Specification
for the
Design,
Fabrication
and Erection
of
Structural
Steel for
Buildings
Effective November 1, 1978

with Commentary

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

Preface

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 Specification is to cover the many everyday design criteria in 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 AISC 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. Each Specification change is based upon essentially unanimous affirmative action on the part of the full Committee.

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 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.

As used throughout the Specification, the term *structural steel* refers exclusively to those items enumerated in Section 2 of the AISC *Code of Standard Practice for Steel Buildings and Bridges,* and 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.

The principal changes incorporated in this edition of the Specification include:

- New rules for the design stresses for tension members.
- New design stresses for mechanical fasteners.
- Provisions for the design of composite beams using formed steel deck.
- Improved provisions for the design for fatigue.

A Commentary has been prepared to provide background for these and other provisions. In contrast to past versions, the Specification and Commentary are published as a single document, in order to assure that the Commentary is conveniently available to the user at all times.

The reader is cautioned that independent professional judgment must be exercised when data or recommendations set forth in this Specification 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—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 architect, structural engineer, or other licensed professional for the application of principles to a particular structure.

By the Committee,

Robert S. Sherman, *Chairman*
Harold J. Engstrom, *Vice Chairman*
Horatio Allison
William C. Alsmeyer
Arthur P. Arndt
Lynn S. Beedle
Walter E. Blessey
Omer W. Blodgett
Robin Chen
James Chinn
Carson F. Diefenderfer
Richard F. Ferguson
John W. Fisher
Theodore V. Galambos
Edwin H. Gaylord
John A. Gilligan
Eero M. Hekkanen
Theodore R. Higgins
James C. Holesappie
Ira M. Hooper
A. L. Johnson

James D. Johnson
Bruce G. Johnston
Fazlur R. Khan
William J. LeMessurier
Stanley D. Lindsey
William A. Milek
William H. Munse
Clarkson W. Pinkham
Egor P. Popov
Norman W. Rimmer
David E. Stevens
D. C. Shields
Sophus A. Thompson
Charles C. Veys
Ivan M. Viest
Lyle L. Wilson
George Winter
Joseph A. Yura
M. Zar
Frank W. Stockwell, *Secretary*

August 1978

Table of Contents

Nomenclature

A Gross area of an axially loaded compression member, Part 2 (square inches)

A_b Nominal body area of a fastener (square inches); area of an upset rod based upon the major diameter of its threads, i.e., the diameter of a coaxial cylinder which would bound the crests of the upset threads (square inches)

A_c Actual area of effective concrete flange in composite design (square inches)

A_e Effective net area of an axially loaded tension member (square inches)

A_f Area of compression flange (square inches)

A_n Net area of an axially loaded tension member (square inches)

A_s Area of steel beam in composite design (square inches)

A'_s Area of compressive reinforcing steel (square inches)

A_{sr} Area of reinforcing steel providing composite action at point of negative moment (square inches)

A_{st} Cross-sectional area of stiffener or pair of stiffeners (square inches)

A_w Area of girder web (square inches)

A_1 Area of steel bearing on a concrete support (square inches)

A_2 Total cross-sectional area of a concrete support (square inches)

B Bending coefficient dependent upon computed moment or stress at the ends of unbraced segments of a tapered member

C_a Coefficient used in Table 4, Appendix A

C_b Bending coefficient dependent upon moment gradient

C_c Column slenderness ratio separating elastic and inelastic buckling

C'_c Slenderness ratio of compression elements as defined in Appendix C

C_h Coefficient used in Table 12, Appendix A

C_m Coefficient applied to bending term in interaction formula for prismatic members and dependent upon column curvature caused by applied moments

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

C_p Stiffness factor for primary member in a flat roof

C_s Stiffness factor for secondary member in a flat roof

C_t Reduction coefficient in computing effective net area of an axially loaded tension member

C_v Ratio of "critical" web stress, according to the linear buckling theory, to the shear yield stress of web material

C_1 Increment used in computing minimum spacing of oversized and slotted holes

C_2 Increment used in computing minimum edge distance for oversized and slotted holes

D Factor depending upon type of transverse stiffeners; outside diameter of tubular member (inches)

E Modulus of elasticity of steel (29,000 kips per square inch)

E_c Modulus of elasticity of concrete (kips per square inch)

F_a Axial compressive stress permitted in a prismatic member in the absence of bending moment (kips per square inch)

F_{as} Axial compressive stress permitted in the absence of bending moment, for bracing and other secondary members (kips per square inch)

$F_{a\gamma}$ Axial compressive stress permitted in a tapered member in the absence of bending moment (kips per square inch)

F_b Bending stress permitted in a prismatic member in the absence of axial force (kips per square inch)

F'_b Allowable bending stress in compression flange of plate girders as reduced for hybrid girders or because of large web depth-to-thickness ratio (kips per square inch)

$F_{b\gamma}$ Bending stress permitted in a tapered member in the absence of axial force (kips per square inch)

F'_e Euler stress for a prismatic member divided by factor of safety (kips per square inch)

$F'_{e\gamma}$ Euler stress for a tapered member divided by factor of safety (kips per square inch)

F_p Allowable bearing stress (kips per square inch)

F_{sr} Stress range (kips per square inch)

$F_{s\gamma}$ St. Venant torsion resistance bending stress in a tapered member (kips per square inch)

F_t Allowable axial tensile stress (kips per square inch)

F_u Specified minimum tensile strength of the type of steel or fastener being used (kips per square inch)

F_v Allowable shear stress (kips per square inch)

$F_{w\gamma}$ Flange warping torsion resistance bending stress in a tapered member (kips per square inch)

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

F_{yc} Column yield stress (kips per square inch)

F_{yr} Specified minimum yield stress of the longitudinal reinforcing steel (kips per square inch)

F_{yst} Stiffener yield stress (kips per square inch)

H_s Length of a stud shear connector after welding (inches)

I_d Moment of inertia of steel deck supported on secondary members (inches^4)

I_{eff} Effective moment of inertia of composite sections for deflection computations (inches^4)

I_p Moment of inertia of primary member in flat roof framing (inches^4)

I_s Moment of inertia of secondary member in flat roof framing (inches^4); moment of inertia of steel beam in composite construction (inches^4)

I_{tr} Moment of inertia of transformed composite section (inches^4)

K Effective length factor for a prismatic member

K_γ Effective length factor for a tapered member

L_p Length of primary member in flat roof framing (feet)

L_s Length of secondary member in flat roof framing (feet)

M Moment, Part 1 (kip-feet); factored bending moment, Part 2 (kip-feet)

M_1 Smaller moment at end of unbraced length of beam-column

M_2 Larger moment at end of unbraced length of beam-column

M_D Moment produced by dead load

M_L Moment produced by live load

M_m Critical moment that can be resisted by a plastically designed member in the absence of axial load (kip-feet)

M_p Plastic moment (kip-feet)

N Length of bearing of applied load (inches)

N_r — Number of stud shear connectors on a beam in one rib of a metal deck, not to exceed 3 in calculations

N_1 — Number of shear connectors required between point of maximum moment and point of zero moment

N_2 — Number of shear connectors required between concentrated load and point of zero moment

P — Force transmitted by a fastener, Part 1 (kips); factored axial load, Part 2 (kips)

P_{bf} — Factored beam flange or connection plate force in a restrained connection (kips)

P_{cr} — Maximum strength of an axially loaded compression member or beam, Part 2 (kips)

P_e — Euler buckling load, Part 2 (kips)

P_R — Beam reaction divided by the number of bolts in high-strength bolted connection (kips)

P_y — Plastic axial load, equal to profile area times specified minimum yield stress (kips)

Q_a — Ratio of effective profile area of an axially loaded member to its total profile area, Appendix C

Q_s — Axial stress reduction factor where width-thickness ratio of unstiffened elements exceeds limiting value given in Sect. 1.9.1.2, Appendix C

R — Reaction or concentrated load applied to beam or girder (kips); radius (inches)

S — Spacing of secondary members in a flat roof (feet); governing slenderness ratio of a tapered member

S_{eff} — Effective section modulus corresponding to partial composite action (inches^3)

S_s — Section modulus of steel beam used in composite design, referred to the bottom flange (inches^3)

S_{tr} — Section modulus of transformed composite cross section, referred to the bottom flange; based upon maximum permitted effective width of concrete flange (inches^3)

T_b — Specified pretension of a high-strength bolt (kips)

V_h — Total horizontal shear to be resisted by connectors under full composite action (kips)

V'_h — Total horizontal shear provided by the connectors in providing partial composite action (kips)

V_u — Statical shear produced by "ultimate" load in plastic design (kips)

Y — Ratio of yield stress of web steel to yield stress of stiffener steel

Z — Plastic section modulus (inches^3)

a — Clear distance between transverse stiffeners (inches); dimension parallel to the direction of stress, Sect. B1, Table B2 (inches)

a' — Distance required at ends of welded partial length cover plate to develop stress (inches)

b — Actual width of stiffened and unstiffened compression elements (inches); dimension normal to the direction of stress, Sect. B1, Table B2 (inches)

b_e — Effective width of stiffened compression element (inches)

b_f — Flange width of rolled beam or plate girder (inches)

d — Depth of beam or girder (inches); diameter of a roller or rocker bearing (inches); nominal diameter of a fastener (inches)

d_c — Column web depth clear of fillets (inches)

d_L — Depth at the larger end of a tapered member (inches)

d_l — Depth at the larger end of an unbraced segment of a tapered member (inches)

d_o	Depth at the smaller end of a tapered member or unbraced segment thereof (inches)
f	Axial compression stress on member based on effective area, Sect. C3 (kips per square inch)
f_a	Computed axial stress (kips per square inch)
f_{ao}	Computed axial stress at the smaller end of a tapered member or unbraced segment thereof (kips per square inch)
f_b	Computed bending stress (kips per square inch)
f_{b1}	Smallest computed bending stress at one end of a tapered segment (kips per square inch)
f_{b2}	Largest computed bending stress at one end of a tapered segment (kips per square inch)
f_{bl}	Computed bending stress at the larger end of a tapered member or unbraced segment thereof (kips per square inch)
f'_c	Specified compression strength of concrete (kips per square inch)
f_t	Computed tensile stress (kips per square inch)
f_v	Computed shear stress (kips per square inch)
f_{vs}	Shear between girder web and transverse stiffeners (kips per linear inch of single stiffener or pair of stiffeners)
g	Transverse spacing between fastener gage lines (inches)
h	Clear distance between flanges of a beam or girder at the section under investigation (inches)
h_r	Nominal rib height for steel deck (inches)
h_s	Factor applied to the unbraced length of a tapered member
h_w	Factor applied to the unbraced length of a tapered member
k	Coefficient relating linear buckling strength of a plate to its dimensions and condition of edge support; distance from outer face of flange to web toe of fillet of rolled shape or equivalent distance on welded section (inches)
l	For beams, distance between cross sections braced against twist or lateral displacement of the compression flange (inches); for columns, actual unbraced length of member (inches); unsupported length of a lacing bar (inches)
l_b	Actual unbraced length in plane of bending (inches)
l_{cr}	Critical unbraced length adjacent to plastic hinge (inches)
n	Modular ratio (E/E_c)
q	Allowable horizontal shear to be resisted by a shear connector (kips)
r	Governing radius of gyration (inches)
r_b	Radius of gyration about axis of concurrent bending (inches)
r_{bo}	Radius of gyration about axis of concurrent bending at the smaller end of a tapered member or unbraced segment thereof (inches)
r_o	Radius of gyration at the smaller end of a tapered member (inches)
r_T	Radius of gyration of a section comprising the compression flange plus $\frac{1}{3}$ of the compression web area, taken about an axis in the plane of the web (inches)
r_{To}	Radius of gyration at the smaller end of a tapered member or unbraced segment thereof, considering only the compression flange plus $\frac{1}{3}$ of the compression web area, taken about an axis in the plane of the web (inches)
s	Longitudinal center-to-center spacing (pitch) of any two consecutive holes (inches)
t	Girder, beam, or column web thickness (inches); thickness of a connected part (inches); wall thickness of a tubular member (inches)

t_b Thickness of beam flange or moment connection plate at rigid beam-to-column connection (inches)

t_f Flange thickness (inches)

w Length of channel shear connectors (inches)

w_r Average width of rib or haunch of concrete slab on formed steel deck (inches)

x Subscript relating symbol to strong axis bending

y Subscript relating symbol to weak axis bending

z Distance from the smaller end of a tapered member (inches)

α Ratio of hybrid girder web yield stress to flange yield stress

β Ratio S_{tr}/S_s or S_{eff}/S_s

γ Tapering ratio of a tapered member or unbraced segment of a tapered member; subscript relating symbol to tapered members

Δ Displacement of the neutral axis of a loaded member from its position when the member is not loaded (inches)

Design, Fabrication and Erection of Structural Steel for Buildings

Adopted Effective November 1, 1978

PART 1

SECTION 1.1 PLANS AND DRAWINGS

1.1.1 Plans

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

Plans shall indicate the type or types of construction (as defined in Sect. 1.2) to be employed, and they shall be supplemented by such data concerning the assumed loads, shears, moments, and axial forces to be resisted by all members and their connections, as may be required for the proper preparation of the shop drawings.

Where joints are to be assembled with high-strength bolts and are required to resist shear between the connected parts, the plans shall indicate the type of connections to be provided, namely, friction or bearing.

Camber of trusses, beams and girders, if required, shall be called for on the design drawings.

1.1.2 Shop Drawings

Shop drawings, giving complete information necessary for the fabrication of the component parts of the structure, including the location, type, and size of all rivets, bolts, and welds, shall be prepared in advance of the actual fabrication. They shall clearly distinguish between shop and field rivets, bolts, and welds.

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

1.1.3 Notations for Welding

Note shall be made on the plans and on the shop drawings of those joints or groups of joints in which it is especially important that the welding sequence and technique of welding be carefully controlled to minimize welding under restraint and to avoid undue distortion.

Weld lengths called for on the plans and on the shop drawings shall be the net effective lengths.

1.1.4 Standard Symbols and Nomenclature

Welding 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 on the plans or drawings.

Unless otherwise noted, the standard nomenclature contained in the joint AISC-SJI *Standard Specifications for Open Web Steel Joists, Longspan Steel Joists, and Deep Longspan Steel Joists,* latest edition, shall be used in describing steel joists.

SECTION 1.2 TYPES OF CONSTRUCTION

Three basic types of construction and associated design assumptions are permissible under the respective conditions stated hereinafter, and each will govern in a specific manner the size of members and the types and strength of their connections:

Type 1, commonly designated as "rigid-frame" (continuous frame), assumes that beam-to-column connections have sufficient rigidity to hold virtually unchanged the original angles between intersecting members.

Type 2, commonly designated as "simple framing" (unrestrained, free-ended), assumes that, insofar as gravity loading is concerned, the ends of beams and girders are connected for shear only, and are free to rotate under gravity load.

Type 3, commonly designated as "semi-rigid framing" (partially restrained), assumes that the connections of beams and girders possess a dependable and known moment capacity intermediate in degree between the rigidity of Type 1 and the flexibility of Type 2.

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

Type 1 construction is unconditionally permitted under this Specification. Two different methods of design are recognized. Within the limitations laid down in Sect. 2.1, members of continuous frames or continuous portions of frames may be proportioned, on the basis of their maximum predictable strength, to resist the specified design loads multiplied by the prescribed load factors. Otherwise, Type 1 construction shall be designed, within the limitations of Sect. 1.5, to resist the stresses produced by the specified design loads, assuming moment distribution in accordance with the elastic theory.

Type 2 construction is permitted under this Specification, subject to the stipulations of the following paragraph, wherever applicable.

In buildings designed as Type 2 construction (i.e., with beam-to-column connections other than wind connections assumed flexible under gravity loading) the wind moments may be distributed among selected joints of the frame, provided that:

1. The connections and connected members have adequate capacity to resist the wind moments.

2. The girders are adequate to carry the full gravity load as "simple beams".

3. The connections have adequate inelastic rotation capacity to avoid overstress of the fasteners or welds under combined gravity and wind loading.

Type 3 (semi-rigid) construction will be permitted only upon evidence that the connections to be used are capable of furnishing, as a minimum, a predictable proportion of full end restraint. The proportioning of main members joined by such connections shall be predicated upon no greater degree of end restraint than this minimum.

Types 2 and 3 construction may necessitate some nonelastic, but self-limiting, deformation of a structural steel part.

SECTION 1.3 LOADS AND FORCES

1.3.1 Dead Load

The dead load to be assumed in design shall consist of the weight of steelwork and all material permanently fastened thereto or supported thereby.

1.3.2 Live Load

The live load, including snow load if any, shall be that stipulated by the applicable code under which the structure is being designed or that dictated by the conditions involved. Snow load shall be considered as applied either to the entire roof area or to a portion of the roof area, and any probable arrangement of loads resulting in the highest stresses in the supporting members shall be used in the design.

1.3.3 Impact

For structures carrying live loads* which induce impact, the assumed live load shall be increased sufficiently to provide for same.

If not otherwise specified, the increase shall be:

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

1.3.4 Crane Runway Horizontal Forces

The lateral force on crane runways to provide for the effect of moving crane trolleys shall, if not otherwise specified, be 20 percent of the sum of the 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, if not otherwise specified, be taken as 10 percent of the maximum wheel loads of the crane applied at the top of rail.

---

* Live loads on crane support girders shall be taken as the maximum crane wheel loads.

1.3.5 Wind

Proper provision shall be made for stresses caused by wind, both during erection and after completion of the building.

1.3.6 Other Forces

Structures in localities subject to earthquakes, hurricanes and other extraordinary conditions shall be designed with due regard for such conditions.

1.3.7 Minimum Loads

In the absence of any applicable building code requirements, the loads referred to in Sects. 1.3.1, 1.3.2, 1.3.5, and 1.3.6 shall be not less than those recommended in the American National Standards Institute *Building Code Requirements for Minimum Design Loads in Buildings and Other Structures,* ANSI A58.1, latest edition.

SECTION 1.4 MATERIAL

1.4.1 Structural Steel

1.4.1.1 Material conforming to one of the following standard specifications (latest date of issue) is approved for use under this Specification:

> *Structural Steel,* ASTM A36
> *Welded and Seamless Steel Pipe,* ASTM A53, Grade 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, Grades D and E
> *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, and the governing specification shall constitute sufficient evidence of conformity with one of the above ASTM standards. Additionally, the fabricator shall, if requested, provide an affidavit stating that the structural steel furnished meets the requirements of the grade specified.

1.4.1.2 Unidentified steel, if free from surface imperfections, may be used for parts of minor importance, or for unimportant details, where the precise physical properties of the steel and its weldability would not affect the strength of the structure.

