_n with L_b are shown in Fig. C-F1.1. The discussion of plastic, inelastic and elastic buckling in Commentary Sect. F1 with reference to lateral-torsional buckling applies here except for an important difference in the significance of λ_p for lateral-torsional buckling and local buckling. Values of λ_p for FLB and WLB produce a compact section with a rotation capacity of about four (after reaching M_p) before the onset of local buckling, and therefore meet the requirements for plastic analysis of load effects (Commentary Sect. B5). On the other hand, values of λ_p for LTB do not allow plastic analysis because they do not provide rotation capacity beyond that needed to develop M_p. Instead $L_b \leq L_{pd}$ (Sect. F1.1) must be satisfied.

Analyses to include restraint effects of adjoining elements are discussed in Ref. 11. Analysis of the lateral stability of members with shapes not covered in this appendix must be performed according to the available literature.[11]

See the Commentary for Sect. B5 for the discussion of the equation regarding the bending capacity of circular sections.

F4. WEB-TAPERED MEMBERS

1. General Requirements

The provision contained in Appendix F4 covers only those aspects of the design of tapered members that are unique to tapered members. For other criteria of design not specifically covered in Appendix F4, see the appropriate portions of this Specification and Commentary.

The design of wide-flange columns with a single web taper and constant flanges follows the same procedure as for uniform columns according to Sect. E2, except the column slenderness parameter λ_c for major axis buckling is determined for a slenderness ratio $K_\gamma L / r_{ox}$, and for minor axis buckling for KL / r_{oy}, where K_γ is an effective length factor for tapered members, K is the effective length factor for prismatic members and r_{ox} and r_{oy} are the radii of gyration about the x and the y axes, respectively, taken at the smaller end of the tapered member.

For stepped columns or columns with other than a single web taper, the elastic critical stress is determined by analysis or from data in reference texts or research reports (Chaps. 11 and 13 in Ref. 49 and Refs. 50 and 15), and then the same procedure of using λ_{eff} is utilized in calculating the factored resistance.

This same approach is recommended for open section built-up columns (columns with perforated cover plates, lacing and battens) where the elastic critical buckling stress determination must include a reduction for the effect of shear. Methods for calculating the elastic buckling strength of such columns are given in Chap. 12 of the SSRC *Guide to Design Criteria for Metal Compression Members*[11] and in Refs. 49 and 50.

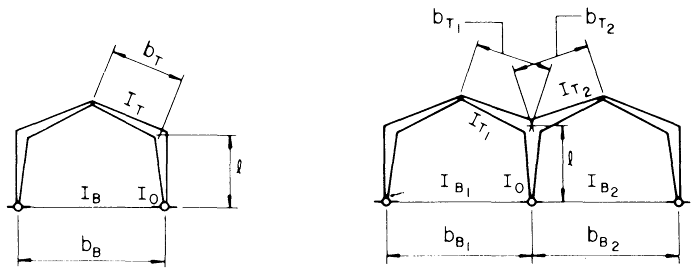
3. Design Compressive Strength

The approach in formulating $F_{a\gamma}$ of tapered columns is based on the concept that the critical stress for an axially loaded tapered column is equal to that of a prismatic column of different length, but of the same cross section as the smaller end of the tapered column. This has resulted in an equivalent effective length factor K_γ for a tapered member subjected to axial compression.[73] This factor, which is used to determine the value of S in Formulas A-F4-2 and E2-3, can be determined accurately for a symmetrical rectangular rigid frame composed of prismatic beams and tapered columns.

With modifying assumptions, such a frame can be used as a mathematical model to determine with sufficient accuracy the influence of the stiffness $\Sigma(I/b)_g$ of beams and rafters which afford restraint at the ends of a tapered column in other cases such as those shown in Fig. C-A-F4.1. From Formulas A-F4-2 and E2-3, the critical load P_{cr} can be expressed as $\pi^2 EI_o/(K\gamma 1)^2$. The value of K_γ can be obtained by interpolation, using the appropriate chart from Ref. 73 and restraint modifiers G_T and G_B. In each of these modifiers the tapered column, treated as a prismatic member having a moment of inertia I_o, computed at the smaller end, and its actual length ℓ, is assigned the stiffness I_o / ℓ, which is then divided by the stiffness of the restraining members at the end of the tapered column under consideration.