1.4.2 Other Metals

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

> *Mild-to-Medium-Strength Carbon-Steel Castings for General Applications,* ASTM A27, Grade 65-35
>
> *High-Strength Steel Castings for Structural Purposes,* ASTM A148, Grade 80-50

Steel forgings shall conform to the following standard specification, latest edition:

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

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

1.4.3 Rivets

Steel rivets shall conform to the following standard specification, latest edition:

> *Steel Structural Rivets,* ASTM A502

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

1.4.4 Bolts

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

> *Low-Carbon Steel Externally and Internally Threaded Standard Fasteners,* ASTM A307
>
> *High Strength Bolts for Structural Steel Joints, Including Suitable Nuts and Plain Hardened Washers,* ASTM A325
>
> *Quenched and Tempered Steel Bolts and Studs,* ASTM A449
>
> *Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints,* ASTM A490

In connections, A449 bolts may be used only in bearing-type connections requiring bolt diameters greater than $1\frac{1}{2}$ inches. A449 bolt material is also acceptable for high-strength anchor bolts and threaded rods of any diameter.

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

1.4.5 Filler Metal and Flux for Welding

Welding electrodes and fluxes shall conform to one of the following specifications of the American Welding Society, latest adoption, as appropriate:*

---

* Approval of these welding electrode specifications is given without regard to weld metal notch toughness requirements, which are generally not necessary for building construction. See Commentary Sect. 1.4.

> *Specification for Mild Steel Covered Arc-Welding Electrodes,* AWS A5.1
>
> *Specification for Low-Alloy Steel Covered Arc-Welding Electrodes,* AWS A5.5
>
> *Specification for Bare Mild Steel Electrodes and Fluxes for Submerged-Arc Welding,* AWS A5.17
>
> *Specification for Mild Steel Electrodes for Gas Metal-Arc Welding,* AWS A5.18
>
> *Specification for Mild Steel Electrodes for Flux-Cored Arc Welding,* AWS A5.20
>
> *Specification for Bare Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding,* AWS A5.23

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

1.4.6   Stud Shear Connectors

Steel stud shear connectors shall conform to the requirements of Articles 4.26 and 4.27, *Structural Welding Code,* AWS D1.1-77, of the American Welding Society.

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

SECTION 1.5   ALLOWABLE STRESSES*

Except as provided in Sects. 1.6, 1.7, 1.10, 1.11, 1.16.4, and in Part 2, all components of the structure shall be so proportioned that the stress, in kips per square inch, shall not exceed the following values, except as rounded off in Appendix A.   See Appendix D for allowable stresses for web-tapered members.

1.5.1   Structural Steel

1.5.1.1   Tension

Except for pin-connected members, F_t shall not exceed $0.60F_y$ on the gross area nor $0.50F_u$ on the effective net area.**

For pin-connected members: $F_t = 0.45F_y$ on the net area.**

For tension on threaded parts: See Table 1.5.2.1

1.5.1.2   Shear[†]

1.5.1.2.1   Except as provided in Sects. 1.5.1.2.2 and 1.10.5.2, on the cross-sectional area effective in resisting shear:

$$F_v = 0.40F_y$$

The effective area in resisting shear of rolled and fabricated shapes may be taken as the overall depth times the web thickness.

---

   * See Appendix A for tables of numerical values for various grades of steel corresponding to provisions of this Section.

   ** For determination of effective net area, see Sect. 1.14.

   † See Commentary Sect. 1.5.1.2.

1.5.1.2.2   At beam end connections where the top flange is coped, and in similar situations where failure might occur by shear along a plane through the fasteners, or by a combination of shear along a plane through the fasteners plus tension along a perpendicular plane, on the area effective in resisting tearing failure:

$$F_v = 0.30F_u$$

The effective area is the minimum net failure surface, bounded by the bolt holes.*

1.5.1.3   Compression

1.5.1.3.1   On the gross section of axially loaded compression members whose cross sections meet the provisions of Sect. 1.9, when Kl/r, the largest effective slenderness ratio of any unbraced segment as defined in Sect. 1.8, is less than C_c:

$$F_a = \frac{\left[1 - \dfrac{(Kl/r)^2}{2C_c^{\,2}}\right]F_y}{\dfrac{5}{3} + \dfrac{3(Kl/r)}{8C_c} - \dfrac{(Kl/r)^3}{8C_c^{\,3}}} \tag{1.5-1}$$

where
$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$$

1.5.1.3.2   On the gross section of axially loaded compression members, when Kl/r exceeds C_c:

$$F_a = \frac{12\pi^2 E}{23(Kl/r)^2} \tag{1.5-2}$$

1.5.1.3.3   On the gross section of axially loaded bracing and secondary members, when l/r exceeds 120:**

$$F_{as} = \frac{F_a \,[\text{by Formula (1.5-1) or (1.5-2)}]}{1.6 - \dfrac{l}{200r}} \tag{1.5-3}$$

1.5.1.3.4   On the gross area of plate girder stiffeners:

$$F_a = 0.60F_y$$

1.5.1.3.5   On the web of rolled shapes at the toe of the fillet (crippling, see Sect. 1.10.10):

$$F_a = 0.75F_y$$

---

* See Commentary Fig. C1.5.1.2.

** For this case, K is taken as unity.

1.5.1.4 Bending

1.5.1.4.1   Tension and compression on extreme fibers of compact hot-rolled or built-up members (except hybrid girders and members of A514 steel) symmetrical about, and loaded in, the plane of their minor axis and meeting the requirements of this section:

$$F_b = 0.66F_y$$

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

1. The flanges shall be continuously connected to the web or webs.

2. The width-thickness ratio of unstiffened projecting elements of the compression flange, as defined in Sect. 1.9.1.1, shall not exceed $65/\sqrt{F_y}$.

3. The width-thickness ratio of stiffened elements of the compression flange, as defined in Sect. 1.9.2.1, shall not exceed $190/\sqrt{F_y}$.

4. The depth-thickness ratio of the web or webs shall not exceed the value given by Formula (1.5-4a) or (1.5-4b), as applicable.

$$d/t = \frac{640}{\sqrt{F_y}} \left(1 - 3.74 \frac{f_a}{F_y}\right) \qquad \text{when } f_a/F_y \leq 0.16 \qquad (1.5\text{-}4a)$$

$$d/t = 257/\sqrt{F_y} \qquad \text{when } f_a/F_y > 0.16 \qquad (1.5\text{-}4b)$$

5. The laterally unsupported length of the compression flange of members other than circular or box members shall not exceed the value

$$\frac{76b_f}{\sqrt{F_y}} \quad \text{nor} \quad \frac{20{,}000}{(d/A_f)F_y}$$

6. The laterally unsupported length of the compression flange of a box-shaped member of rectangular cross section whose depth is not more than 6 times the width and whose flange thickness is not more than 2 times the web thickness shall not exceed the value

$$\left(1950 + 1200 \frac{M_1}{M_2}\right) \frac{b}{F_y}$$

except that it need not be less than $1200(b/F_y)$.

7. The diameter-thickness ratio of hollow circular sections shall not exceed $3300/F_y$.

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 subparagraphs 1 through 7, 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 $0.15F_a$.

1.5.1.4.2   Members (except hybrid girders and members of A514 steel) which meet the requirements of Sect. 1.5.1.4.1, except that $b_f/2t_f$ exceeds $65/\sqrt{F_y}$ but is less than $95/\sqrt{F_y}$, may be designed on the basis of an allowable bending stress

$$F_b = F_y \left[0.79 - 0.002 \left(\frac{b_f}{2t_f} \right) \sqrt{F_y} \right] \qquad (1.5\text{-}5a)$$

1.5.1.4.3   Tension and compression on extreme fibers of doubly-symmetrical I- and H-shape members meeting the requirements of Sect. 1.5.1.4.1, subparagraphs 1 and 2, and bent about their minor axis (except members of A514 steel); solid round and square bars;   solid rectangular sections bent about their weaker axis:

$$F_b = 0.75F_y$$

Doubly-symmetrical I- and H-shape members bent about their minor axis (except hybrid girders and members of A514 steel) meeting the requirements of Sect. 1.5.1.4.1, subparagraph 1, except where $b_f/2t_f$ exceeds $65/\sqrt{F_y}$ but is less than $95/\sqrt{F_y}$, may be designed on the basis of an allowable bending stress

$$F_b = F_y \left[1.075 - 0.005 \left(\frac{b_f}{2t_f} \right) \sqrt{F_y} \right] \qquad (1.5\text{-}5b)$$

Rectangular tubular sections meeting the requirements of Sect. 1.5.1.4.1, subparagraphs 1, 3, and 4, and bent about their minor axis, may be designed on the basis of an allowable bending stress

$$F_b = 0.66F_y$$

1.5.1.4.4   Tension and compression on extreme fibers of box-type flexural members whose compression flange or web width-thickness ratio does not meet the requirements of Sect. 1.5.1.4.1, but does conform to the requirements of Sect. 1.9:

$$F_b = 0.60F_y$$

Lateral torsional buckling need not be investigated for a box section whose depth is less than 6 times its width.   Lateral support requirements for box sections of larger depth-to-width ratios must be determined by special analysis.

1.5.1.4.5   On extreme fibers of flexural members not covered in Sect. 1.5.1.4.1, 1.5.1.4.2, 1.5.1.4.3, or 1.5.1.4.4:

1. Tension:

$$F_b = 0.60F_y$$

2. Compression:

a. For members meeting the requirements of Sect. 1.9.1.2, having an axis of symmetry in, and loaded in, the plane of their web, and compression on extreme fibers of channels bent about their major axis:

The larger value computed by Formulas (1.5-6a) or (1.5-6b) and (1.5-7), as applicable* (unless a higher value can be justified on the basis of a more precise analysis**), but not more than $0.60F_y$.†

When $\quad \sqrt{\dfrac{102 \times 10^3 C_b}{F_y}} \leq \dfrac{l}{r_T} \leq \sqrt{\dfrac{510 \times 10^3 C_b}{F_y}}$:

$$F_b = \left[\frac{2}{3} - \frac{F_y(l/r_T)^2}{1530 \times 10^3 C_b}\right]F_y \qquad\qquad (1.5\text{-}6a)$$

When $\quad l/r_T \geq \sqrt{\dfrac{510 \times 10^3 C_b}{F_y}}$:

$$F_b = \frac{170 \times 10^3 C_b}{(l/r_T)^2} \qquad\qquad (1.5\text{-}6b)$$

Or, when the compression flange is solid and approximately rectangular in cross section and its area is not less than that of the tension flange:

$$F_b = \frac{12 \times 10^3 C_b}{ld/A_f} \qquad\qquad (1.5\text{-}7)$$

In the foregoing,

$l \ =$ distance between cross sections braced against twist or lateral displacement of the compression flange, inches. For cantilevers braced against twist only at the support, l may conservatively be taken as the actual length.

$r_T \ =$ radius of gyration of a section comprising the compression flange plus $\frac{1}{3}$ of the compression web area, taken about an axis in the plane of the web, inches

$A_f \ =$ area of the compression flange, square inches

$C_b \ =$ $1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2$, but not more than 2.3,‡ where M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length, taken about the strong axis of the member, and where M_1/M_2, the ratio of end moments, is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite signs (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, the value of C_b shall be taken as unity. When computing F_{bx} and F_{by} to be used in Formula (1.6-1a), C_b may be

---

* Only Formula (1.5-7) applicable to channels.
** See Commentary Sect. 1.5.1.4.5 for alternate procedures.
† See Sect. 1.10 for further limitations in plate girder flange stress.
‡ C_b can be conservatively taken as unity. For smaller values see Appendix A, Table 7.

computed by the formula given above for frames subject to joint translation, and it shall be taken as unity for frames braced against joint translation. C_b may conservatively be taken as unity for cantilever beams.*

For hybrid plate girders, F_y for Formulas (1.5-6a) and (1.5-6b) is the yield stress of the compression flange. Formula (1.5-7) shall not apply to hybrid girders.

b. For members meeting the requirements of Sect. 1.9.1.2, but not included in subparagraph 2a of this Section:

$$F_b = 0.60F_y$$

provided that sections bent about their major axis are braced laterally in the region of compression stress at intervals not exceeding $76b_f/\sqrt{F_y}$.

1.5.1.5 Bearing

1.5.1.5.1 On contact area of milled surfaces and ends of fitted bearing stiffeners; on projected area of pins in reamed, drilled, or bored holes:

$$F_p = 0.90F_y**$$

1.5.1.5.2 Expansion rollers and rockers, kips per linear inch:

$$F_p = \left(\frac{F_y - 13}{20}\right) 0.66d$$

where d is the diameter of roller or rocker, inches.

1.5.1.5.3 On projected area of bolts and rivets in shear connections.†

$$F_p = 1.5F_u$$

where F_u is the specified minimum tensile strength of the connected parts, kips per square inch.

1.5.2 Rivets, Bolts, and Threaded Parts‡

1.5.2.1 Allowable tension and shear stresses on rivets, bolts, and threaded parts shall be as given in Table 1.5.2.1, in kips per square inch of the nominal body area of rivets (before driving) or the unthreaded nominal body area of bolts and threaded parts other than upset rods (see Footnote c, Table 1.5.2.1). High-strength bolts required to support applied load by means of direct tension shall be so proportioned that their average tensile stress, computed on the basis of nominal bolt area and independent of any initial tightening force, will not exceed the appropriate stress given in Table 1.5.2.1. The applied load shall be the sum of the external load and any tension resulting from prying action produced by deformation of the connected parts.

---

   * For the use of larger C_b values, see Structural Stability Research Council *Guide to Stability Design Criteria for Metal Structures,* Third Edition, p. 135.

  ** When parts in contact have different yield stresses, F_y shall be the smaller value.

   † For minimum spacing and edge distances, see Sects. 1.16.4 and 1.16.5.

   ‡ For allowable bearing stress on connected parts in bolted or riveted bearing-type connections, see Sect. 1.5.1.5.3.

1.5.2.2 Design for rivets, bolts, and threaded parts subject to fatigue loading shall be in accordance with Appendix B, Sect. B3.

TABLE 1.5.2.1

ALLOWABLE STRESS ON FASTENERS, KSI

Description of Fasteners	Allow-able Tension[g] (F_t)	Allowable Shear[g] (F_v)			
		Friction-type Connections[e,i]			Bearing-type Connec-tions[i]
		Standard size Holes	Oversized and Short-slotted Holes	Long-slotted Holes	
A502, Grade 1, hot-driven rivets	23.0[a]				17.5[f]
A502, Grades 2 and 3, hot-driven rivets	29.0[a]				22.0[f]
A307 bolts	20.0[a]				10.0[b,f]
Threaded parts meeting the requirements of Sects. 1.4.1 and 1.4.4, and A449 bolts meeting the requirements of Sect. 1.4.4, when threads are not excluded from shear planes	$0.33F_u$[a,c,h]				$0.17F_u$[h]
Threaded parts meeting the requirements of Sects. 1.4.1 and 1.4.4, and A449 bolts meeting the requirements of Sect. 1.4.4, when threads are excluded from shear planes	$0.33F_u$[a,h]				$0.22F_u$[h]
A325 bolts, when threads are not excluded from shear planes	44.0[d]	17.5	15.0	12.5	21.0[f]
A325 bolts, when threads are excluded from shear planes	44.0[d]	17.5	15.0	12.5	30.0[f]
A490 bolts, when threads are not excluded from shear planes	54.0[d]	22.0	19.0	16.0	28.0[f]
A490 bolts, when threads are excluded from shear planes	54.0[d]	22.0	19.0	16.0	40.0[f]

[a] Static loading only.

[b] Threads permitted in shear planes.

[c] The tensile capacity 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 $0.60F_y$.

[d] For A325 and A490 bolts subject to tensile fatigue loading, see Appendix B, Sect. B3.

[e] When specified by the designer, the allowable shear stress, F_v, for friction-type connections having special faying surface conditions may be increased to the applicable value given in Appendix E.

[f] When bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 50 inches, tabulated values shall be reduced by 20 percent.

[g] See Sect. 1.5.6.

[h] See Appendix A, Table 2, for values for specific ASTM steel specifications.

[i] For limitations on use of oversized and slotted holes, see Sect. 1.23.4.

1.5.3   Welds

Except as modified by the provisions of Sect. 1.7, welds shall be proportioned to meet the stress requirements given in Table 1.5.3.

<div align="center">TABLE 1.5.3</div>

<div align="center">ALLOWABLE STRESS ON WELDS</div>

Type of Weld and Stress[a]	Allowable Stress	Required Weld Strength Level[b,c]
Complete-Penetration Groove Welds		
Tension normal to effective area	Same as base metal	"Matching" weld metal must be used.
Compression normal to effective area	Same as base metal	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	Same as base metal	Weld metal with a strength level equal to or less than "matching" weld metal may be used.
Shear on effective area	$0.30 \times$ nominal tensile strength of weld metal (ksi), except shear stress on base metal shall not exceed $0.40 \times$ yield stress of base metal	
Partial-Penetration Groove Welds[d]		
Compression normal to effective area	Same as base metal	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[e]	Same as base metal	
Shear parallel to axis of weld	$0.30 \times$ nominal tensile strength of weld metal (ksi), except shear stress on base metal shall not exceed $0.40 \times$ yield stress of base metal	
Tension normal to effective area	$0.30 \times$ nominal tensile strength of weld metal (ksi), except tensile stress on base metal shall not exceed $0.60 \times$ yield stress of base metal	
Fillet Welds		
Shear on effective area	$0.30 \times$ nominal tensile strength of weld metal (ksi), except shear stress on base metal shall not exceed $0.40 \times$ yield stress of base metal	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[e]	Same as base metal	
Plug and Slot Welds		
Shear parallel to faying surfaces (on effective area)	$0.30 \times$ nominal tensile strength of weld metal (ksi), except shear stress on base metal shall not exceed $0.40 \times$ yield stress of base metal	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. 1.14.6.
[b] for "matching" weld metal, see Table 4.1.1, AWS D1.1-77.
[c] Weld metal one strength level stronger than "matching" weld metal will be permitted.
[d] See Sect. 1.10.8 for a limitation on use of partial-penetration groove welded joints.
[e] Fillet welds and partial-penetration groove welds joining the 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.

1.5.4   Cast Steel and Steel Forgings

Allowable stresses shall be the same as those provided in Sect. 1.5.1, where applicable.