4. Design Flexural Strength

The development of the design bending stress for tapered beams follows closely with that for prismatic beams. The basic concept is to replace a tapered beam by an equivalent prismatic beam with a different length, but with a cross section identical to



$$G_T = \frac{b_T I_O}{\ell I_T} \qquad G_T = \frac{I_O}{\ell} \sum \frac{b_T}{I_T}$$

$$G_B = \frac{b_B I_O}{\ell I_B} \qquad G_B = \frac{I_O}{\ell} \sum \frac{b_B}{I_B}$$

Figure C-A-F4.1

that of the smaller end of the tapered beam.[73] This has led to the modified length factors h_s and h_w in Formulas A-F4-6 and A-F4-7.

Formulas A-F4-6 and A-F4-7 are based on total resistance to lateral buckling, using both St. Venant and warping resistance. The factor B modifies the basic $F_{b\gamma}$ to account for moment gradient and lateral restraint offered by adjacent segments. For members which are continuous past lateral supports, categories 1, 2 and 3 of Sect. D3 usually apply; however, it is to be noted that they apply only when the axial force is small and adjacent unbraced segments are approximately equal in length. For a single member, or segments which do not fall into category 1, 2, 3 or 4, the recommended value of B is unity. The value of B should also be taken as unity when computing the value of $F_{b\gamma}$ to obtain M_n to be used in Formulas H1-1 through H1-3, since the effect of moment gradient is provided for by the factor C_m. The background material is given in WRC Bulletin No. 192.[74]

APPENDIX G.
PLATE GIRDERS

Appendix G is taken from AISI Bulletin 27.[9] Comparable provisions are included in Sect. 1.10 of the 1978 AISC Specification.[1] The provisions have been moved to an appendix as they are seldom used and produce designs which are often less economical than plate girders designed without tension field action.

The web slenderness ratio $h_c/t_w = 970/\sqrt{F_{yf}}$ that distinguishes plate girders from beams is written in terms of the flange yield stress, because for hybrid girders inelastic buckling of the web due to bending depends on the flange strain.

APPENDIX H. MEMBERS UNDER TORSION AND COMBINED FORCES

H3. ALTERNATE INTERACTION EQUATIONS FOR MEMBERS UNDER COMBINED STRESS

In the case of members not subject to local column or flexural buckling, i.e., $L_b < L_{pd}$, the use of somewhat more liberal interaction Formulas A-H3-5 and A-H3-6 are acceptable as an alternate when the flexure is about one axis only. Formula A-H3-5 is retained from Part 2 of the 1978 AISC Specification.[1]

The alternate interaction Formulas A-H3-1 and A-H3-2 for biaxially loaded H and wide flange column shapes were taken from Refs. 11 and 23.

APPENDIX K. STRENGTH DESIGN CONSIDERATIONS

K4. FATIGUE

Because most members in building frames are not subject to a large enough number of cycles of full design stress application to require design for fatigue, the provisions covering such designs have been placed in Appendix K4.

When fatigue is a design consideration, its severity is most significantly affected by the number of load applications, the magnitude of the stress range and the severity of the stress concentrations associated with the particular details. These factors are not encountered in normal building designs; however, when encountered and when fatigue is of concern, all provisions of Appendix K4 must be satisfied.

Members or connections subject to less than 20,000 cycles of loading will not involve a fatigue condition, except in the case of repeated loading involving large ranges of stress. For such conditions, the admissible range of stress can conservatively be taken as one and one-half times the applicable value given in Table A-K4.3 for "Loading Condition 1."

Fluctuation in stress which does not involve tensile stress does not cause crack propagation and is not considered to be a fatigue situation. On the other hand, in elements of members subject solely to calculated compression stress, fatigue cracks may initiate in regions of high tensile residual stress. In such situations, the cracks generally do not propagate beyond the region of the residual tensile stress, because the residual stress is relieved by the crack. For this reason stress ranges that are completely in compression are not included in the column headed by "Kind of Stress" in Table A-K4.2 of Appendix K4. This is also true of comparable tables of the current AASHTO and AREA specifications.

When fabrication details involving more than one category occur at the same location in a member, the stress range at that location must be limited to that of the most restrictive category. By locating notch-producing fabrication details in regions subject to a small range of stress, the need for a member larger than required by static loading will often be eliminated.