1.5.5   Masonry Bearing

In the absence of Code regulations the following stresses apply:

On sandstone and limestone . . . . $F_p = 0.40$ ksi

On brick in cement mortar . . . . $F_p = 0.25$ ksi

On the full area of a concrete support . . . . $F_p = 0.35f'_c$

On less than the full area of a
concrete support . . . . $F_p \;=\; 0.35f'_c\sqrt{A_2/A_1} \;\leq\; 0.7f'_c$

where f'_c = specified compressive strength of concrete, kips per square inch
A_1 = bearing area, square inches
A_2 = full cross-sectional area of concrete support, square inches

1.5.6   Wind and Seismic Stresses

Allowable stresses may be increased $\frac{1}{3}$ above the values otherwise provided when produced by wind or seismic loading, acting alone or in combination with the design dead and live loads, provided the required section computed on this basis is not less than that required for the design dead and live load and impact (if any), computed without the $\frac{1}{3}$ stress increase, and further provided that stresses are not otherwise* required to be calculated on the basis of reduction factors applied to design loads in combinations.   The above stress increase does not apply to allowable stress ranges provided in Appendix B.

SECTION 1.6   COMBINED STRESSES

1.6.1   Axial Compression and Bending

Members subjected to both axial compression and bending stresses shall be proportioned to satisfy the following requirements:

$$\frac{f_a}{F_a} + \frac{C_{mx}f_{bx}}{\left(1 - \dfrac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my}f_{by}}{\left(1 - \dfrac{f_a}{F'_{ey}}\right) F_{by}} \;\leq\; 1.0 \qquad (1.6\text{-}1a)$$

$$\frac{f_a}{0.60F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \;\leq\; 1.0 \qquad (1.6\text{-}1b)$$

When $f_a/F_a \leq 0.15$, Formula (1.6-2) may be used in lieu of Formulas (1.6-1a) and (1.6-1b):

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \;\leq\; 1.0 \qquad (1.6\text{-}2)$$

In Formulas (1.6-1a), (1.6-1b), and (1.6-2), the subscripts x and y, combined with subscripts b, m, and e, indicate the axis of bending about which a particular stress or design property applies, and

---

* For example, see ANSI A58.1-72, Sect. 4.2.

F_a = axial compressive stress that would be permitted if axial force alone existed, kips per square inch

F_b = compressive bending stress that would be permitted if bending moment alone existed, kips per square inch

$$F'_e = \frac{12 \, \pi^2 E}{23(Kl_b/r_b)^2}$$

= Euler stress divided by a factor of safety, kips per square inch. (In the expression for F'_e, l_b is the actual unbraced length *in the plane of bending* and r_b is the corresponding radius of gyration. K is the effective length factor *in the plane of bending.* As in the case of F_a, F_b, and $0.60F_y$, F'_e may be increased $\frac{1}{3}$ in accordance with Sect. 1.5.6)

f_a = computed axial stress, kips per square inch

f_b = computed compressive bending stress at the point under consideration, kips per square inch

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

1. For compression members in frames subject to joint translation (sidesway), $C_m = 0.85$.

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

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

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

3. 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 may be determined by rational analysis. However, in lieu of such analysis, the following values may be used:

   a. For members whose ends are restrained . . . $C_m = 0.85$

   b. For members whose ends are unrestrained . . $C_m = 1.0$

1.6.2 Axial Tension and Bending

Members subject to both axial tension and bending stresses shall be proportioned at all points along their length to satisfy the requirements of Formula (1.6-1b), where f_b is the computed bending tensile stress. However, the computed bending compressive stress, taken alone, shall not exceed the applicable value according to Sect. 1.5.1.4.

1.6.3 Shear and Tension

Rivets and bolts subject to combined shear and tension shall be so proportioned that the tension stress, F_t, in kips per square inch on the nominal body area, A_b, produced by forces applied to the connected parts, shall not exceed the values computed from the formulas in Table 1.6.3, where f_v, the shear stress produced by the same forces, shall not exceed the value for shear given in Sect. 1.5.2. When allowable stresses are increased for wind or seismic loads in accord-

ance with Sect. 1.5.6, the constants in the formulas listed in Table 1.6.3 shall be increased by $\frac{1}{3}$, but the coefficient applied to f_v shall not be increased.

For A325 and A490 bolts used in friction-type connections, the maximum shear stress allowed by Table 1.5.2.1 shall be multiplied by the reduction factor $(1 - f_t A_b / T_b)$, where f_t is the average tensile stress due to a direct load applied to all of the bolts in a connection and T_b is the specified* pretension load of the bolt. When allowable stresses are increased for wind or seismic loads in accordance with the provisions of Sect. 1.5.6, the reduced allowable shear stress shall be increased by $\frac{1}{3}$.

<div align="center">

TABLE 1.6.3

ALLOWABLE TENSION STRESS (F_t) FOR FASTENERS
IN BEARING-TYPE CONNECTIONS

</div>

Description of Fastener	Threads **Not** Excluded from Shear Planes	Threads Excluded from Shear Planes
Threaded parts		
A449 bolts over $1\frac{1}{2}$-in. diameter	$0.43F_u - 1.8f_v \leq 0.33F_u$	$0.43F_u - 1.4f_v \leq 0.33F_u$
A325 bolts	$55 - 1.8f_v \leq 44$	$55 - 1.4f_v \leq 44$
A490 bolts	$68 - 1.8f_v \leq 54$	$68 - 1.4f_v \leq 54$
A502 Grade 1 rivets	$30 - 1.3f_v \leq 23$	
A502 Grades 2 and 3 rivets	$38 - 1.3f_v \leq 29$	
A307 bolts	$26 - 1.8f_v \leq 20$	

SECTION 1.7   MEMBERS AND CONNECTIONS SUBJECT TO REPEATED VARIATION OF STRESS (FATIGUE)

1.7.1   General

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, 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 arrangements of live load.

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.

---

* See "Minimum Bolt Tension" values, Table 1.23.5.

1.7.2 Design for Fatigue

Members and their connections subject to fatigue loading shall be proportioned in accordance with the provisions of Appendix B.

SECTION 1.8 STABILITY AND SLENDERNESS RATIOS

1.8.1 General

General stability shall be provided for the structure as a whole and for each compression element. Design consideration should be given to significant load effects resulting from the deflected shape of the structure or of individual elements of the lateral load resisting system, including the effects on beams, columns, bracing, connections, and shear walls.

In determining the slenderness ratio of an axially loaded compression member, except as provided in Sect. 1.5.1.3.3, the length shall be taken as its effective length Kl and r as the corresponding radius of gyration.

1.8.2 Braced Frames

In trusses and in those frames where lateral stability is provided by adequate attachment to diagonal bracing, to shear walls, to an adjacent structure having adequate lateral stability, or to floor slabs or roof decks secured horizontally by walls or bracing systems parallel to the plane of the frame, the effective length factor, K, for the compression members shall be taken as unity, unless analysis shows that a smaller value may be used.

1.8.3 Unbraced Frames

In frames where lateral stability is dependent upon the bending stiffness of rigidly connected beams and columns, the effective length Kl of compression members shall be determined by a rational method and shall not be less than the actual unbraced length.

1.8.4 Maximum Ratios

The slenderness ratio, Kl/r, of compression members shall not exceed 200.

The slenderness ratio, l/r, of tension members, other than rods, preferably should not exceed:

> For main members   240
> For lateral bracing members and other secondary members . .   300

SECTION 1.9 WIDTH-THICKNESS RATIOS

1.9.1 Unstiffened Elements Under Compression

1.9.1.1   Unstiffened (projecting) compression elements are those having one free edge parallel to the direction of compression stress. The width of unstiffened plates shall be taken from the free edge to the first row of fasteners or welds; the width of legs of angles, channel and zee flanges, and stems of tees shall be taken as the full nominal dimension; the width of flanges of I- and H-shape members and tees shall be taken as $\frac{1}{2}$ the full nominal width. The thickness of a sloping flange shall be measured at a section half-way between a free edge and the corresponding face of the web.

1.9.1.2   Unstiffened elements subject to axial compression or compression due to bending shall be considered as fully effective when the ratio of width to thickness is not greater than the following:

Single-angle struts;   double-angle struts with separators . . . $76/\sqrt{F_y}$

Struts comprising double angles in contact;   angles or plates projecting from girders, columns, or other compression members;   compression flanges of beams;   stiffeners on plate girders . . . . $95/\sqrt{F_y}$

Stems of tees . . . . $127/\sqrt{F_y}$

When the actual width-to-thickness ratio exceeds these values, the design stress shall be governed by the provisions of Appendix C.

1.9.2   Stiffened Elements Under Compression

1.9.2.1   Stiffened compression elements are those having lateral support along both edges that are parallel to the direction of the compression stress. The width of such elements shall be taken as the distance between nearest lines of fasteners or welds, or between the roots of the flanges in the case of rolled sections.

1.9.2.2   Stiffened elements subject to axial compression, or to uniform compression due to bending as in the case of the flange of a flexural* member, shall be considered as fully effective when the ratio of width to thickness is not greater than the following:

Flanges of square and rectangular box sections of uniform thickness . . . . $238/\sqrt{F_y}$

Unsupported width of cover plates perforated with a succession of access holes** . . . . $317/\sqrt{F_y}$

All other uniformly compressed stiffened elements . . . . . . $253/\sqrt{F_y}$

Except in the case of perforated cover plates, when the actual width-to-thickness ratio exceeds these values the design shall be governed by the provisions of Appendix C.

1.9.2.3   Circular tubular elements subject to axial compression shall be considered as fully effective when the ratio of the outside diameter to the wall thickness is not greater than $3300/F_y$. For diameter-to-thickness ratios greater than $3300/F_y$ but less than $13,000/F_y$, see Appendix C.

SECTION 1.10   PLATE GIRDERS AND ROLLED BEAMS

1.10.1   Proportions

Plate girders, coverplated beams, and rolled or welded beams shall in general be proportioned by the moment of inertia of the gross section. No deduction shall be made for shop or field rivet or bolt holes in either flange, except that in cases where the reduction of the area of either flange by such holes, calculated in accordance with the provisions of Sect. 1.14.2, exceeds 15 percent of the gross flange area, the excess shall be deducted.

---

 * Webs of flexural members are covered by the provisions of Sects. 1.10.2 and 1.10.6 and are not subject to the provisions of this section.

** Assumes net area of plate at widest hole as basis for computing compression stress.

Hybrid girders may be proportioned by the moment of inertia of their gross section,* subject to the applicable provisions in Sect. 1.10, provided that they are not required to resist an axial force greater than $0.15F_y$ times the area of the gross section, where F_y is the yield stress of the flange material. To qualify as hybrid girders, the flanges at any given section shall have the same cross-sectional area and be made of the same grade of steel.

1.10.2 Web

The ratio of the clear distance between flanges to the web thickness shall not exceed

$$\frac{14,000}{\sqrt{F_y(F_y + 16.5)}}$$

except that when transverse stiffeners are provided, spaced not more than $1\frac{1}{2}$ times the girder depth, the limiting ratio may be $2000/\sqrt{F_y}$, where F_y is the yield stress of the compression flange.

1.10.3 Flanges

The thickness of outstanding parts of flanges shall conform to the requirements of Sect. 1.9.1.2.

Flanges of welded plate girders may be varied in thickness or width by splicing a series of plates or by the use of cover plates.

The total cross-sectional area of cover plates of riveted girders shall not exceed 70 percent of the total flange area.

1.10.4 Flange Development

Rivets, high-strength bolts, or welds connecting flange to web, or cover plate to flange, shall be proportioned to resist the total horizontal shear resulting from the bending forces on the girder. The longitudinal distribution of these rivets, bolts, or intermittent welds shall be in proportion to the intensity of the shear. However, the longitudinal spacing shall not exceed the maximum permitted for compression or tension members in Sect. 1.18.2.3 or 1.18.3.1, respectively. Additionally, rivets or welds connecting flange to web shall be proportioned to transmit to the web any loads applied directly to the flange, unless provision is made to transmit such loads by direct bearing.

Partial length cover plates shall be extended beyond the theoretical cutoff point and the extended portion shall be attached to the beam or girder by rivets, high-strength bolts (friction-type connection), or fillet welds adequate, at the applicable stresses allowed in Sect. 1.5.2, 1.5.3, or 1.7, to develop the cover plate's portion of the flexural stresses in the beam or girder at the theoretical cutoff point. In addition, for welded cover plates, the welds connecting the cover plate termination to the beam or girder in the length a', defined below, shall be adequate, at the allowed stresses, to develop the cover plate's portion of the flexural stresses in the beam or girder at the distance a' from the end of the cover plate.** The length a', measured from the end of the cover plate, shall be:

---

* No limit is placed on the web stresses produced by the applied bending moment for which a hybrid girder is designed, except as provided in Sect. 1.7 and Appendix B.

** This may require the cover plate termination to be placed at a point in the beam or girder that has lower bending stress than the stress at the theoretical cutoff point.

1. A distance equal to the width of the cover plate when there is a continuous weld equal to or larger than $3/4$ of the plate thickness across the end of the plate and continuous welds along both edges of the cover plate in the length a'.

2. A distance equal to $1\frac{1}{2}$ times the width of the cover plate when there is a continuous weld smaller than $3/4$ of the plate thickness across the end of the plate and continuous welds along both edges of the cover plate in the length a'.

3. A distance equal to 2 times the width of the cover plate when there is no weld across the end of the plate, but continuous welds along both edges of the cover plate in the length a'.

1.10.5 Stiffeners

1.10.5.1 Bearing stiffeners shall be placed in pairs at unframed ends on the webs of plate girders and, where required,* at points of concentrated loads. Such stiffeners shall have a close bearing against the flange, or flanges, through which they receive their loads or reactions, and shall extend approximately to the edge of the flange plates or flange angles. They shall be designed as columns subject to the provisions of Sect. 1.5.1, assuming the column section to comprise the pair of stiffeners and a centrally located strip of the web whose width is equal to not more than 25 times its thickness at interior stiffeners or not more than 12 times its thickness when the stiffeners are located at the end of the web. The effective length shall be taken as not less than $3/4$ of the length of the stiffeners in computing the ratio l/r. Only that portion of the stiffener outside of the flange angle fillet or the flange-to-web welds shall be considered effective in bearing.

1.10.5.2 Except as hereinafter provided, the largest average web shear, f_v, in kips per square inch, computed for any condition of complete or partial loading, shall not exceed the value given by Formula (1.10-1).

$$F_v = \frac{F_y}{2.89}(C_v) \leq 0.40F_y \qquad (1.10\text{-}1)$$

where

$$C_v = \frac{45,000k}{F_y(h/t)^2} \quad \text{when } C_v \text{ is less than } 0.8$$

$$= \frac{190}{h/t}\sqrt{\frac{k}{F_y}} \quad \text{when } C_v \text{ is more than } 0.8$$

$$k = 4.00 + \frac{5.34}{(a/h)^2} \quad \text{when } a/h \text{ is less than } 1.0$$

$$= 5.34 + \frac{4.00}{(a/h)^2} \quad \text{when } a/h \text{ is more than } 1.0$$

t = thickness of web, inches
a = clear distance between transverse stiffeners, inches
h = clear distance between flanges at the section under investigation, inches

---

* For provisions governing welded plate girders, see Sect. 1.10.10.

Alternatively, for girders other than hybrid girders, if intermediate stiffeners are provided and spaced to satisfy the provisions of Sect. 1.10.5.3 and if $C_v \leq 1$, the allowable shear given by Formula (1.10-2) may be used in lieu of the value given by Formula (1.10-1).

$$F_v = \frac{F_y}{2.89}\left[C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}}\right] \leq 0.40F_y \qquad (1.10\text{-}2)^*$$

1.10.5.3 Subject to the limitations of Sect. 1.10.2, intermediate stiffeners are not required when the ratio h/t is less than 260 and the maximum web shear stress, f_v, is less than that permitted by Formula (1.10-1).

The spacing of intermediate stiffeners, where stiffeners are required, shall be such that the web shear stress will not exceed the value for F_v given by Formula (1.10-1) or (1.10-2), as applicable, and the ratio a/h shall not exceed $[260/(h/t)]^2$ nor 3.0.

In girders designed on the basis of tension field action, the spacing between stiffeners at end panels, at panels containing large holes, and at panels adjacent to panels containing large holes shall be such that f_v does not exceed the value given by Formula (1.10-1).

1.10.5.4 The moment of inertia of a pair of intermediate stiffeners, or a single intermediate stiffener, with reference to an axis in the plane of the web, shall not be less than $(h/50)^4$.

The gross area, in square inches, of intermediate stiffeners spaced as required for Formula (1.10-2) (*total* area, when stiffeners are furnished in pairs) shall be not less than that computed by Formula (1.10-3).

$$A_{st} = \frac{1 - C_v}{2}\left[\frac{a}{h} - \frac{(a/h)^2}{\sqrt{1 + (a/h)^2}}\right] YDht \qquad (1.10\text{-}3)$$

where

C_v, a, h, and t are as defined in Sect. 1.10.5.2
Y = ratio of yield stress of web steel to yield stress of stiffener steel
D = 1.0 for stiffeners furnished in pairs
  = 1.8 for single angle stiffeners
  = 2.4 for single plate stiffeners

When the greatest shear stress, f_v, in a panel is less than that permitted by Formula (1.10-2), this gross area requirement may be reduced in like proportion.

Intermediate stiffeners required by Formula (1.10-2) shall be connected for a total shear transfer, in kips per linear inch of single stiffener or pair of stiffeners, not less than that computed by Formula (1.10-4).

$$f_{vs} = h\sqrt{\left(\frac{F_y}{340}\right)^3} \qquad (1.10\text{-}4)$$

where F_y = yield stress of web steel.

This shear transfer may be reduced in the same proportion that the largest computed shear stress, f_v, in the adjacent panels is less than that permitted by Formula (1.10-2). However, rivets and welds in intermediate stiffeners which are required to transmit to the web an applied concentrated load or reaction shall be proportioned for not less than the applied load or reaction.

---

* Formula (1.10-2) recognizes the contribution of tension field action. For values of F_v provided by this formula, see Tables 11-36 and 11-50 in Appendix A.

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 closer than 4 times nor more than 6 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 1 percent of the total flange stress, unless the flange is composed only of angles.

Rivets connecting stiffeners to the girder web shall be spaced not more than 12 inches on center. 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 inches.

1.10.6   Reduction in Flange Stress

When the web depth-to-thickness ratio exceeds $760/\sqrt{F_b}$, the maximum bending stress in the compression flange shall not exceed

$$F'_b \leq F_b \left[1.0 - 0.0005 \frac{A_w}{A_f} \left(\frac{h}{t} - \frac{760}{\sqrt{F_b}} \right) \right] \qquad (1.10\text{-}5)$$

where

F_b = applicable bending stress given in Sect. 1.5.1.4, kips per square inch
A_w = area of web at the section under investigation, square inches
A_f = area of compression flange, square inches

The maximum stress in either flange of a hybrid girder shall not exceed the value given by Formula (1.10-5) nor

$$F'_b \leq F_b \left[\frac{12 + \left(\dfrac{A_w}{A_f} \right) (3\alpha - \alpha^3)}{12 + 2 \left(\dfrac{A_w}{A_f} \right)} \right] \qquad (1.10\text{-}6)$$

where α = ratio of web yield stress to flange yield stress.

1.10.7   Combined Shear and Tension Stress

Plate girder webs which depend upon tension field action, as provided in Formula (1.10-2), shall be so proportioned that bending tensile stress, due to moment in the plane of the girder web, shall not exceed $0.60F_y$ nor

$$\left(0.825 - 0.375 \frac{f_v}{F_v} \right) F_y \qquad (1.10\text{-}7)$$

where

f_v = computed average web shear stress (total shear divided by web area), kips per square inch
F_v = allowable web shear stress according to Formula (1.10-2), kips per square inch

The allowable shear stress in the webs of girders having A514 flanges and webs shall not exceed the values given by Formula (1.10-1) if the flexural stress in the flange, f_b, exceeds $0.75F_b$.

1.10.8   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 stresses at the point of splice.

1.10.9   Horizontal Forces

The flanges of plate girders supporting cranes or other moving loads shall be proportioned to resist the horizontal forces produced by such loads.   (See Sect. 1.3.4.)

1.10.10   Web Crippling

1.10.10.1   Webs of beams and welded plate girders shall be so proportioned that the compressive stress at the web toe of the fillets, resulting from concentrated loads not supported by bearing stiffeners, shall not exceed $0.75F_y$; otherwise, bearing stiffeners shall be provided.   The governing formulas shall be:

For interior loads:

$$\frac{R}{t(N + 2k)} \leq 0.75F_y \qquad\qquad (1.10\text{-}8)$$

For end-reactions:

$$\frac{R}{t(N + k)} \leq 0.75F_y \qquad\qquad (1.10\text{-}9)$$

where
R = concentrated load or reaction, kips
t = thickness of web, inches
N = length of bearing (not less than k for end reactions), inches
k = distance from outer face of flange to web toe of fillet, inches

1.10.10.2   Webs of plate girders shall also be so proportioned or stiffened that the sum of the compression stresses resulting from concentrated and distributed loads, bearing directly on or through a flange plate, upon the compression edge of the web plate, and not supported directly by bearing stiffeners, shall not exceed:

$$\left[5.5 + \frac{4}{(a/h)^2}\right] \frac{10{,}000}{(h/t)^2} \quad \text{kips per square inch} \qquad\qquad (1.10\text{-}10)$$

when the flange is restrained against rotation, nor

$$\left[2 + \frac{4}{(a/h)^2}\right] \frac{10{,}000}{(h/t)^2} \quad \text{kips per square inch} \qquad\qquad (1.10\text{-}11)$$

when the flange is not so restrained.

These stresses shall be computed as follows:

1. Concentrated loads, in kips, shall be divided by the product of the web thickness and either the girder depth or the length of panel in which the load is placed, whichever is the lesser panel dimension.

2. Distributed loads, in kips per linear inch of length, shall be divided by the web thickness.

1.10.11 Rotational Restraint at Points of Support

At points of support, beams, girders, and trusses shall be restrained against rotation about their longitudinal axis.

SECTION 1.11 COMPOSITE CONSTRUCTION

1.11.1 Definition

Composite construction shall consist of steel beams or girders supporting a reinforced concrete slab,* so interconnected that the beam and slab act together to resist bending. When the slab extends on both sides of the beam, the *effective width* of the concrete flange shall be taken as not more than $\frac{1}{4}$ the span of the beam, and its *effective projection* beyond the edge of the beam shall not be taken as more than $\frac{1}{2}$ the clear distance to the adjacent beam, nor more than 8 times the slab thickness. When the slab is present on only one side of the beam, the *effective projection* shall be taken as not more than $\frac{1}{12}$ of the beam span, nor 6 times its thickness, nor $\frac{1}{2}$ the clear distance to the adjacent beam.

Beams totally encased 2 inches or more on their sides and soffit in concrete cast integrally with the slab may be assumed to be interconnected to the concrete by natural bond, without additional anchorage, provided the top of the beam is at least $1\frac{1}{2}$ inches below the top and 2 inches above the bottom of the slab, and further provided that the encasement has adequate mesh or other reinforcing steel throughout the whole depth and across the soffit of the beam to prevent spalling of the concrete. When shear connectors are provided in accordance with Sect. 1.11.4, encasement of the beam to achieve composite action is not required.

1.11.2 Design Assumptions

1.11.2.1 Encased beams shall be proportioned to support, unassisted, all dead loads applied prior to the hardening of the concrete (unless these loads are supported temporarily on shoring) and, acting in conjunction with the slab, to support all dead and live loads applied after hardening of the concrete, without exceeding a computed bending stress of $0.66F_y$, where F_y is the yield stress of the steel beam. The bending stress produced by loads after the concrete has hardened shall be computed on the basis of the section properties of the composite section. Concrete tension stresses shall be neglected. Alternatively, the steel beam alone may be proportioned to resist, unassisted, the positive moment produced by all loads, live and dead, using a bending stress equal to $0.76F_y$, in which case temporary shoring is not required.

1.11.2.2 When shear connectors are used in accordance with Sect. 1.11.4, the composite section shall be proportioned to support all of the loads without exceeding the allowable stress prescribed in Sect. 1.5.1.4, even when the steel section is not shored during construction. In calculations involving composite sections in positive moment areas, the steel cross section is exempt from the compactness requirements of subparagraphs 2, 3, and 5 of Sect. 1.5.1.4.1.

Reinforcement parallel to the beam within the effective width of the slab, when anchored in accordance with the provisions of the applicable building code, may be included in computing the properties of composite sections, provided shear connectors are furnished in accordance with the requirements of Sect. 1.11.4. The section properties of the composite section shall be computed in accordance with the elastic theory. Concrete tension stresses shall be neglected. For stress

---

* See Commentary Sect. 1.11.1.

computations, the compression area of lightweight or normal weight concrete shall be treated as an equivalent area of steel by dividing it by the modular ratio, n, for normal weight concrete of the strength specified when determining the section properties. For deflection calculations, the transformed section properties shall be based on the appropriate modular ratio, n, for the strength and weight concrete specified, where $n = E/E_c$.

In cases where it is not feasible or necessary to provide adequate connectors to satisfy the horizontal shear requirements for full composite action, the effective section modulus shall be determined as

$$S_{eff} = S_s + \sqrt{\frac{V'_h}{V_h}} (S_{tr} - S_s) \qquad (1.11\text{-}1)$$

where

V_h and V'_h are as defined in Sect. 1.11.4

S_s = section modulus of the steel beam referred to its bottom flange, inches^3

S_{tr} = section modulus of the transformed composite section referred to its bottom flange, based upon maximum permitted effective width of concrete flange (Sect. 1.11.1), inches^3

For construction without temporary shoring, stress in the steel section may be computed from the total dead plus live load moment and the transformed section modulus, S_{tr}, provided that the numerical value of S_{tr} so used shall not exceed

$$S_{tr} = \left(1.35 + 0.35 \frac{M_L}{M_D} \right) S_s \qquad (1.11\text{-}2)*$$

In this expression for the limiting value of S_{tr}, M_L is the moment caused by loads applied subsequent to the time when the concrete has reached 75 percent of its required strength, M_D is the moment caused by loads applied prior to this time, and S_s is the section modulus of the steel beam referred to the flange where the stress is being computed. At sections subject to positive bending moment, the stress shall be computed for the steel tension flange. At sections subject to negative bending moment, the stress shall be computed for the steel tension and compression flanges. These stresses shall not exceed the appropriate value in Sect. 1.5.1. Section 1.5.6 shall not apply to stresses in the negative moment area computed under the provisions of this paragraph.

The actual section modulus of the transformed composite section shall be used in calculating the concrete flexural compression stress and, for construction without temporary shores, this stress shall be based upon loading applied after the concrete has reached 75 percent of its required strength. The stress in the concrete shall not exceed $0.45f'_c$.

1.11.3 End Shear

The web and the end connections of the steel beam shall be designed to carry the total reaction.

1.11.4 Shear Connectors

Except in the case of encased beams, as defined in Sect. 1.11.1, the entire horizontal shear at the junction of the steel beam and the concrete slab shall be assumed to be transferred by shear connectors welded to the top flange of the beam

---

* See Commentary Sect. 1.11.2.

and embedded in the concrete. For full composite action with concrete subject to flexural compression, the total horizontal shear to be resisted between the point of maximum positive moment and points of zero moment shall be taken as the smaller value using Formulas (1.11-3) and (1.11-4):

$$V_h = 0.85f'_c A_c/2 \qquad\qquad (1.11\text{-}3)*$$

and

$$V_h = A_s F_y/2 \qquad\qquad (1.11\text{-}4)$$

where

f'_c = specified compression strength of concrete, kips per square inch
A_c = actual area of effective concrete flange defined in Sect. 1.11.1, square inches
A_s = area of steel beam, square inches

In continuous composite beams where longitudinal reinforcing steel is considered to act compositely with the steel beam in the negative moment regions, the total horizontal shear to be resisted by shear connectors between an interior support and each adjacent point of contraflexure shall be taken as

$$V_h = A_{sr} F_{yr}/2 \qquad\qquad (1.11\text{-}5)$$

where

A_{sr} = total area of longitudinal reinforcing steel at the interior support located within the effective flange width specified in Sect. 1.11.1, square inches
F_{yr} = specified minimum yield stress of the longitudinal reinforcing steel, kips per square inch

For full composite action, the number of connectors resisting the horizontal shear, V_h , each side of the point of maximum moment, shall not be less than that determined by the relationship V_h/q, where q, the allowable shear load for one connector, is given in Table 1.11.4 for flat soffit concrete slabs made with ASTM C33 aggregates. For flat soffit concrete slabs made with rotary kiln produced aggregates, conforming to ASTM C330 with concrete unit weight not less than 90 pounds per cubic foot, the allowable shear load for one connector is obtained by multiplying the values from Table 1.11.4 by the coefficient from Table 1.11.4A.

For partial composite action with concrete subject to flexural compression, the horizontal shear, V'_h , to be used in computing S_{eff} shall be taken as the product of q times the number of connectors furnished between the point of maximum moment and the nearest point of zero moment.

The value of V'_h shall not be less than $1/4$ the smaller value of Formula (1.11-3), using the maximum permitted effective width of the concrete flange, or Formula (1.11-4). The effective moment of inertia for deflection computations shall be determined by:

$$I_{eff} = I_s + \sqrt{\frac{V'_h}{V_h}}\,(I_{tr} - I_s) \qquad\qquad (1.11\text{-}6)$$

where

I_s = moment of inertia of the steel beam, inches^4
I_{tr} = moment of inertia of the transformed composite section, inches^4

---

* The term $1/2\, A'_s F_{yr}$ should be added to the right-hand side of Formula (1.11-3) if longitudinal reinforcing steel with area A'_s located within the effective width of the concrete flange is included in the properties of the composite section.

TABLE 1.11.4

ALLOWABLE HORIZONTAL SHEAR LOAD FOR ONE CONNECTOR (q), KIPS[a]

Connector[b]	Specified Compressive Strength of Concrete (f'_c), ksi		
	3.0	3.5	≥4.0
$\frac{1}{2}''$ diam. x $2''$ hooked or headed stud	5.1	5.5	5.9
$\frac{5}{8}''$ diam. x $2\frac{1}{2}''$ hooked or headed stud	8.0	8.6	9.2
$\frac{3}{4}''$ diam. x $3''$ hooked or headed stud	11.5	12.5	13.3
$\frac{7}{8}''$ diam. x $3\frac{1}{2}''$ hooked or headed stud	15.6	16.8	18.0
Channel C3 x 4.1	$4.3w^c$	$4.7w^c$	$5.0w^c$
Channel C4 x 5.4	$4.6w^c$	$5.0w^c$	$5.3w^c$
Channel C5 x 6.7	$4.9w^c$	$5.3w^c$	$5.6w^c$

[a] Applicable only to concrete made with ASTM C33 aggregates.
[b] The allowable horizontal loads tabulated may also be used for studs longer than shown.
[c] w = length of channel, inches.

TABLE 1.11.4A

COEFFICIENTS FOR USE WITH CONCRETE MADE WITH C330 AGGREGATES

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

The connectors required each side of the point of maximum moment in an area of positive bending may be uniformly distributed between that point and adjacent points of zero moment, except that N_2, the number of shear connectors required between any concentrated load in that area and the nearest point of zero moment, shall be not less than that determined by Formula (1.11-7).

$$N_2 = \frac{N_1\left[\dfrac{M\beta}{M_{max}} - 1\right]}{\beta - 1} \qquad (1.11\text{-}7)$$

where

M = moment (less than the maximum moment) at a concentrated load point

N_1 = number of connectors required between point of maximum moment and point of zero moment, determined by the relationship V_h/q or V'_h/q, as applicable

β = $\dfrac{S_{tr}}{S_s}$ or $\dfrac{S_{eff}}{S_s}$, as applicable

For a continuous beam, connectors required in the region of negative bending may be uniformly distributed between the point of maximum moment and each point of zero moment.

Shear connectors shall have at least 1 inch of lateral concrete cover, except for connectors installed in the ribs of formed steel decks. Unless located directly over the web, the diameter of studs shall not be greater than $2\frac{1}{2}$ times the thickness

of the flange to which they are welded. The minimum center-to-center 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. The maximum center-to-center spacing of stud connectors shall not exceed 8 times the total slab thickness.

1.11.5 Composite Beams or Girders with Formed Steel Deck

Composite construction of concrete slabs on formed steel deck connected to steel beams or girders shall be designed by the applicable portions of Sects. 1.11.1 through 1.11.4, with the following modifications.

1.11.5.1 General

1. Section 1.11.5 is applicable to decks with nominal rib height not greater than 3 inches.

2. The average width of concrete rib or haunch, w_r , shall be not less than 2 inches, but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck. See Sect. 1.11.5.3, subparagraphs 2 and 3, for additional provisions.

3. The concrete slab shall be connected to the steel beam or girder with welded stud shear connectors $3/4$-inch or less in diameter (AWS D1.1-77, Section 4, Part F). Studs may be welded through the deck or directly to the steel member.

4. Stud shear connectors shall extend not less than $1\frac{1}{2}$ inches above the top of the steel deck after installation.

5. Total slab thickness, including ribs, shall be used in determining the effective width of concrete flange.

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

1.11.5.2 Deck Ribs Oriented Perpendicular to Steel Beam or Girder

1. Concrete below the top of the steel deck shall be neglected when determining section properties and in calculating A_c for Formula (1.11-3).

2. The spacing of stud shear connectors along the length of a supporting beam or girder shall not exceed 32 inches.

3. The allowable horizontal shear load per stud connector, q , shall be the value stipulated in Sect. 1.11.4 (Tables 1.11.4 and 1.11.4A) multiplied by the following reduction factor:

$$\left(\frac{0.85}{\sqrt{N_r}}\right)\left(\frac{w_r}{h_r}\right)\left(\frac{H_s}{h_r} - 1.0\right) \leq 1.0 \qquad (1.11\text{-}8)$$

where

h_r = nominal rib height, inches

H_s = length of stud connector after welding, inches, not to exceed the value $(h_r + 3)$ in computations, although the actual length may be greater

N_r = number of stud connectors on a beam in one rib, not to exceed 3 in computations, although more than 3 studs may be installed

w_r = average width of concrete rib, inches (see Sect. 1.11.5.1, subparagraph 2)

4. To resist uplift, the steel deck shall be anchored to all compositely designed steel beams or girders at a spacing not to exceed 16 inches. 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.

1.11.5.3 Deck Ribs Oriented Parallel to Steel Beam or Girder

1. Concrete below the top of the steel deck may be included when determining section properties and shall be included in calculating A_c for Formula (1.11-3).

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

3. When the nominal depth of steel deck is $1\frac{1}{2}$ inches or greater, the average width, w_r, of the supported haunch or rib shall be not less than 2 inches for the first stud in the transverse row plus 4 stud diameters for each additional stud.

4. The allowable horizontal shear load per stud connector, q, shall be the value stipulated in Sect. 1.11.4 (Tables 1.11.4 and 1.11.4A), except that when the ratio w_r/h_r is less than 1.5, the allowable load shall be multiplied by the following reduction factor:

$$0.6 \left(\frac{w_r}{h_r}\right) \left(\frac{H_s}{h_r} - 1.0\right) \leq 1.0 \qquad (1.11\text{-}9)$$

where h_r and H_s are as defined in Sect. 1.11.5.2 and w_r is the average width of concrete rib or haunch (see Sect. 1.11.5.1, subparagraph 2, and Sect. 1.11.5.3, subparagraph 3).

1.11.6 Special Cases

When composite construction does not conform to the requirements of Sects. 1.11.1 through 1.11.5, allowable load per shear connector must be established by a suitable test program.

SECTION 1.12 SIMPLE AND CONTINUOUS SPANS

1.12.1 Simple Spans

Beams, girders, and trusses shall ordinarily be designed on the basis of simple spans whose effective length is equal to the distance between centers of gravity of the members to which they deliver their end reactions.

1.12.2 End Restraint

When designed on the assumption of full or partial end restraint due to continuous, semi-continuous, or cantilever action, the beams, girders, and trusses, as well as the sections of the members to which they connect, shall be designed to carry the shears and moments so introduced, as well as all other forces, without exceeding at any point the unit stresses prescribed in Sect. 1.5.1, except that some non-elastic but self-limiting deformation of a part of the connection may be permitted when this is essential to the avoidance of overstressing of fasteners.

SECTION 1.13 DEFLECTIONS, VIBRATION, AND PONDING

1.13.1 Deflections

Beams and girders supporting floors and roofs shall be proportioned with due regard to the deflection produced by the design loads. Beams and girders supporting plastered ceilings shall be so proportioned that the maximum live load deflection does not exceed $\frac{1}{360}$ of the span.

1.13.2 Vibration

Beams and girders supporting large open floor areas free of partitions or other sources of damping, where transient vibration due to pedestrian traffic might not be acceptable, shall be designed with due regard for vibration.

1.13.3 Ponding

Unless a roof surface is provided with sufficient slope toward points of free drainage or adequate individual drains to prevent the accumulation of rainwater, the roof system shall be investigated by rational analysis to assure stability under ponding conditions, except as follows:

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

$$C_p + 0.9C_s \;\le\; 0.25 \ \text{ and } \ I_d \;\ge\; 25S^4/10^6$$

where

$$C_p \;=\; \frac{32L_sL_p{}^4}{10^7I_p} \ \text{ and } \ C_s \;=\; \frac{32SL_s{}^4}{10^7I_s}$$

L_p = column spacing in direction of girder, feet (length of primary members)

L_s = column spacing perpendicular to direction of girder, feet (length of secondary members)

S = spacing of secondary members, feet

I_p = moment of inertia of primary members, inches^4

I_s = moment of inertia of secondary members, inches^4

I_d = moment of inertia of the steel deck supported on secondary members, inches^4 per foot

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

Total bending stress due to dead loads, gravity live loads (if any), and ponding shall not exceed $0.80F_y$ for primary and secondary members. Stresses due to wind or seismic forces need not be included in a ponding analysis.