Extensive test programs[66,67] using full size specimens, substantiated by theoretical stress analysis, have confirmed the following general conclusions:

1. Stress range and notch severity are the dominant stress variables for welded details and beams.
2. Other variables such as minimum stress, mean stress and maximum stress are not significant for design purposes.
3. Structural steels with yield points of 36 to 100 ksi do not exhibit significantly different fatigue strength for given welded details fabricated in the same manner.

Allowable stress ranges can be read directly from Table A-K4.3 for a particular category and loading condition. The values are based on recent research.[100]

Provisions for bolts subjected to tension are given in Table A-K4.4. Tests have uncovered dramatic differences in fatigue life, not completely predictable from the various published formulas for estimating the actual magnitude of prying force.[43] To limit the uncertainties regarding prying action on the fatigue behavior of these bolts, the tensile stresses given in Table J3.2 are approved for use under extended cyclic loading only if the prying force, included in the design tensile force, is small. When this cannot be assured, the design tensile stress is drastically reduced to cover any conceivable prying effect.

The use of other types of mechanical fasteners to resist applied cyclic loading in tension is not recommended. Lacking a high degree of assured pretension, the range of stress is generally too great to resist such loading for long.

However, all types of mechanical fasteners survive unharmed when subject to cyclic shear stresses sufficient to fracture the connected parts, which is provided for elsewhere in Appendix K4.

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Notes

GLOSSARY

Alignment chart for columns. A nomograph for determining the effective length factor K for some types of columns

Amplification factor. A multiplier of the value of moment or deflection in the unbraced length of an axially loaded member to reflect the secondary values generated by the eccentricity of the applied axial load within the member

Aspect ratio. In any rectangular configuration, the ratio of the lengths of the sides

Batten plate. A plate element used to join two parallel components of a built-up column, girder or strut rigidly connected to the parallel components and designed to transmit shear between them

Beam. A structural member whose primary function is to carry loads transverse to its longitudinal axis

Beam-column. A structural member whose primary function is to carry loads both transverse and parallel to its longitudinal axis

Bent. A plane framework of beam or truss members which support loads and the columns which support these members

Biaxial bending. Simultaneous bending of a member about two perpendicular axes

Bifurcation. The phenomenon whereby a perfectly straight member under compression may either assume a deflected position or may remain undeflected, or a beam under flexure may either deflect and twist out of plane or remain in its in-plane deflected position

Braced frame. A frame in which the resistance to lateral load or frame instability is primarily provided by a diagonal, a K-brace or other auxiliary system of bracing

Brittle fracture. Abrupt cleavage with little or no prior ductile deformation

Buckling load. The load at which a perfectly straight member under compression assumes a deflected position

Built-up member. A member made of structural metal elements that are welded, bolted or riveted together

Cladding. The exterior covering of the structural components of a building

Cold-formed members. Structural members formed from steel without the application of heat

Column. A structural member whose primary function is to carry loads parallel to its longitudinal axis

Column curve. A curve expressing the relationship between axial column strength and slenderness ratio

Combined mechanism. A mechanism determined by plastic analysis procedure which combines elementary beam, panel and joint mechanisms

Compact section. Compact sections are capable of developing a fully plastic stress distribution and possess rotation capacity of approximately 3 before the onset of local buckling

Composite beam. A steel beam structurally connected to a concrete slab so that the beam and slab respond to loads as a unit. See also concrete-encased beam

Composite column. A steel column fabricated from rolled or built-up steel shapes and encased in structural concrete or fabricated from steel pipe or tubing and filled with structural concrete

Concrete-encased beam. A beam totally encased in concrete cast integrally with the slab

Connection. Combination of joints used to transmit forces between two or mo members. Categorized by the type and amount of force transferred (momer shear, end reaction). See also splices

Critical load. The load at which bifurcation occurs as determined by a theoretic stability analysis

Curvature. The rotation per unit length due to bending

Design documents. See structural design documents

Design strength. Resistance (force, moment, stress, as appropriate) provided t element or connection; the product of the nominal strength and the resistan factor

Diagonal bracing. Inclined structural members carrying primarily axial load en ployed to enable a structural frame to act as a truss to resist horizontal loac