SECTION 1.14 GROSS AND NET AREAS

1.14.1 Definitions

The gross area of a member at any point shall be determined by summing the products of the thickness and the gross width of each element as measured normal to the axis of the member. The net area shall be determined by substituting for the gross width the net width computed in accordance with Sects. 1.14.2 to 1.14.5, inclusive.

1.14.2   Net Area and Effective Net Area

1.14.2.1   In the case of 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 of all the 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, inches

 g = transverse center-to-center spacing (gage) of the same two holes, inches

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.

1.14.2.2   The effective net area, A_e , of axially loaded tension members, where the load is transmitted by bolts or rivets through some but not all of the cross-sectional elements of the member,* shall be computed from the formula

$$A_e = C_t A_n$$

where

 A_n = net area of the member
 C_t = a reduction coefficient

Unless a larger coefficient can be justified by tests or other recognized criteria,* the following values of C_t shall be used in computations:

1.   W, M, or S shapes with flange widths not less than $\frac{2}{3}$ the depth, and structural tees cut from these shapes, provided the connection is to the flanges and has no fewer than 3 fasteners per line in the direction of stress . . . . $C_t = 0.90$

2.   W, M, or S shapes not meeting the conditions of subparagraph 1, structural tees cut from these shapes, and all other shapes, including built-up cross sections, provided the connection has not less than 3 fasteners per line in the direction of stress . . . . $C_t = 0.85$

3.   All members whose connections have only 2 fasteners per line in the direction of stress . . . . $C_t = 0.75$

1.14.2.3   Riveted and bolted splice and gusset plates and other connection fittings subject to tensile force shall be designed in accordance with the provisions of Sect. 1.5.1.1, where the effective net area shall be taken as the actual net area, except that, for the purpose of design calculations, it shall not be taken as greater than 85 percent of the gross area.

1.14.3   Angles

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

---

* See Commentary Sect. 1.14.2.2.

1.14.4   Size of Holes

In computing net area, the width of a rivet or bolt hole shall be taken as $\frac{1}{16}$-inch greater than the nominal dimension of the hole normal to the direction of applied stress.

1.14.5   Pin-Connected Members

Eyebars shall be of uniform thickness, without reinforcement at the pin holes. They shall have "circular" heads in which the periphery of the head beyond the pin hole is concentric with the pin hole.   The radius of transition between the circular head and the body of the eyebar shall be equal to or greater than the diameter of the head.

The width of the body of the eyebar shall not exceed 8 times its thickness, and the thickness shall not be less than $\frac{1}{2}$-inch.   The net area of the head through the pin hole, transverse to the axis of the eyebar, shall not be less than 1.33 nor more than 1.50 times the cross-sectional area of the body of the eyebar.   The diameter of the pin shall not be less than $\frac{7}{8}$ the width of the body of the eyebar.   The diameter of the pin hole shall not be more than $\frac{1}{32}$-inch greater than the diameter of the pin.   For steels having a yield stress greater than 70 ksi, the diameter of the pin hole shall not exceed 5 times the plate thickness.

In pin-connected plates other than eyebars, the tensile stress on the net area, transverse to the axis of the member, shall not exceed the stress allowed in Sect. 1.5.1.1 and the bearing stress on the projected area of the pin shall not exceed the stress allowed in Sect. 1.5.1.5.1.   The minimum net area beyond the pin hole, parallel to the axis of the member, shall not be less than $\frac{2}{3}$ of the net area across the pin hole.

The distance transverse to the axis of a pin-connected plate or any individual element of a built-up member, from the edge of the pin hole to the edge of the member or element, shall not exceed 4 times the thickness at the pin hole.   The diameter of the pin hole shall not be less than 1.25 times the smaller of the distances from the edge of the pin hole to the edge of a pin-connected plate or separated element of a built-up member at the pin hole.   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 the pin hole shall not be more than $\frac{1}{32}$-inch greater than the diameter of the pin.

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 allowable stress in bearing shall be no greater than as specified in Sect. 1.5.1.5.1.

1.14.6   Effective Areas of Weld Metal

1.14.6.1   Groove Welds

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.

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

TABLE 1.14.6.1.2

EFFECTIVE THROAT THICKNESS OF PARTIAL-PENETRATION GROOVE WELDS

Welding Process	Welding Position	Included Angle at Root of Groove	Effective Throat Thickness
Shielded metal arc or submerged arc	All	<60° but ≥45°	Depth of chamfer minus $\frac{1}{8}$-inch
		≥60°	Depth of chamfer
Gas metal arc or flux cored arc	All	≥60°	Depth of chamfer
	Horizontal or flat	<60° but ≥45°	Depth of chamfer
	Vertical or overhead	<60° but ≥45°	Depth of chamfer minus $\frac{1}{8}$-inch
Electrogas	All	≥60°	Depth of chamfer

TABLE 1.14.6.1.3

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 $R \geq 1$ inch.

1.14.6.1.2   The effective throat thickness of a partial-penetration groove weld shall be as shown in Table 1.14.6.1.2.

1.14.6.1.3   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 1.14.6.1.3. Random sections of production welds for each welding procedure, or such test sections as may be required by the Engineer, shall be used to verify that the effective throat is consistently obtained.

Larger effective throats than those in Table 1.14.6.1.3 are permitted, provided the fabricator can establish, by qualification, that he can consistently provide such larger effective throats. Qualification shall consist of sectioning the radiused member, normal to its axis, at mid-length and terminal ends of the weld. 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.

1.14.6.2   Fillet Welds

The effective area of fillet welds shall be considered 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 fillet, including returns.

The effective throat thickness of a fillet weld shall be the shortest distance from the root 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 $\frac{3}{8}$-inch and smaller fillet welds, and equal to the theoretical throat plus 0.11-inch for fillet welds over $\frac{3}{8}$-inch.

For fillet welds in holes and slots, the effective length shall be the length of the center line of the weld through the center of the plane through the throat. However, 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.

1.14.6.3   Plug and Slot Welds

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.

SECTION 1.15   CONNECTIONS

1.15.1   Minimum Connections

Connections carrying calculated stresses, except for lacing, sag bars, and girts, shall be designed to support not less than 6 kips.

1.15.2   Eccentric Connections

Axially stressed members meeting at a point shall have their gravity axes intersect at a point, if practicable; if not, provision shall be made for bending stresses due to the eccentricity.

1.15.3   Placement of Rivets, Bolts, and Welds

Except as hereinafter provided, groups of rivets, bolts, or welds at the ends of any member transmitting axial stress into that member shall have their centers of gravity on the gravity axis of the member, unless provision is made for the effect of the resulting eccentricity.   Except in members subject to repeated variation in stress, as defined in Sect. 1.7, disposition of fillet welds to balance the forces about the neutral axis or axes for end connections of single angle, double angle, and similar type members is not required.   Eccentricity between the gravity axes of such members and the gage lines for their riveted or bolted end connections may be neglected in statically loaded members, but should be considered in members subject to fatigue loading.

1.15.4   Unrestrained Members

Except as otherwise indicated by the designer, 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 action in the connection is permitted.

1.15.5   Restrained Members*

1.15.5.1   Fasteners or welds for end connections of 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.

---

* See Commentary Sect. 1.15.5.

1.15.5.2 When flanges or moment connection plates for end connections of beams and girders are welded to the flange of an I- or H-shape column, a pair of column-web stiffeners having a combined cross-sectional area, A_{st} , not less than that computed from Formula (1.15-1) shall be provided whenever the calculated value of A_{st} is positive.

$$A_{st} = \frac{P_{bf} - F_{yc}t(t_b + 5k)}{F_{yst}} \qquad (1.15\text{-}1)$$

where

F_{yc} = column yield stress, kips per square inch
F_{yst} = stiffener yield stress, kips per square inch
  k = distance between outer face of column flange and web toe of its fillet, if column is a rolled shape, or equivalent distance if column is a welded shape, inches
P_{bf} = the computed force delivered by the flange or moment connection plate multiplied by $\frac{5}{3}$, when the computed force is due to live and dead load only, or by $\frac{4}{3}$,* when the computed force is due to live and dead load in conjunction with wind or earthquake forces, kips
  t = thickness of column web, inches
t_b = thickness of flange or moment connection plate delivering concentrated force, inches

1.15.5.3 Notwithstanding the requirements of Sect. 1.15.5.2, a stiffener or a pair of stiffeners shall be provided opposite the compression flange when the column web depth clear of fillets, d_c , is greater than

$$\frac{4100\, t^3\sqrt{F_{yc}}}{P_{bf}} \qquad (1.15\text{-}2)$$

and a pair of stiffeners shall be provided opposite the tension flange when the thickness of the column flange, t_f , is less than

$$0.4\sqrt{\frac{P_{bf}}{F_{yc}}} \qquad (1.15\text{-}3)$$

1.15.5.4 Stiffeners required by the provisions of Sects. 1.15.5.2 and 1.15.5.3 shall comply with the following criteria:

1. The width of each stiffener plus $\frac{1}{2}$ the thickness of the column web shall be not less than $\frac{1}{3}$ the width of the flange or moment connection plate delivering the concentrated force.

2. The thickness of stiffeners shall be not less than $t_b/2$.**

3. When the concentrated force delivered occurs on only one column flange, the stiffener length need not exceed $\frac{1}{2}$ the column depth.

4. The weld joining stiffeners to the column web shall be sized to carry the force in the stiffener caused by unbalanced moments on opposite sides of the column.

1.15.5.5 Connections having high shear in the column web shall be investigated.[†]

---

  * Except where other codes may govern. For example, see Section 4(D) "Recommended Lateral Force Requirements and Commentary", Structural Engineers Assoc. of California, 1975.
 ** See Commentary Sect. 1.15.5 for comment on width-thickness ratio of stiffeners.
  [†] See Commentary Sect. 1.5.1.2.

1.15.6   Fillers

When rivets or bolts carrying computed stress pass through fillers thicker than $\frac{1}{4}$-inch, except in friction-type connections assembled with high-strength bolts, the fillers shall be extended beyond the splice material and the filler extension shall be secured by enough rivets or bolts 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.

In welded construction, any filler $\frac{1}{4}$-inch 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 stress, applied at the surface of the filler as an eccentric load. The welds joining the splice plate to the filler shall be sufficient to transmit the splice plate stress and shall be long enough to avoid overstressing the filler along the toe of the weld. Any filler less than $\frac{1}{4}$-inch 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 plate stress plus the thickness of the filler plate.

1.15.7   Connections of Tension and Compression Members in Trusses

The connections at ends of tension or compression members in trusses shall develop the force due to the design load, but not less than 50 percent of the effective strength of the member, based upon the kind of stress that governs the selection of the member.

1.15.8   Compression Members with Bearing Joints

Where columns bear on bearing plates, or are finished to bear at splices, there shall be sufficient rivets, bolts, or welds to hold all parts securely in place.

Where other compression members are finished to bear, the splice material and its riveting, bolting, or welding shall be arranged to hold all parts in line and shall be proportioned for 50 percent of the computed stress.

All of the foregoing joints shall be proportioned to resist any tension that would be developed by specified lateral forces acting in conjunction with 75 percent of the calculated dead load stress and no live load.

1.15.9   Combination of Welds

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

1.15.10   Rivets and Bolts in Combination with Welds

In new work, rivets, A307 bolts, or high-strength bolts used in bearing-type connections shall not be considered as sharing the stress in combination with welds. Welds, if used, shall be provided to carry the entire stress in the connection. High-strength bolts installed in a friction-type connection may be considered as sharing the stress with the welds.

In making welded alterations to structures, existing rivets and properly tightened high-strength bolts may be utilized for carrying stresses resulting from existing design loads, and the welding need be adequate only to carry all additional stress.

1.15.11 High-Strength Bolts (in Friction-Type Connections) in Combination with Rivets

In new work and in making alterations, rivets and high-strength bolts, installed in accordance with the provisions of Sect. 1.16.1 as friction-type connections, may be considered as sharing the stresses resulting from dead and live loads.

1.15.12 Field Connections

Rivets, high-strength bolts, or welds shall be used for the following connections:

> Column splices in all tier structures 200 feet or more in height.
>
> Column splices in tier structures 100 to 200 feet in height, if the least horizontal dimension is less than 40 percent of the height.
>
> Column splices in tier structures less than 100 feet in height, if the least horizontal dimension is less than 25 percent 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 feet 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 field connections may be made with A307 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.

SECTION 1.16 RIVETS AND BOLTS

1.16.1 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,* latest edition, as approved by the Research Council on Riveted and Bolted Structural Joints.

If required to be tightened to more than 50 percent of their minimum specified tensile strength, ASTM A449 bolts in tension and bearing-type shear connections shall have a hardened washer installed under the bolt head, and the nuts shall meet the requirements of ASTM A325.

1.16.2 Effective Bearing Area

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

1.16.3   Long Grips

Rivets and A307 bolts which carry calculated stress, and the grip of which exceeds 5 diameters, shall have their number increased 1 percent for each additional $1/16$-inch in the grip.

1.16.4   Minimum Spacing

1.16.4.1   The distance between centers of standard, oversized, or slotted fastener holes shall be not less than $2\frac{2}{3}d$,* where d is the nominal diameter of the fastener, inches, nor less than that required by Sect. 1.16.4.2, if applicable.

1.16.4.2   Along a line of transmitted force, the distance between centers of holes shall be not less than the following:

1.   Standard Holes:

$$2P/F_ut + d/2 \tag{1.16-1}$$

   where

   P = force transmitted by one fastener to the critical connected part, kips
   F_u = specified minimum tensile strength of the critical connected part, kips per square inch
   t = thickness of the critical connected part, inches

2.   Oversized and Slotted Holes:

   The distance required for standard holes in subparagraph 1, above, plus the applicable increment C_1 in Table 1.16.4.2, but the clear distance between holes shall not be less than one bolt diameter.

1.16.5   Minimum Edge Distance

1.16.5.1   The distance from the center of a standard hole to an edge of a connected part shall be not less than the applicable value in Table 1.16.5.1 nor the value from Sect. 1.16.5.2 or 1.16.5.3, as applicable.

1.16.5.2.   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

$$2P/F_ut \tag{1.16-2}$$

where P, F_u, and t are as defined in Sect. 1.16.4.2.

1.16.5.3   At end connections bolted to the web of a beam and designed for beam shear reaction only (without use of an analysis which accounts for the effects induced by fastener eccentricity), the distance from the center of the nearest standard hole to the end of the beam web shall be not less than

$$2P_R/F_ut \tag{1.16-3}$$

where P_R is the beam reaction, in kips, divided by the number of bolts, and F_u and t are as defined in Sect. 1.16.4.2.   Alternatively, the requirement of Formula (1.16-3) may be waived provided the bearing stress induced by the fastener is limited to not more than $0.90F_u$.

---

* A distance of $3d$ is preferred.   See Commentary Sect. 1.16.4.

TABLE 1.16.4.2

VALUES OF SPACING INCREMENT C_1 IN SECT. 1.16.4.2, INCHES

Nominal Diameter of Fastener (Inches)	Oversized 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	$1/4$	$1^{7}/16$
$\geq 1\frac{1}{8}$	$1/4$	0	$5/16$	$1\frac{1}{2}d - 1/16$

[a] When length of slot is less than maximum allowable (see Table 1.23.4), C_1 may be reduced by the difference between the maximum and actual slot lengths.

TABLE 1.16.5.1

MINIMUM EDGE DISTANCE, INCHES

(CENTER OF STANDARD HOLE[a] TO EDGE OF CONNECTED PART)

Nominal Rivet or Bolt Diameter (Inches)	At Sheared Edges	At Rolled Edges of Plates, Shapes or Bars or Gas Cut Edges[b]
$1/2$	$7/8$	$3/4$
$5/8$	$1\frac{1}{8}$	$7/8$
$3/4$	$1\frac{1}{4}$	1
$7/8$	$1\frac{1}{2}$[c]	$1\frac{1}{8}$
1	$1\frac{3}{4}$[c]	$1\frac{1}{4}$
$1\frac{1}{8}$	2	$1\frac{1}{2}$
$1\frac{1}{4}$	$2\frac{1}{4}$	$1\frac{5}{8}$
Over $1\frac{1}{4}$	$1\frac{3}{4} \times$ Diameter	$1\frac{1}{4} \times$ Diameter

[a] For oversized or slotted holes, see Sect. 1.16.5.4.

[b] All edge distances in this column may be reduced $1/8$-in. when the hole is at a point where stress does not exceed 25% of the maximum allowed stress in the element.

[c] These may be $1\frac{1}{4}$-in. at the ends of beam connection angles.

TABLE 1.16.5.4

VALUES OF EDGE DISTANCE INCREMENT C_2 IN SECT. 1.16.5.4, INCHES

Nominal Diameter of Fastener (Inches)	Oversized Holes	Slotted Holes		
		Perpendicular to Edge		Parallel to Edge
		Short Slots	Long Slots[a]	
$\leq 7/8$	$1/16$	$1/8$		
1	$1/8$	$1/8$	$3/4d$	0
$\geq 1\frac{1}{8}$	$1/8$	$3/16$		

[a] When length of slot is less than maximum allowable (see Table 1.23.4), C_2 may be reduced by one-half the difference between the maximum and actual slot lengths.

1.16.5.4 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 by Sect. 1.16.5.1, 1.16.5.2, or 1.16.5.3, as applicable, plus the applicable increment C_2 in Table 1.16.5.4.

1.16.6 Maximum Edge Distance

The maximum distance from the center of any rivet or bolt to the nearest edge of parts in contact with one another shall be 12 times the thickness of the connected part under consideration, but shall not exceed 6 inches.

SECTION 1.17 WELDS

1.17.1 General

All provisions of the *Structural Welding Code,* AWS D1.1-77, of the American Welding Society, except 2.3.2.4, 2.5, 8.13.1.2, and Section 9, as appropriate, apply to work performed under this Specification.

1.17.2 Minimum Size of Fillet Welds and Partial-Penetration Welds

The minimum size of fillet weld shall be as shown in Table 1.17.2A. The minimum effective throat thickness of a partial-penetration groove weld shall be as shown in Table 1.17.2B. Weld size is determined by the thicker of the two parts joined, except that the weld size need not exceed the thickness of the thinner part joined unless a larger size is required by calculated stress. For this exception, particular care shall be taken to provide sufficient preheat for soundness of the weld.

TABLE 1.17.2A

MINIMUM SIZE FILLET WELD

Material Thickness of Thicker Part Joined (Inches)	Minimum[a] Size of Fillet Weld (Inches)
To $\frac{1}{4}$ inclusive	$\frac{1}{8}$
Over $\frac{1}{4}$ to $\frac{1}{2}$	$\frac{3}{16}$
Over $\frac{1}{2}$ to $\frac{3}{4}$	$\frac{1}{4}$
Over $\frac{3}{4}$	$\frac{5}{16}$
[a] Leg dimension of fillet welds.	

TABLE 1.17.2B

MINIMUM EFFECTIVE THROAT THICKNESS OF PARTIAL-PENETRATION GROOVE WELD

Material Thickness of Thicker Part Joined (Inches)	Minimum Effective[a] Throat Thickness (Inches)
To $\frac{1}{4}$ inclusive	$\frac{1}{8}$
Over $\frac{1}{4}$ to $\frac{1}{2}$	$\frac{3}{16}$
Over $\frac{1}{2}$ to $\frac{3}{4}$	$\frac{1}{4}$
Over $\frac{3}{4}$ to $1\frac{1}{2}$	$\frac{5}{16}$
Over $1\frac{1}{2}$ to $2\frac{1}{4}$	$\frac{3}{8}$
Over $2\frac{1}{4}$ to 6	$\frac{1}{2}$
Over 6	$\frac{5}{8}$
[a] See Sect. 1.14.6.	

1.17.3 Maximum Size of Fillet Welds

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

1. Along edges of material less than $\frac{1}{4}$-inch thick, not greater than the thickness of the material.

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

1.17.4 Length of Fillet Welds

The minimum effective length* of a fillet weld 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 $\frac{1}{4}$ 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 shall not exceed 8 inches, unless the design otherwise prevents excessive transverse bending in the connection.

1.17.5 Intermittent Fillet Welds

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 $1\frac{1}{2}$ inches.

1.17.6 Lap Joints

The minimum amount of lap on lap joints shall be 5 times the thickness of the thinner part joined, but not less than 1 inch. 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.

1.17.7 End Returns of Fillet Welds

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. End returns shall be indicated on the design and detail drawings.

1.17.8 Fillet Welds in Holes and Slots

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. 1.14.6.2. Fillet welds in holes or slots are not to be considered plug or slot welds.

---

* See Sect. 1.14.6.2.

1.17.9 Plug and Slot Welds

Plug or slot welds may be used to transmit shear in a lap joint 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 $\frac{5}{16}$-inch, rounded to the next greater odd $\frac{1}{16}$-inch, nor greater than $2\frac{1}{4}$ times the thickness of the weld metal.

The minimum center-to-center spacing of plug welds shall be 4 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 $\frac{5}{16}$-inch, rounded to the next greater odd $\frac{1}{16}$-inch, nor shall it be greater than $2\frac{1}{4}$ 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 center-to-center 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 $\frac{5}{8}$-inch or less in thickness shall be equal to the thickness of the material. In material over $\frac{5}{8}$-inch in thickness, it shall be at least $\frac{1}{2}$ the thickness of the material but not less than $\frac{5}{8}$-inch.

SECTION 1.18 BUILT-UP MEMBERS

1.18.1 Open Box-Type Beams and Grillages

Where two or more rolled beams or channels are used side-by-side to form a flexural member, they shall be connected together at intervals of not more than 5 feet. Through-bolts and separators may be used, provided that, in beams having a depth of 12 inches or more, no fewer than 2 bolts shall be used at each separator location. When concentrated loads are carried from one beam to the other, or distributed between the beams, diaphragms having sufficient stiffness to distribute the load shall be riveted, bolted, or welded between the beams. Where beams are exposed, they shall be sealed against corrosion of interior surfaces or spaced sufficiently far apart to permit cleaning and painting.

1.18.2 Compression Members

1.18.2.1 All parts of built-up compression members and the transverse spacing of their lines of fasteners shall meet the requirements of Sects. 1.8 and 1.9.

1.18.2.2 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 rivets or bolts spaced longitudinally not more than 4 diameters apart for a distance equal to $1\frac{1}{2}$ times the maximum width of the member, or by continuous welds having a length not less than the maximum width of the member.

1.18.2.3 The longitudinal spacing for intermediate rivets, bolts, or intermittent welds in built-up members 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, the maximum spacing shall not exceed the thickness of the thinner outside plate times $127/\sqrt{F_y}$, nor shall it exceed 12 inches, when fasteners are provided on all gage lines at each section or when intermittent welds are provided along the edges of the components. 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 inches. The maximum longitudinal spacing of rivets, bolts, or intermittent welds connecting two rolled shapes in contact with one another shall not exceed 24 inches.

1.18.2.4   Compression members composed of two or more rolled shapes separated from one another by intermittent fillers shall be connected to one another at these fillers 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.

1.18.2.5   Open sides of compression members built up from plates or shapes shall be provided with lacing having 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 carrying calculated stress, 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 $\frac{1}{2}$ of this distance. The thickness of tie plates shall be not less than $\frac{1}{50}$ of the distance between the lines of fasteners or welds connecting them to the segments of the members. In riveted and bolted construction, the spacing in the direction of stress in tie plates shall be not more than 6 diameters and the tie plates shall be connected to each segment by at least 3 fasteners. In welded construction, the welding on each line connecting a tie plate shall aggregate not less than $\frac{1}{3}$ the length of the plate.

1.18.2.6   Lacing, including flat bars, angles, channels, or other shapes employed as lacing, shall be so spaced that the ratio l/r of the flange included between their connections shall not exceed the governing ratio for the member as a whole. Lacing shall be proportioned to resist a shearing stress normal to the axis of the member equal to 2 percent of the total compressive stress in the member. The ratio l/r 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. Lacing bars in compression may be treated as secondary members, with l being taken as the unsupported length of the lacing bar between fasteners or welds connecting it to the components of the built-up member for single lacing, and 70 percent of that distance for double lacing. The inclination of lacing bars to the axis of the member shall preferably be not less than 60 degrees for single lacing and 45 degrees for double lacing. When the distance between the lines of fasteners or welds in the flanges is more than 15 inches, the lacing shall preferably be double or be made of angles.

1.18.2.7   The function of tie plates and lacing may be performed by continuous cover plates perforated with a succession of access holes. The unsupported width of such plates at access holes, as defined in Sect. 1.9.2, is assumed available to resist axial stress, provided that: the width-to-thickness ratio conforms to the limitations of Sect. 1.9.2; the ratio of length (in direction of stress) to width of hole shall not exceed 2; the clear distance between holes in the di-

rection of stress shall be not less than the transverse distance between nearest lines of connecting fasteners or welds; and the periphery of the holes at all points shall have a minimum radius of $1\frac{1}{2}$ inches.

1.18.3 Tension Members

1.18.3.1 The longitudinal spacing of fasteners and intermittent fillet welds connecting a plate and a rolled shape in a built-up tension member, or two plate components in contact with one another, shall not exceed 24 times the thickness of the thinner plate nor 12 inches. The longitudinal spacing of fasteners and intermittent welds connecting two or more shapes in contact with one another in a tension member shall not exceed 24 inches. Tension members composed of two or more shapes or plates separated from one another by intermittent fillers shall be connected to one another at these fillers at intervals such that the slenderness ratio of either component between the fasteners does not exceed 240.

1.18.3.2 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 $\frac{2}{3}$ the distance between the lines of fasteners or welds connecting them to the components of the member. The thickness of such tie plates shall not be less than $\frac{1}{50}$ of the distance between these lines. The longitudinal spacing of fasteners or intermittent welds at tie plates shall not exceed 6 inches. The spacing of tie plates shall be such that the slenderness ratio of any component in the length between tie plates will not exceed 240.

SECTION 1.19 CAMBER

1.19.1 Trusses and Girders

Trusses of 80 feet or greater span should generally be cambered for approximately the dead load deflection. Crane girders of 75 feet or greater span should generally be cambered for approximately the dead load deflection plus $\frac{1}{2}$ the live load deflection.

1.19.2 Camber for Other Trades

If any special camber requirements are necessary in order 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 on the plans and on the detail drawings.

1.19.3 Erection

Beams and trusses detailed without specified camber shall be fabricated so that after erection any minor camber due to rolling or shop assembly shall be upward. If camber involves the erection of any member under a straining force, this shall be noted on the erection diagram.

SECTION 1.20 EXPANSION

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

SECTION 1.21 COLUMN BASES

1.21.1 Loads

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

1.21.2 Alignment

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

1.21.3 Finishing

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

1. Rolled steel bearing plates 2 inches or less in thickness may be used without milling,* provided a satisfactory contact bearing is obtained; rolled steel bearing plates over 2 inches but not over 4 inches in thickness may be straightened by pressing or, if presses are not available, by milling for all bearing surfaces (except as noted in subparagraph 3 of this Section), to obtain a satisfactory contact bearing; rolled steel bearing plates over 4 inches in thickness shall be milled for all bearing surfaces (except as noted in subparagraph 3 of this Section).

2. Column bases other than rolled steel bearing plates shall be milled for all bearing surfaces (except as noted in subparagraph 3 of this Section).

3. The bottom surfaces of bearing plates and column bases which are grouted to insure full bearing contact on foundations need not be milled.

SECTION 1.22 ANCHOR BOLTS

Anchor bolts shall be designed to provide resistance to all conditions of tension and shear at the bases of columns, including the net tensile components of any bending moments which may result from fixation or partial fixation of columns.

SECTION 1.23 FABRICATION

1.23.1 Cambering, Curving, and Straightening

The local application of heat or mechanical means may be used to introduce or correct camber, curvature, and straightness. The temperature of heated areas, as measured by approved methods, shall not exceed 1100°F for A514 steel nor 1200°F for other steels.

1.23.2 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; occasional notches or gouges not more than $\frac{3}{16}$-inch deep will be permitted. Notches or

---

* See Commentary Sect. 1.5.1.5.1.

TABLE 1.23.4

MAXIMUM SIZES[a] OF FASTENER HOLES, INCHES

Nominal Fastener Diameter (d)	Standard Hole Diameter	Oversized[b] Hole Diameter	Short-Slotted[b] Hole Dimensions	Long-Slotted[b] Hole Dimensions
$\leq 7/8$	$d + 1/16$	$d + 3/16$	$(d + 1/16) \times (d + 1/4)$	$(d + 1/16) \times 2\frac{1}{2}d$
1	$1\frac{1}{16}$	$1\frac{1}{4}$	$1\frac{1}{16} \times 1\frac{5}{16}$	$1\frac{1}{16} \times 2\frac{1}{2}$
$\geq 1\frac{1}{8}$	$d + 1/16$	$d + 5/16$	$(d + 1/16) \times (d + 3/8)$	$(d + 1/16) \times 2\frac{1}{2}d$
[a] Sizes are nominal.				
[b] Not permitted for riveted connections.				

gouges greater than $3/16$-inch that remain from cutting shall be removed by grinding. All re-entrant corners shall be shaped notch-free to a radius of at least $1/2$-inch.

1.23.3   Planing of Edges

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

1.23.4   Riveted and Bolted Construction—Holes

1.23.4.1   The maximum sizes of holes for rivets and bolts shall be as stipulated in Table 1.23.4, except that larger holes, required for tolerance on location of anchor bolts in concrete foundations, may be used in column base details.

1.23.4.2   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. Oversized and slotted holes shall not be used in riveted connections.

If the thickness of the material is not greater than the nominal diameter of the rivet or bolt plus $1/8$-inch, the holes may be punched. If the thickness of the material is greater than the nominal diameter of the rivet or bolt plus $1/8$-inch, the holes shall be either drilled from the solid, or sub-punched and reamed. The die for all sub-punched holes, and the drill for all sub-drilled holes, shall be at least $1/16$-inch smaller than the nominal diameter of the rivet or bolt. Holes in A514 steel plates over $1/2$-inch thick shall be drilled.

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

1.23.4.4   Short-slotted holes may be used in any or all plies of friction-type or bearing-type connections. The slots may be used without regard to direction of loading in friction-type 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.

TABLE 1.23.5

MINIMUM BOLT TENSION, KIPS[a]

Bolt Size, Inches	A325 Bolts	A490 Bolts
$1/2$	12	15
$5/8$	19	24
$3/4$	28	35
$7/8$	39	49
1	51	64
$1 1/8$	56	80
$1 1/4$	71	102
$1 3/8$	85	121
$1 1/2$	103	148

[a] Equal to 0.70 of specified minimum tensile strengths of bolts, rounded off to nearest kip.

1.23.4.5   Long-slotted holes may be used in only one of the connected parts of either a friction-type or bearing-type connection at an individual faying surface. Long-slotted holes may be used without regard to direction of loading in friction-type connections, but shall be normal to the direction of the load in bearing-type connections.   Where long-slotted holes are used on 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 to satisfy Specification provisions for use of high-strength bolts, the hardened washers shall be placed over the outer surface of the plate washer or bar.

1.23.5   Riveted and High-Strength-Bolted Construction—Assembling

All parts of riveted members shall be well pinned or bolted and rigidly held together while riveting.   Use of a drift pin in rivet or bolt holes during assembling shall not distort the metal or enlarge the holes.   Holes that must be enlarged to admit the rivets or bolts shall be reamed.   Poor matching of holes shall be cause for rejection.

Rivets shall be driven by power riveters, of either compression or manually-operated type, employing pneumatic, hydraulic, or electric power.   After driving, they shall be tight and their heads shall be in full contact with the surface.

Rivets shall ordinarily be hot-driven, in which case their finished heads shall be of approximately hemispherical shape and shall be of uniform size throughout the work for the same size rivet, full, neatly finished, and concentric with the holes. Hot-driven rivets shall be heated uniformly to a temperature not exceeding 1950° F;   they shall not be driven after their temperature has fallen below 1000° F.

Surfaces of high-strength-bolted parts in contact with the bolt head and nut shall not have a slope of more than 1:20 with respect to a plane normal to the bolt axis.   Where the surface of a high-strength-bolted part has a slope of more than 1:20, a beveled washer shall be used to compensate for the lack of parallelism. High-strength-bolted parts shall fit solidly together when assembled and shall not be separated by gaskets or any other interposed compressible materials.   When assembled, all joint surfaces, including those adjacent to the washers, shall be free of scale, except tight mill scale.   They shall be free of dirt, loose scale, burrs, and other defects that would prevent solid seating of the parts.   Contact surfaces

within friction-type connections shall be free of oil, paint, lacquer, or other coatings, except as listed in Appendix E.

All A325 and A490 bolts shall be tightened to a bolt tension not less than that given in Table 1.23.5. Tightening shall be done by the turn-of-nut method,* by a direct tension indicator, or by properly calibrated wrenches. Bolts tightened by means of a calibrated wrench shall be installed with a hardened washer under the nut or bolt head, whichever is the element turned in tightening. Hardened washers are not required when bolts are tightened by the turn-of-nut method, except that hardened washers are required under the nut and bolt head when A490 bolts are used to connect material having a specified yield point less than 40 kips per square inch.

1.23.6 Welded Construction

The technique of welding, the workmanship, appearance and quality of welds made, and the methods used in correcting nonconforming work shall be in accordance with "Section 3—Workmanship" and "Section 4—Technique" of the *Structural Welding Code,* AWS D1.1-77, of the American Welding Society.

1.23.7 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 to a common plane by milling, sawing, or other suitable means.

1.23.8 Dimensional Tolerances

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

SECTION 1.24 SHOP PAINTING

1.24.1 General Requirements

Shop painting and surface preparation shall be in accordance with the provisions of the *Code of Standard Practice,* latest edition, 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.

1.24.2 Inaccessible Surfaces

Except for contact surfaces, surfaces inaccessible after shop assembly shall be cleaned and painted prior to assembly, in accordance with job specifications.

1.24.3 Contact Surfaces

Paint is permitted unconditionally in bearing-type connections. Except where the design is based on special surface conditions meeting the requirements of Appendix E, shop contact surfaces shall be cleaned prior to assembly in accordance with the provisions of the *Code of Standard Practice,* latest edition, of

---

* See Commentary Sect. 1.23.5.

the American Institute of Steel Construction, but shall not be painted. Field contact surfaces and surfaces meeting the requirements of Appendix E shall be shop cleaned in accordance with job specifications, except as provided by Sect. 1.24.5.

1.24.4   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.

1.24.5   Surfaces Adjacent to Field Welds

Unless otherwise provided, surfaces within 2 inches of any field weld location shall be free of materials that would prevent proper welding or produce toxic fumes while welding is being done.

SECTION 1.25   ERECTION

1.25.1   Bracing

The frame of steel skeleton buildings shall be carried up true and plumb, within the limits defined in the *Code of Standard Practice,* latest edition, 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.

Wherever piles of material, erection equipment, or other loads are supported during erection, proper provision shall be made to take care of stresses resulting from such loads.

1.25.2   Adequacy of Temporary Connections

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

1.25.3   Alignment

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

1.25.4   Fit of Column Compression Joints

Lack of contact bearing not exceeding a gap of $\frac{1}{16}$-inch, regardless of the type of splice used (riveted, bolted, partial-penetration welded), shall be acceptable. If the gap exceeds $\frac{1}{16}$-inch, but is less than $\frac{1}{4}$-inch, 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.

1.25.5   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.

1.25.6 Field Painting

Responsibility for touch-up painting and cleaning, as well as for general painting, shall be allocated in accordance with accepted local practices, and this allocation shall be set forth explicitly in the contract.

SECTION 1.26   QUALITY CONTROL

1.26.1 General

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 information furnished to the bidders.

1.26.2 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 so schedule his work as to provide the minimum interruption to the work of the fabricator.

1.26.3 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.

1.26.4 Inspection of Welding

The inspection of welding shall be performed in accordance with the provisions of Section 6 of the *Structural Welding Code,* AWS D1.1-77, of the American Welding Society.

When non-destructive testing is required, the process, extent, technique, and standards of acceptance shall be clearly defined in information furnished to the bidders.

1.26.5 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 stress carrying 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

PART 2

SECTION 2.1 SCOPE

Subject to the limitations contained herein, simple and continuous beams, braced and unbraced planar rigid frames, and similar portions of structures rigidly constructed so as to be continuous over at least one interior support,* may be proportioned on the basis of plastic design, i.e., on the basis of their maximum strength. This strength, as determined by rational analysis, shall be not less than that required to support a factored load equal to 1.7 times the given live load and dead load, or 1.3 times these loads acting in conjunction with 1.3 times any specified wind or earthquake forces.

Rigid frames shall satisfy the requirements for Type 1 construction in the plane of the frame, as provided in Sect. 1.2. This does not preclude the use of some simple connections, provided that the provisions of Sect. 2.3 are satisfied. Type 2 construction is permitted for members between rigid frames. Connections joining a portion of a structure designed on the basis of plastic behavior with a portion not so designed need be no more rigid than ordinary seat-and-top-angle or ordinary web connections.

Where plastic design is used as the basis for proportioning continuous beams and structural frames, the provisions relating to allowable working stress, contained in Part 1, are waived. Except as modified by these rules, however, all other pertinent provisions of Part 1 shall govern.