Diaphragm. Floor slab, metal wall or roof panel possessing a large in-plane she stiffness and strength adequate to transmit horizontal forces to resisting sy tems

Diaphragm action. The in-plane action of a floor system (also roofs and walls) suc that all columns framing into the floor from above and below are maintained i their same position relative to each other

Double curvature. A bending condition in which end moments on a member caus the member to assume an S-shape

Drift. Lateral deflection of a building

Drift index. The ratio of lateral deflection to the height of the building

Ductility factor. The ratio of the total deformation at maximum load to th elastic-limit deformation

Effective length. The equivalent length *KL* used in compression formulas an determined by a bifurcation analysis

Effective length factor K. The ratio between the effective length and the unbrace length of the member measured between the centers of gravity of the bracin members

Effective moment of inertia. The moment of inertia of the cross section of a member that remains elastic when partial plastification of the cross section takes place, usually under the combination of residual stress and applied stress. Also, the moment of inertia based on effective widths of elements that buckle locally. Also, the moment of inertia used in the design of partially composite members

Effective stiffness. The stiffness of a member computed using the effective moment of inertia of its cross section

Effective width. The reduced width of a plate or slab which, with an assumed uniform stress distribution, produces the same effect on the behavior of a structural member as the actual plate width with its nonuniform stress distribution

Elastic analysis. Determination of load effects (force, moment, stress, as appropriate) on members and connections based on the assumption that material deformation disappears on removal of the force that produced it

Elastic-perfectly plastic. A material which has an idealized stress-strain curve that varies linearly from the point of zero strain and zero stress up to the yield point of the material, and then increases in strain at the value of the yield stress without any further increases in stress

Embedment. A steel component cast in a concrete structure which is used to transmit externally applied loads to the concrete structure by means of bearing, shear, bond, friction or any combination thereof. The embedment may be fabricated of structural-steel plates, shapes, bars, bolts, pipe, studs, concrete reinforcing bars, shear connectors or any combination thereof

Encased steel structure. A steel-framed structure in which all of the individual frame members are completely encased in cast-in-place concrete

Euler formula. The mathematical relationship expressing the value of the Euler load in terms of the modulus of elasticity, the moment of inertia of the cross section and the length of a column

Euler load. The critical load of a perfectly straight centrally loaded pin-ended column

Eyebar. A particular type of pin-connected tension member of uniform thickness with forged or flame cut head of greater width than the body proportioned to provide approximately equal strength in the head and body

Factored load. The product of the nominal load and a load factor

Fastener. Generic term for welds, bolts, rivets or other connecting device

Fatigue. A fracture phenomenon resulting from a fluctuating stress cycle

First-order analysis. Analysis based on first-order deformations in which equilibrium conditions are formulated on the undeformed structure

Flame-cut plate. A plate in which the longitudinal edges have been prepared by oxygen cutting from a larger plate

Flat width. For a rectangular tube, the nominal width minus twice the outside corner radius. In absence of knowledge of the corner radius, the flat width may be taken as the total section width minus three times the thickness

Flexible connection. A connection permitting a portion, but not all, of the simple beam rotation of a member end

Floor system. The system of structural components separating the stories of a building

Force. Resultant of distribution of stress over a prescribed area. A reaction that develops in a member as a result of load (formerly called total stress or stress). Generic term signifying axial loads, bending moment, torques and shears

Fracture toughness. Measurement of the ability to absorb energy without fracture. Generally determined by impact loading of specimens containing a notch having a prescribed geometry

Frame buckling. A condition under which bifurcation may occur in a frame

Frame instability. A condition under which a frame deforms with increasing lateral deflection under a system of increasing applied monotonic loads until a maximum value of the load called the stability limit is reached, after which the frame will continue to deflect without further increase in load

Fully composite beam. A composite beam with sufficient shear connectors to develop the full flexural strength of the composite section

High-cycle fatigue. Failure resulting from more than 20,000 applications of cyclic stress

Hybrid beam. A fabricated steel beam composed of flanges with a greater yield strength than that of the web. Whenever the maximum flange stress is less than or equal to the web yield stress the girder is considered homogeneous

Hysteresis loop. A plot of force versus displacement of a structure or member subjected to reversed, repeated load into the inelastic range, in which the path followed during release and removal of load is different from the path for the addition of load over the same range of displacement