It is not recommended that crane runways be designed continuous over interior vertical supports on the basis of maximum strength. However, rigid frame bents supporting crane runways may be considered as coming within the scope of the rules.

SECTION 2.2 STRUCTURAL STEEL

Structural steel shall conform to one of the following specifications, latest edition:

> *Structural Steel,* ASTM A36
>
> *High-Strength Low-Alloy Structural Steel,* ASTM A242
>
> *High-Strength Low-Alloy Structural Manganese Vanadium Steel,* ASTM A441
>
> *Structural Steel with 42,000 psi Minimum Yield Point,* ASTM A529
>
> *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

---

* As used here, "interior support" may be taken to include a rigid frame knee formed by the junction of a column and a sloping or horizontal beam or girder.

SECTION 2.3 BASIS FOR MAXIMUM STRENGTH DETERMINATION

For one- or two-story frames, the maximum strength may be determined by a routine plastic analysis procedure and the frame instability effect ($P\Delta$) may be ignored. For braced multistory frames, provisions should be made to include the frame instability effect in the design of bracing system and frame members. For unbraced multistory frames, the frame instability effect should be included directly in the calculations for maximum strength.

2.3.1 Stability of Braced Frames

The vertical bracing system for a plastically designed braced multistory frame shall be adequate, as determined by a rational analysis, to:

1. Prevent buckling of the structure under factored gravity loads
2. Maintain the lateral stability of the structure, including the overturning effects of drift, under factored gravity plus factored horizontal loads

The vertical bracing system may be considered to function together with in-plane shear-resisting exterior and interior walls, floor slabs, and roof decks, if these walls, slabs, and decks are 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 stability. Axial deformation of all members in the vertical bracing system shall be included in the lateral stability analysis. The axial force in these members caused by factored gravity plus factored horizontal loads shall not exceed $0.85P_y$, where P_y is the product of yield stress times the profile area of the member.

Girders and beams included in the vertical bracing system of a braced multistory frame shall be proportioned for axial force and moment caused by the concurrent factored horizontal and gravity loads, in accordance with Formula (2.4-2), with P_{cr} taken as the maximum axial strength of the beam, based on the actual slenderness ratio between braced points in the plane of bending.

2.3.2 Stability of Unbraced Frames

The strength of an unbraced multistory frame shall be determined by a rational analysis which includes the effect of frame instability and column axial deformation. Such a frame shall be designed to be stable under (1) factored gravity loads and (2) factored gravity loads plus factored horizontal loads. The axial force in the columns at factored load levels shall not exceed $0.75P_y$.

SECTION 2.4 COLUMNS

In the plane of bending of columns which would develop a plastic hinge at ultimate loading, the slenderness ratio l/r shall not exceed C_c, defined in Sect. 1.5.1.3.

The maximum strength of an axially loaded compression member shall be taken as

$$P_{cr} = 1.7AF_a \qquad (2.4-1)$$

where A is the gross area of the member and F_a, as defined by Formula (1.5-1), is based upon the applicable slenderness ratio.*

---

* See Commentary Sect. 2.4.

Members subject to combined axial load and bending moment shall be proportioned to satisfy the following interaction formulas:

$$\frac{P}{P_{cr}} + \frac{C_m M}{\left(1 - \dfrac{P}{P_e}\right) M_m} \leq 1.0 \tag{2.4-2}$$

$$\frac{P}{P_y} + \frac{M}{1.18 M_p} \leq 1.0; \quad M \leq M_p \tag{2.4-3}$$

in which

M = maximum factored moment, kip-feet
P = applied axial load, kips
P_e = Euler buckling load, kips
  = $(23/12) A F'_e$, where F'_e is as defined in Sect. 1.6.1
C_m = coefficient defined in Sect. 1.6.1
M_m = maximum moment that can be resisted by the member in the absence of axial load, kip-feet
M_p = plastic moment, kip-feet
  = $Z F_y$
Z = plastic section modulus, inches^3

For columns braced in the weak direction:

$$M_m = M_{px}$$

For columns unbraced in the weak direction:

$$M_m = \left[1.07 - \frac{(l/r_y)\sqrt{F_y}}{3160} \right] M_{px} \leq M_{px} \tag{2.4-4}$$

SECTION 2.5   SHEAR

Unless reinforced by diagonal stiffeners or a doubler plate, the webs of columns, beams, and girders, including areas within the boundaries of the connections, shall be so proportioned that

$$V_u \leq 0.55 F_y t d \tag{2.5-1}$$

where

V_u = shear that would be produced by the required factored loading, kips
d = depth of the member, inches
t = web thickness, inches

SECTION 2.6   WEB CRIPPLING

Web stiffeners are required on a member at a point of load application where a plastic hinge would form.

At points on a member where the concentrated load delivered by the flanges of a member framing into it would produce web crippling opposite the compression flange or high tensile stress in the connection of the tension flange, web stiffeners are required in accordance with the provisions of Sect. 1.15.5.

SECTION 2.7   MINIMUM THICKNESS
(WIDTH-THICKNESS RATIOS)

The width-thickness ratio for flanges of rolled W, M, or S shapes and similar built-up single-web shapes that would be subjected to compression involving hinge rotation under ultimate loading shall not exceed the following values:

F_y	$b_f/2t_f$
36	8.5
42	8.0
45	7.4
50	7.0
55	6.6
60	6.3
65	6.0

The thickness of sloping flanges may be taken as their average thickness.

The width-thickness ratio of similarly compressed flange plates in box sections and cover plates shall not exceed $190/\sqrt{F_y}$. For this purpose the width of a cover plate shall be taken as the distance between longitudinal lines of connecting rivets, high-strength bolts, or welds.

The depth-thickness ratio of webs of members subjected to plastic bending shall not exceed the value given by Formula (2.7-1a) or (2.7-1b), as applicable.

$$\frac{d}{t} = \frac{412}{\sqrt{F_y}}\left(1 - 1.4\frac{P}{P_y}\right) \quad \text{when} \quad \frac{P}{P_y} \leq 0.27 \qquad (2.7\text{-}1a)$$

$$\frac{d}{t} = \frac{257}{\sqrt{F_y}} \quad \text{when} \quad \frac{P}{P_y} > 0.27 \qquad (2.7\text{-}1b)$$

SECTION 2.8   CONNECTIONS

All connections, the rigidity of which is essential to the continuity assumed as the basis of the analysis, shall be capable of resisting the moments, shears, and axial loads to which they would be subjected by the full factored loading, or any probable partial distribution thereof.

Corner connections (haunches) that are tapered or curved for architectural reasons shall be so proportioned that the full plastic bending strength of the section adjacent to the connection can be developed, if required.

Stiffeners shall be used, as required, to preserve the flange continuity of interrupted members at their junction with other members in a continuous frame. Such stiffeners shall be placed in pairs on opposite sides of the web of the member which extends continuously through the joint.

High-strength bolts, A307 bolts, rivets, and welds shall be proportioned to resist the forces produced at factored load, using stresses equal to 1.7 times those given in Part 1. In general, groove welds are preferable to fillet welds, but their use is not mandatory.

High-strength bolts may be used in joints having painted contact surfaces when these joints are of such size that the slip required to produce bearing would not interfere with the formation, at factored loading, of the plastic hinges assumed in the design.

SECTION 2.9 LATERAL BRACING

Members shall be adequately braced to resist lateral and torsional displacements at the plastic hinge locations associated with the failure mechanism. The laterally unsupported distance, l_{cr} , from such braced hinge locations to similarly braced adjacent points on the member or frame shall not exceed the value determined from Formula (2.9-1a) or (2.9-1b), as applicable.

$$\frac{l_{cr}}{r_y} = \frac{1375}{F_y} + 25 \quad \text{when} \quad +1.0 > \frac{M}{M_p} > -0.5 \qquad (2.9\text{-}1a)$$

$$\frac{l_{cr}}{r_y} = \frac{1375}{F_y} \quad \text{when} \quad -0.5 \geq \frac{M}{M_p} > -1.0 \qquad (2.9\text{-}1b)$$

where

r_y = radius of gyration of the member about its weak axis, inches
M = lesser of the moments at the ends of the unbraced segment, kip-feet
M/M_p = end moment ratio, positive when the segment is bent in reverse curvature and negative when bent in single curvature

The foregoing provisions need not apply in the region of the last hinge to form in the failure mechanism assumed as the basis for proportioning a given member, nor in members oriented with their weak axis normal to the plane of bending. However, in the region of the last hinge to form, and in regions not adjacent to a plastic hinge, the maximum distance between points of lateral support shall be such as to satisfy the requirements of Formulas (1.5-6a), (1.5-6b) or (1.5-7), as well as Formulas (1.6-1a) and (1.6-1b), in Part 1 of this Specification. For this case, the values of f_a and f_b shall be computed from the moment and axial force at factored loading, divided by the applicable load factor.

Members built into a masonry wall and having their web perpendicular to this wall can be assumed to be laterally supported with respect to their weak axis of bending.

SECTION 2.10 FABRICATION

The provisions of Part 1 with respect to workmanship shall govern the fabrication of structures, or portions of structures, designed on the basis of maximum strength, subject to the following limitations:

1. The use of sheared edges shall be avoided in locations subject to plastic hinge rotation at factored loading. If used, they shall be finished smooth by grinding, chipping, or planing.

2. In locations subject to plastic hinge rotation at factored loading, holes for rivets or bolts in the tension area shall be sub-punched and reamed or drilled full size.

APPENDIX A

Tables of Numerical Equivalents

LIST OF TABLES—APPENDIX A

TABLE 1

ALLOWABLE STRESS AS A FUNCTION OF F_y

F_y (ksi)	Allowable Stress (ksi)					
	$0.4F_y$ [b,f]	$0.45F_y$ [a]	$0.60F_y$ [a,c,d]	$0.66F_y$ [d]	$0.75F_y$ [c]	$0.90F_y$ [e]
33	13.2	14.9	19.8	21.8	24.8	29.7
35	14.0	15.8	21.0	23.1	26.3	31.5
36	14.5	16.2	22.0	24.0	27.0	32.4
40	16.0	18.0	24.0	26.4	30.0	36.0
42	16.8	18.9	25.2	27.7	31.5	37.8
45	18.0	20.3	27.0	29.7	33.8	40.5
46	18.4	20.7	27.6	30.4	34.5	41.4
50	20.0	22.5	30.0	33.0	37.5	45.0
55	22.0	24.8	33.0	36.3	41.3	49.5
60	24.0	27.0	36.0	39.6	45.0	54.0
65	26.0	29.3	39.0	42.9	48.8	58.5
70	28.0	31.5	42.0	46.2	52.5	63.0
90	36.0	40.5	54.0			81.0
100	40.0	45.0	60.0			90.0

[a] See Sect. 1.5.1.1 Tension
[b] See Sect. 1.5.1.2 Shear
[c] See Sect. 1.5.1.3 Compression
[d] See Sect. 1.5.1.4 Bending
[e] See Sect. 1.5.1.5 Bearing
[f] See Sect. 1.10.5.2 Shear in Plate Girders

TABLE 2

ALLOWABLE STRESS AS A FUNCTION OF F_u

Item	ASTM Designation	F_y (ksi)	F_u (ksi)	Allowable Stress (ksi)				
				Connected Part of Designated Steel		Bolt or Threaded Part of Designated Steel		
				Tension $0.5F_u$[a]	Bearing $1.5F_u$[b]	Tension $0.33F_u$[c]	Shear $0.17F_u$[d]	Shear $0.22F_u$[e]
Shapes, Plates, Bars, Sheet and Tubing, or Threaded Parts.	A36	36	58—80	29.0	87.0	19.1	9.9	12.8
	A53	35	60	30.0	90.0	—	—	—
	A242	50	70	35.0	105	23.1	11.9	15.4
	A441	46	67	33.5	101	22.1	11.4	14.7
	A588	42	63	31.5	94.5	20.8	10.7	13.9
		40[f]	60	30.0	90.0	19.8	10.2	13.2
	A500	33/39[g]	45	22.5	67.5	—	—	—
		42/46[g]	58	29.0	87.0	—	—	—
		46/50[g]	62	31.0	93.0	—	—	—
	A501	36	58	29.0	87.0	—	—	—
	A529	42	60—85	30.0	90.0	19.8	10.2	13.2
	A570	40	55	27.5	82.5	—	—	—
		42	58	29.0	87.0	—	—	—
	A572	42	60	30.0	90.0	19.8	10.2	13.2
		50	65	32.5	97.5	21.5	11.1	14.3
		60	75	37.5	113	24.8	12.8	16.5
		65	80	40.0	120	26.4	13.6	17.6
	A514	100	110—130	55.0	165	36.3	18.7	24.2
		90	100—130	50.0	150	33.0	17.0	22.0
	A606	45	65	32.5	97.5	—	—	—
		50	70	35.0	105	—	—	—
	A607	45	60	30.0	90.0	—	—	—
		50	65	32.5	97.5	—	—	—
		55	70	35.0	105	—	—	—
		60	75	37.5	113	—	—	—
		65	80	40.0	120	—	—	—
		70	85	42.5	128	—	—	—
	A618	50	70	35.0	105	—	—	—
		50	65	32.5	97.5	—	—	—
Bolts	A449	92	120	—	—	39.6	20.4	26.4
		81	105	—	—	34.7	17.9	23.1
		58	90	—	—	29.7	15.3	19.8

[a] On effective net area, see Sect. 1.5.1.1.
[b] Produced by fastener in shear, see Sect. 1.5.1.5.3. Note that smaller maximum allowable bearing stresses, as a function of hole spacing, may be required by Sect. 1.16.
[c] On nominal body area, see Table 1.5.2.1.
[d] Threads *not* excluded from shear plane, see Table 1.5.2.1.
[e] Threads excluded from shear plane, see Table 1.5.2.1.
[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

ALLOWABLE STRESS
FOR COMPRESSION MEMBERS OF 36 KSI SPECIFIED YIELD STRESS STEEL

$F_y = 36$ ksi

Main and Secondary Members Kl/r not over 120						Main Members Kl/r 121 to 200				Secondary Members[a] l/r 121 to 200			
$\frac{Kl}{r}$	F_a (ksi)	$\frac{Kl}{r}$	F_a (ksi)	$\frac{Kl}{r}$	F_a (ksi)	$\frac{Kl}{r}$	F_a (ksi)	$\frac{Kl}{r}$	F_a (ksi)	$\frac{l}{r}$	F_{as} (ksi)	$\frac{l}{r}$	F_{as} (ksi)
1	21.56	41	19.11	81	15.24	121	10.14	161	5.76	121	10.19	161	7.25
2	21.52	42	19.03	82	15.13	122	9.99	162	5.69	122	10.09	162	7.20
3	21.48	43	18.95	83	15.02	123	9.85	163	5.62	123	10.00	163	7.16
4	21.44	44	18.86	84	14.90	124	9.70	164	5.55	124	9.90	164	7.12
5	21.39	45	18.78	85	14.79	125	9.55	165	5.49	125	9.80	165	7.08
6	21.35	46	18.70	86	14.67	126	9.41	166	5.42	126	9.70	166	7.04
7	21.30	47	18.61	87	14.56	127	9.26	167	5.35	127	9.59	167	7.00
8	21.25	48	18.53	88	14.44	128	9.11	168	5.29	128	9.49	168	6.96
9	21.21	49	18.44	89	14.32	129	8.97	169	5.23	129	9.40	169	6.93
10	21.16	50	18.35	90	14.20	130	8.84	170	5.17	130	9.30	170	6.89
11	21.10	51	18.26	91	14.09	131	8.70	171	5.11	131	9.21	171	6.85
12	21.05	52	18.17	92	13.97	132	8.57	172	5.05	132	9.12	172	6.82
13	21.00	53	18.08	93	13.84	133	8.44	173	4.99	133	9.03	173	6.79
14	20.95	54	17.99	94	13.72	134	8.32	174	4.93	134	8.94	174	6.76
15	20.89	55	17.90	95	13.60	135	8.19	175	4.88	135	8.86	175	6.73
16	20.83	56	17.81	96	13.48	136	8.07	176	4.82	136	8.78	176	6.70
17	20.78	57	17.71	97	13.35	137	7.96	177	4.77	137	8.70	177	6.67
18	20.72	58	17.62	98	13.23	138	7.84	178	4.71	138	8.62	178	6.64
19	20.66	59	17.53	99	13.10	139	7.73	179	4.66	139	8.54	179	6.61
20	20.60	60	17.43	100	12.98	140	7.62	180	4.61	140	8.47	180	6.58
21	20.54	61	17.33	101	12.85	141	7.51	181	4.56	141	8.39	181	6.56
22	20.48	62	17.24	102	12.72	142	7.41	182	4.51	142	8.32	182	6.53
23	20.41	63	17.14	103	12.59	143	7.30	183	4.46	143	8.25	183	6.51
24	20.35	64	17.04	104	12.47	144	7.20	184	4.41	144	8.18	184	6.49
25	20.28	65	16.94	105	12.33	145	7.10	185	4.36	145	8.12	185	6.46
26	20.22	66	16.84	106	12.20	146	7.01	186	4.32	146	8.05	186	6.44
27	20.15	67	16.74	107	12.07	147	6.91	187	4.27	147	7.99	187	6.42
28	20.08	68	16.64	108	11.94	148	6.82	188	4.23	148	7.93	188	6.40
29	20.01	69	16.53	109	11.81	149	6.73	189	4.18	149	7.87	189	6.38
30	19.94	70	16.43	110	11.67	150	6.64	190	4.14	150	7.81	190	6.36
31	19.87	71	16.33	111	11.54	151	6.55	191	4.09	151	7.75	191	6.35
32	19.80	72	16.22	112	11.40	152	6.46	192	4.05	152	7.69	192	6.33
33	19.73	73	16.12	113	11.26	153	6.38	193	4.01	153	7.64	193	6.31
34	19.65	74	16.01	114	11.13	154	6.30	194	3.97	154	7.59	194	6.30
35	19.58	75	15.90	115	10.99	155	6.22	195	3.93	155	7.53	195	6.28
36	19.50	76	15.79	116	10.85	156	6.14	196	3.89	156	7.48	196	6.27
37	19.42	77	15.69	117	10.71	157	6.06	197	3.85	157	7.43	197	6.26
38	19.35	78	15.58	118	10.57	158	5.98	198	3.81	158	7.39	198	6.24
39	19.27	79	15.47	119	10.43	159	5.91	199	3.77	159	7.34	199	6.23
40	19.19	80	15.36	120	10.28	160	5.83	200	3.73	160	7.29	200	6.22

[a] K taken as 1.0 for secondary members.