Inclusions. Nonmetallic material entrapped in otherwise sound metal

Incomplete fusion. Lack of union by melting of filler and base metal over enti prescribed area

Inelastic action. Material deformation that does not disappear on removal of th force that produced it

Instability. A condition reached in the loading of an element or structure in whic continued deformation results in a decrease of load-resisting capacity

Joint. Area where two or more ends, surfaces, or edges are attached. Categorize by type of fastener or weld used and method of force transfer

K-bracing. A system of struts used in a braced frame in which the pattern of th struts resembles the letter *K*, either normal or on its side

Lamellar tearing. Separation in highly restrained base metal caused by through thickness strains induced by shrinkage of adjacent weld metal

Lateral bracing member. A member utilized individually or as a component of lateral bracing system to prevent buckling of members or elements and/or t resist lateral loads

Lateral (or lateral-torsional) buckling. Buckling of a member involving latera deflection and twist

Limit state. A condition in which a structure or component becomes unfit fo service and is judged either to be no longer useful for its intended functior (*serviceability limit state*) or to be unsafe (*strength limit state*)

Limit states. Limits of structural usefulness, such as brittle fracture, plastic col- lapse, excessive deformation, durability, fatigue, instability and serviceability

Load factor. A factor that accounts for unavoidable deviations of the actual load from the nominal value and for uncertainties in the analysis that transforms the load into a load effect

Loads. Forces or other actions that arise on structural systems from the weight of all permanent construction, occupants and their possessions, environmental effects, differential settlement and restrained dimensional changes. *Permanent* loads are those loads in which variations in time are rare or of small magnitude. All other loads are *variable* loads. See *nominal loads*.

LRFD (Load and Resistance Factor Design). A method of proportioning structural components (members, connectors, connecting elements and assemblages) such that no applicable limit state is exceeded when the structure is subjected to all appropriate load combinations

Local buckling. The buckling of a compression element which may precipitate the failure of the whole member

Low-cycle fatigue. Fracture resulting from a relatively high stress range resulting in a relatively small number of cycles to failure

Lower bound load. A load computed on the basis of an assumed equilibrium moment diagram in which the moments are not greater then M_p that is less than or at best equal to the true ultimate load

Mechanism. An articulated system able to deform without an increase in load, used in the special sense that the linkage may include real hinges or plastic hinges, or both

Mechanism method. A method of plastic analysis in which equilibrium between external forces and internal plastic hinges is calculated on the basis of an assumed mechanism. The failure load so determined is an upper bound

Nominal loads. The magnitudes of the loads specified by the applicable code

Nominal strength. The capacity of a structure or component to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions

Noncompact section. Noncompact sections can develop the yield stress in compression elements before local buckling occurs, but will not resist inelastic local buckling at strain levels required for a fully plastic stress distribution

P-Delta effect. Secondary effect of column axial loads and lateral deflection on the moments in members

Panel zone. The zone in a beam-to-column connection that transmits moment by a shear panel

Partially composite beam. A composite beam for which the shear strength of shear connectors governs the flexural strength

Plane frame. A structural system assumed for the purpose of analysis and design to be two-dimensional

Plastic analysis. Determination of load effects (force, moment, stress, as appropriate) on members and connections based on the assumption of rigid-plastic behavior, i.e., that equilibrium is satisfied throughout the structure and yield is not exceeded anywhere. Second order effects may need to be considered

Plastic design section. The cross section of a member which can maintain a full plastic moment through large rotations so that a mechanism can develop; the section suitable for plastic design

Plastic hinge. A yielded zone which forms in a structural member when the plastic moment is attained. The beam is assumed to rotate as if hinged, except that it is restrained by the plastic moment M_p

Plastic-limit load. The maximum load that is attained when a sufficient number of yield zones have formed to permit the structure to deform plastically without further increase in load. It is the largest load a structure will support, when perfect plasticity is assumed and when such factors as instability, second-order effects, strain hardening and fracture are neglected

Plastic mechanism. See mechanism

Plastic modulus. The section modulus of resistance to bending of a completely yielded cross section. It is the combined static moment about the neutral axis of the cross-sectional areas above and below that axis

Plastic moment. The resisting moment of a fully yielded cross section

Plastic strain. The difference between total strain and elastic strain

Plastic zone. The yielded region of a member

Plastification. The process of successive yielding of fibers in the cross section of a member as bending moment is increased

Plate girder. A built-up structural beam

Post-buckling strength. The load that can be carried by an element, member or frame after buckling

Redistribution of moment. A process which results in the successive formation of plastic hinges so that less highly stressed portions of a structure may carry increased moments

Required strength. Load effect (force, moment, stress, as appropriate) acting on element or connection determined by structural analysis from the factored loads (using most appropriate critical load combinations)

Residual stress. The stresses that remain in an unloaded member after it has b formed into a finished product. (Examples of such stresses include, but are limited to, those induced by cold bending, cooling after rolling, or weldin

Resistance. The capacity of a structure or component to resist the effects of loa It is determined by computations using specified material strengths, dimensi and formulas derived from accepted principles of structural mechanics, or field tests or laboratory tests of scaled models, allowing for modeling effects a differences between laboratory and field conditions. Resistance is a gene term that includes both strength and serviceability limit states

Resistance factor. A factor that accounts for unavoidable deviations of the act strength from the nominal value and the manner and consequences of failu

Rigid frame. A structure in which connections maintain the angular relations between beam and column members under load

Root of the flange. Location on the web of the corner radius termination point the toe of the flange-to-web weld. Measured as the k distance from the far si of the flange

Rotation capacity. The incremental angular rotation that a given shape can acce prior to local failure defined as $R = (\theta_u / \theta_p) - 1$ where θ_u is the overall rotati attained at the factored load state and θ_p is the idealized rotation correspondi to elastic theory applied to the case of $M = M_p$

St. Venant torsion. That portion of the torsion in a member that induces only she stresses in the member

Second-order analysis. Analysis based on second-order deformations, in whi equilibrium conditions are formulated on the deformed structure

Service load. Load expected to be supported by the structure under normal usag often taken as the nominal load

Serviceability limit state. Limiting condition affecting the ability of a structure t preserve its appearance, maintainability, durability or the comfort of its occu pants or function of machinery under normal usage

Shape factor. The ratio of the plastic moment to the yield moment, or the ratio c the plastic modulus to the section modulus for a cross section

Shear-friction. Friction between the embedment and the concrete that transmit shear loads. The relative displacement in the plane of the shear load is consid ered to be resisted by shear-friction anchors located perpendicular to the plan of the shear load.

Shear lugs. Plates, welded studs, bolts and other steel shapes that are embedded i the concrete and located transverse to the direction of the shear force and tha transmit shear loads, introduced into the concrete by local bearing at the shea lug-concrete interface

Shear wall. A wall that in its own plane resists shear forces resulting from applied wind, earthquake or other transverse loads or provides frame stability. Also called a structural wall

Sidesway. The lateral movement of a structure under the action of lateral loads, unsymmetrical vertical loads or unsymmetrical properties of the structure

Sidesway buckling. The buckling mode of a multistory frame precipitated by the relative lateral displacements of joints, leading to failure by sidesway of the frame

Simple plastic theory. See plastic design

Single curvature. A deformed shape of a member having one smooth continuous arc, as opposed to double curvature which contains a reversal

Slender section. The cross section of a member which will experience local buckling in the elastic range

Slenderness ratio. The ratio of the effective length of a column to the radius of gyration of the column, both with respect to the same axis of bending

Slip-critical joint. A bolted joint in which the slip resistance of the connection is required

Space frame. A three-dimensional structural framework (as contrasted to a plane frame)

Splice. The connection between two structural elements joined at their ends to form a single, longer element

Stability-limit load. Maximum (theoretical) load a structure can support when second-order instability effects are included

Stepped column. A column with changes from one cross section to another occurring at abrupt points within the length of the column

Stiffener. A member, usually an angle or plate, attached to a plate or web of a beam or girder to distribute load, to transfer shear or to prevent buckling of the member to which it is attached

Stiffness. The resistance to deformation of a member or structure measured by the ratio of the applied force to the corresponding displacement

Story drift. The difference in horizontal deflection at the top and bottom of a story

Strain hardening. Phenomenon wherein ductile steel, after undergoing considerable deformation at or just above yield point, exhibits the capacity to resist substantially higher loading than that which caused initial yielding

Strain-hardening strain. For structural steels that have a flat (plastic) region in the stress-strain relationship, the value of the strain at the onset of strain hardening

Strength design. A method of proportioning structural members using load factors and resistance factors such that no applicable limit state is exceeded (also called load and resistance factor design)

Strength limit state. Limiting condition affecting the safety of the structure, in which the ultimate load-carrying capacity is reached

Stress. Force per unit area

Stress concentration. Localized stress considerably higher than average (even in uniformly loaded cross sections of uniform thickness) due to abrupt changes in geometry or localized loading

Strong axis. The major principal axis of a cross section

Structural design documents. Documents prepared by the designer (plans, design details and job specifications)

Structural system. An assemblage of load-carrying components which are joined together to provide regular interaction or interdependence

Stub column. A short compression-test specimen, long enough for use in measuring the stress-strain relationship for the complete cross section, but short enough to avoid buckling as a column in the elastic and plastic ranges

Subassemblage. A truncated portion of a structural frame

Supported frame. A frame which depends upon adjacent braced or unbraced frames for resistance to lateral load or frame instability. (This transfer of load is frequently provided by the floor or roof system through diaphragm action or by horizontal cross bracing in the roof.)

Tangent modulus. At any given stress level, the slope of the stress-strain curve of a material in the inelastic range as determined by the compression test of a small specimen under controlled conditions

Temporary structure. A general term for anything that is built or constructed (usually to carry construction loads) that will eventually be removed before or after completion of construction and does not become part of the permanent structural system

Tensile strength. The maximum tensile stress that a material is capable of sustaining

Tension field action. The behavior of a plate girder panel under shear force in which diagonal tensile stresses develop in the web and compressive forces develop in the transverse stiffeners in a manner analogous to a Pratt truss

Toe of the fillet. Termination point of fillet weld or of rolled section fillet

Torque-tension relationship. Term applied to the wrench torque required to produce specified pre-tension in high-strength bolts

Turn-of-nut method. Procedure whereby the specified pre-tension in high-strength bolts is controlled by rotation of the wrench a predetermined amount after the nut has been tightened to a snug fit

Unbraced frame. A frame in which the resistance to lateral load is provided by the bending resistance of frame members and their connections

Unbraced length. The distance between braced points of a member, measured between the centers of gravity of the bracing members

Undercut. A notch resulting from the melting and removal of base metal at the edge of a weld

Universal-mill plate. A plate in which the longitudinal edges have been formed by a rolling process during manufacture. Often abbreviated as UM plate

Upper bound load. A load computed on the basis of an assumed mechanism which will always be at best equal to or greater than the true ultimate load

Vertical bracing system. A system of shear walls, braced frames or both, extending through one or more floors of a building

Von Mises yield criterion. A theory which states that inelastic action at any point in a body under any combination of stresses begins only when the strain energy of distortion per unit volume absorbed at the point is equal to the strain energy of distortion absorbed per unit volume at any point in a simple tensile bar stressed to the elastic limit under a state of uniaxial stress. It is often called the maximum strain-energy-of-distortion theory. Accordingly, shear yield occurs at 0.58 times yield strength

Warping torsion. That portion of the total resistance to torsion that is provided by resistance to warping of the cross section

Weak axis. The minor principal axis of a cross section

Weathering steel. A type of high-strength, low-alloy steel which can be used in normal environments (not marine) and outdoor exposures without protective paint covering. This steel develops a tight adherent rust at a decreasing rate with respect to time

Web buckling. The buckling of a web plate

Web crippling. The local failure of a web plate in the immediate vicinity of a concentrated load or reaction

Working load. Also called service load. The actual load assumed to be acting on the structure

Yield moment. In a member subjected to bending, the moment at which an outer fiber first attains the yield stress

Yield plateau. The portion of the stress-strain curve for uniaxial tension or compression in which the stress remains essentially constant during a period of substantially increased strain

Yield point. The first stress in a material at which an increase in strain occurs without an increase in stress, the yield point less than the maximum attainable stress

Yield strength. The stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain. Deviation expressed in terms of strain

Yield stress. Yield point, yield strength or yield stress level as defined

Yield-stress level. The average stress during yielding in the plastic range, the stress determined in a tension test when the strain reaches 0.005 in. per in.