Note: $C_c = 126.1$

TABLE 3-50

ALLOWABLE STRESS

FOR COMPRESSION MEMBERS OF 50 KSI SPECIFIED YIELD STRESS STEEL

Main and Secondary Members Kl/r not over 120						Main Members[b] Kl/r 121 to 200				Secondary Members[a,b] l/r 121 to 200			
$\frac{Kl}{r}$	F_a (ksi)	$\frac{Kl}{r}$	F_a (ksi)	$\frac{Kl}{r}$	F_a (ksi)	$\frac{Kl}{r}$	F_a (ksi)	$\frac{Kl}{r}$	F_a (ksi)	$\frac{l}{r}$	F_{as} (ksi)	$\frac{l}{r}$	F_{as} (ksi)
1	29.94	41	25.69	81	18.81	121	10.20	161	5.76	121	10.25	161	7.25
2	29.87	42	25.55	82	18.61	122	10.03	162	5.69	122	10.13	162	7.20
3	29.80	43	25.40	83	18.41	123	9.87	163	5.62	123	10.02	163	7.16
4	29.73	44	25.26	84	18.20	124	9.71	164	5.55	124	9.91	164	7.12
5	29.66	45	25.11	85	17.99	125	9.56	165	5.49	125	9.80	165	7.08
6	29.58	46	24.96	86	17.79	126	9.41	166	5.42	126	9.70	166	7.04
7	29.50	47	24.81	87	17.58	127	9.26	167	5.35	127	9.59	167	7.00
8	29.42	48	24.66	88	17.37	128	9.11	168	5.29	128	9.49	168	6.96
9	29.34	49	24.51	89	17.15	129	8.97	169	5.23	129	9.40	169	6.93
10	29.26	50	24.35	90	16.94	130	8.84	170	5.17	130	9.30	170	6.89
11	29.17	51	24.19	91	16.72	131	8.70	171	5.11	131	9.21	171	6.85
12	29.08	52	24.04	92	16.50	132	8.57	172	5.05	132	9.12	172	6.82
13	28.99	53	23.88	93	16.29	133	8.44	173	4.99	133	9.03	173	6.79
14	28.90	54	23.72	94	16.06	134	8.32	174	4.93	134	8.94	174	6.76
15	28.80	55	23.55	95	15.84	135	8.19	175	4.88	135	8.86	175	6.73
16	28.71	56	23.39	96	15.62	136	8.07	176	4.82	136	8.78	176	6.70
17	28.61	57	23.22	97	15.39	137	7.96	177	4.77	137	8.70	177	6.67
18	28.51	58	23.06	98	15.17	138	7.84	178	4.71	138	8.62	178	6.64
19	28.40	59	22.89	99	14.94	139	7.73	179	4.66	139	8.54	179	6.61
20	28.30	60	22.72	100	14.71	140	7.62	180	4.61	140	8.47	180	6.58
21	28.19	61	22.55	101	14.47	141	7.51	181	4.56	141	8.39	181	6.56
22	28.08	62	22.37	102	14.24	142	7.41	182	4.51	142	8.32	182	6.53
23	27.97	63	22.20	103	14.00	143	7.30	183	4.46	143	8.25	183	6.51
24	27.86	64	22.02	104	13.77	144	7.20	184	4.41	144	8.18	184	6.49
25	27.75	65	21.85	105	13.53	145	7.10	185	4.36	145	8.12	185	6.46
26	27.63	66	21.67	106	13.29	146	7.01	186	4.32	146	8.05	186	6.44
27	27.52	67	21.49	107	13.04	147	6.91	187	4.27	147	7.99	187	6.42
28	27.40	68	21.31	108	12.80	148	6.82	188	4.23	148	7.93	188	6.40
29	27.28	69	21.12	109	12.57	149	6.73	189	4.18	149	7.87	189	6.38
30	27.15	70	20.94	110	12.34	150	6.64	190	4.14	150	7.81	190	6.36
31	27.03	71	20.75	111	12.12	151	6.55	191	4.09	151	7.75	191	6.35
32	26.90	72	20.56	112	11.90	152	6.46	192	4.05	152	7.69	192	6.33
33	26.77	73	20.38	113	11.69	153	6.38	193	4.01	153	7.64	193	6.31
34	26.64	74	20.19	114	11.49	154	6.30	194	3.97	154	7.59	194	6.30
35	26.51	75	19.99	115	11.29	155	6.22	195	3.93	155	7.53	195	6.28
36	26.38	76	19.80	116	11.10	156	6.14	196	3.89	156	7.48	196	6.27
37	26.25	77	19.61	117	10.91	157	6.06	197	3.85	157	7.43	197	6.26
38	26.11	78	19.41	118	10.72	158	5.98	198	3.81	158	7.39	198	6.24
39	25.97	79	19.21	119	10.55	159	5.91	199	3.77	159	7.34	199	6.23
40	25.83	80	19.01	120	10.37	160	5.83	200	3.73	160	7.29	200	6.22

[a] K taken as 1.0 for secondary members.

[b] Values also applicable for steel of any yield stress ≥ 39 ksi.

Note: $C_c = 107.0$

TABLE 4

VALUES OF C_a

For Determining Allowable Stress for Main and Secondary Members When $Kl/r \leq C_c$ for Steel of Any Yield Stress (by equation $F_a = C_a F_y$)[a]

All grades of steel

$\dfrac{Kl/r}{C_c}$	C_a	$\dfrac{Kl/r}{C_c}$	C_a	$\dfrac{Kl/r}{C_c}$	C_a	$\dfrac{Kl/r}{C_c}$	C_a
.01	.599	.26	.548	.51	.472	.76	.375
.02	.597	.27	.546	.52	.469	.77	.371
.03	.596	.28	.543	.53	.465	.78	.366
.04	.594	.29	.540	.54	.462	.79	.362
.05	.593	.30	.538	.55	.458	.80	.357
.06	.591	.31	.535	.56	.455	.81	.353
.07	.589	.32	.532	.57	.451	.82	.348
.08	.588	.33	.529	.58	.447	.83	.344
.09	.586	.34	.527	.59	.444	.84	.339
.10	.584	.35	.524	.60	.440	.85	.335
.11	.582	.36	.521	.61	.436	.86	.330
.12	.580	.37	.518	.62	.432	.87	.325
.13	.578	.38	.515	.63	.428	.88	.321
.14	.576	.39	.512	.64	.424	.89	.316
.15	.574	.40	.509	.65	.420	.90	.311
.16	.572	.41	.506	.66	.416	.91	.306
.17	.570	.42	.502	.67	.412	.92	.301
.18	.568	.43	.499	.68	.408	.93	.296
.19	.565	.44	.496	.69	.404	.94	.291
.20	.563	.45	.493	.70	.400	.95	.286
.21	.561	.46	.489	.71	.396	.96	.281
.22	.558	.47	.486	.72	.392	.97	.276
.23	.556	.48	.483	.73	.388	.98	.271
.24	.553	.49	.479	.74	.384	.99	.266
.25	.551	.50	.476	.75	.379	1.00	.261

[a] When ratios exceed the limits of Sect. 1.9, use $\dfrac{Kl/r}{C'_c}$ in lieu of $\dfrac{Kl/r}{C_c}$ values and equation

$F_a = C_a Q_a Q_s F_y$ (Appendix Sect. C5).

TABLE 5

VALUES OF C_c

For Use in Formulas (1.5-1) and (1.5-2) and in Table 4

F_y (ksi)	C_c	F_y (ksi)	C_c
33	131.7	46	111.6
35	127.9	50	107.0
36	126.1	55	102.0
39	121.2	60	97.7
40	119.6	65	93.8
42	116.7	90	79.8
45	112.8	100	75.7

TABLE 6

SLENDERNESS RATIOS OF ELEMENTS AS A FUNCTION OF F_y

Specification Section and Ratios	F_y (ksi)					
	36	42	46	50	60	65
Sect. 1.5.1.4.1						
Subpar. 2: $65/\sqrt{F_y}$	10.8	10.0	9.6	9.2	8.4	8.1
Subpar. 3: $190/\sqrt{F_y}$	31.7	29.3	28.0	26.9	24.5	23.6
Subpar. 4: $640/\sqrt{F_y}$	106.7	98.8	94.4	90.5	82.6	79.4
$257/\sqrt{F_y}$	42.8	39.7	37.9	36.3	33.2	31.9
Sect. 1.5.1.4.5						
$\sqrt{\dfrac{102 \times 10^3 C_b}{F_y}}$	$53\sqrt{C_b}$	$49\sqrt{C_b}$	$47\sqrt{C_b}$	$45\sqrt{C_b}$	$41\sqrt{C_b}$	$40\sqrt{C_b}$
$\sqrt{\dfrac{510 \times 10^3 C_b}{F_y}}$	$119\sqrt{C_b}$	$110\sqrt{C_b}$	$105\sqrt{C_b}$	$101\sqrt{C_b}$	$92\sqrt{C_b}$	$89\sqrt{C_b}$
Sect. 1.9.1.2						
$76/\sqrt{F_y}$	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
$127/\sqrt{F_y}$	21.2	19.6	18.7	18.0	16.4	15.8
Sect. 1.9.2.2						
$238/\sqrt{F_y}$	39.7	36.7	35.1	33.7	30.7	29.5
$317/\sqrt{F_y}$	52.8	48.9	46.7	44.8	40.9	39.3
$253/\sqrt{F_y}$	42.2	39.0	37.3	35.8	32.7	31.4
Sect. 1.9.2.3						
$3300/F_y$	91.7	78.6	71.7	66.0	55.0	50.8
$13000/F_y$	361	310	283	260	217	200
Sect. 1.10.2						
$\dfrac{14000}{\sqrt{F_y(F_y + 16.5)}}$	322	282	261	243	207	192
$2000/\sqrt{F_y}$	333	309	295	283	258	248

TABLE 7

VALUES OF C_b

For Use in Formulas (1.5-6a), (1.5-6b), and (1.5-7)

$\frac{M_1}{M_2}$	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 Formula (1.6-1a)

$\frac{M_1}{M_2}$	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
−0.55	0.82	0	0.60		
−0.50	0.80	0.05	0.58		

Note 1: $C_m = 0.6 - 0.4(M_1/M_2) \geq 0.4$

Note 2: M_1/M_2 is positive for reverse curvature and negative for single curvature.

TABLE 9

VALUES OF F'_e

For Use in Formula (1.6-1a), Sect. 1.6.1, for Steel of Any Yield Stress

$\frac{Kl_b}{r_b}$	F'_e (ksi)	$\frac{Kl_b}{r_b}$	F'_e (ksi)	$\frac{Kl_b}{r_b}$	F'_e (ksi)	$\frac{Kl_b}{r_b}$	F'_e (ksi)	$\frac{Kl_b}{r_b}$	F'_e (ksi)	$\frac{Kl_b}{r_b}$	F'_e (ksi)
21	338.62	51	57.41	81	22.76	111	12.12	141	7.51	171	5.11
22	308.54	52	55.23	82	22.21	112	11.90	142	7.41	172	5.05
23	282.29	53	53.16	83	21.68	113	11.69	143	7.30	173	4.99
24	259.26	54	51.21	84	21.16	114	11.49	144	7.20	174	4.93
25	238.93	55	49.37	85	20.67	115	11.29	145	7.10	175	4.88
26	220.90	56	47.62	86	20.19	116	11.10	146	7.01	176	4.82
27	204.84	57	45.96	87	19.73	117	10.91	147	6.91	177	4.77
28	190.47	58	44.39	88	19.28	118	10.72	148	6.82	178	4.71
29	177.56	59	42.90	89	18.85	119	10.55	149	6.73	179	4.66
30	165.92	60	41.48	90	18.44	120	10.37	150	6.64	180	4.61
31	155.39	61	40.13	91	18.03	121	10.20	151	6.55	181	4.56
32	145.83	62	38.85	92	17.64	122	10.03	152	6.46	182	4.51
33	137.13	63	37.62	93	17.27	123	9.87	153	6.38	183	4.46
34	129.18	64	36.46	94	16.90	124	9.71	154	6.30	184	4.41
35	121.90	65	35.34	95	16.55	125	9.56	155	6.22	185	4.36
36	115.22	66	34.28	96	16.20	126	9.41	156	6.14	186	4.32
37	109.08	67	33.27	97	15.87	127	9.26	157	6.06	187	4.27
38	103.42	68	32.29	98	15.55	128	9.11	158	5.98	188	4.23
39	98.18	69	31.37	99	15.24	129	8.97	159	5.91	189	4.18
40	93.33	70	30.48	100	14.93	130	8.84	160	5.83	190	4.14
41	88.83	71	29.62	101	14.64	131	8.70	161	5.76	191	4.09
42	84.65	72	28.81	102	14.35	132	8.57	162	5.69	192	4.05
43	80.76	73	28.02	103	14.08	133	8.44	163	5.62	193	4.01
44	77.13	74	27.27	104	13.81	134	8.32	164	5.55	194	3.97
45	73.74	75	26.55	105	13.54	135	8.19	165	5.49	195	3.93
46	70.57	76	25.85	106	13.29	136	8.07	166	5.42	196	3.89
47	67.60	77	25.19	107	13.04	137	7.96	167	5.35	197	3.85
48	64.81	78	24.54	108	12.80	138	7.84	168	5.29	198	3.81
49	62.20	79	23.93	109	12.57	139	7.73	169	5.23	199	3.77
50	59.73	80	23.33	110	12.34	140	7.62	170	5.17	200	3.73

Note: $F'_e = \dfrac{12\pi^2 E}{23(Kl_b/r_b)^2}$ (1.6-1a)

All grades of steel

TABLE 10-36

ALLOWABLE SHEAR STRESS (KSI) IN WEBS OF PLATE GIRDERS BY FORMULA (1.10-1)

For 36 ksi Yield Stress Steel, Tension Field Action Not Included

$F_y = 36$ ksi — Web Depth to Web Thickness — Slenderness Ratios h/t:

	Aspect Ratios 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
60											14.5	14.5	14.5	14.5
70									14.5	14.5	14.2	13.8	13.6	13.0
80						14.5	14.1	13.4	13.0	12.6	12.4	12.0	11.9	11.4
90				14.5	14.3	13.4	12.5	11.9	11.5	11.2	11.0	10.7	10.5	10.1
100			14.5	13.9	12.8	12.0	11.2	10.7	10.4	10.1	9.9	9.3	9.0	8.3
110		14.5	13.8	12.6	11.7	10.9	10.2	9.5	8.9	8.5	8.2	7.7	7.4	6.9
120		14.3	12.7	11.6	10.7	10.0	8.8	8.0	7.5	7.1	6.9	6.5	6.3	5.8
130	14.5	13.2	11.7	10.7	9.8	8.6	7.5	6.8	6.4	6.1	5.8	5.5	5.3	4.9
140	14.2	12.2	10.9	9.8	8.4	7.4	6.5	5.9	5.5	5.2	5.0	4.8	4.6	4.2
150	13.2	11.4	10.1	8.5	7.3	6.5	5.6	5.1	4.8	4.5	4.4	4.1	4.0	3.7
160	12.4	10.7	9.1	7.5	6.5	5.7	4.9	4.5	4.2	4.0	3.9	3.6	3.5	3.2
170	11.7	10.1	8.0	6.7	5.7	5.0	4.4	4.0	3.7	3.5	3.4	3.2	3.1	2.9
180	11.0	9.1	7.2	5.9	5.1	4.5	3.9	3.5	3.3	3.2	3.0	2.9	2.8	2.6
200	9.9	7.3	5.8	4.8	4.1	3.6	3.2	2.9	2.7	2.6	2.5	2.3	2.3	2.1
220	8.2	6.1	4.8	4.0	3.4	3.0	2.6	2.4	2.2	2.1	2.0	1.9	1.9	1.7
240	6.9	5.1	4.0	3.3	2.9	2.5	2.2	2.0	1.9	1.8	1.7	1.6	1.6	1.4
260	5.8	4.3	3.4	2.8	2.4	2.2	1.9	1.7	1.6	1.5	1.5	1.4	1.3	1.2
280	5.0	3.7	3.0	2.5										
300	4.4	3.3	2.6											
320	3.9	2.9												

TABLE 10-50

ALLOWABLE SHEAR STRESS (KSI) IN WEBS OF PLATE GIRDERS BY FORMULA (1.10-1)

For 50 ksi Yield Stress Steel, Tension Field Action Not Included

		Aspect Ratios 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							20.0	20.0	20.0	19.9	19.5	18.9	18.6	17.9
	70					20.0	20.0	18.9	18.0	17.4	17.0	16.7	16.2	16.0	15.3
	80			20.0	20.0	18.9	17.8	16.6	15.8	15.3	14.9	14.6	14.2	14.0	13.0
	90			19.9	18.1	16.8	15.8	14.7	14.0	13.3	12.6	12.2	11.5	11.1	10.3
Web Depth to Web Thickness	100		20.0	17.9	16.3	15.1	14.2	12.6	11.5	10.7	10.2	9.9	9.3	9.0	8.3
	110	20.0	18.3	16.3	14.8	13.6	12.0	10.4	9.5	8.9	8.5	8.2	7.7	7.4	6.9
	120	19.5	16.8	15.0	13.3	11.5	10.1	8.8	8.0	7.5	7.1	6.9	6.5	6.3	5.8
	130	18.0	15.5	13.7	11.4	9.8	8.6	7.5	6.8	6.4	6.1	5.8	5.5	5.3	4.9
Slenderness Ratios h/t:	140	16.7	14.4	11.8	9.8	8.4	7.4	6.4	5.9	5.5	5.2	5.0	4.8	4.6	4.2
	150	15.6	13.0	10.3	8.5	7.3	6.5	5.6	5.1	4.8	4.5	4.4	4.1	4.0	3.7
	160	14.6	11.5	9.1	7.5	6.4	5.7	4.9	4.5	4.2	4.0	3.9	3.6	3.5	3.2
	170	13.7	10.1	8.0	6.7	5.7	5.0	4.4	4.0	3.7	3.5	3.4	3.2	3.1	2.9
	180	12.2	9.1	7.2	5.9	5.1	4.5	3.9	3.5	3.3	3.2	3.0	2.9	2.8	2.6
	200	9.9	7.3	5.8	4.8	4.1	3.6	3.2	2.9	2.7	2.6	2.5	2.3	2.3	2.1
	220	8.2	6.1	4.8	4.0	3.4	3.0	2.6	2.4	2.2	2.1	2.0	1.9	1.9	1.7
	240	6.9	5.1	4.0	3.3	2.9	2.5	2.2	2.0	1.9	1.8	1.7	1.6	1.6	1.4
	260	5.8	4.3	3.4	2.8	2.4	2.2								
	280	5.0	3.7	3.0	2.5										

TABLE 11-36

ALLOWABLE SHEAR STRESS (KSI) IN WEBS OF PLATE GIRDERS
For 36 ksi Yield Stress Steel, Tension Field Action Included

(*Italic* values indicate gross area, as percent of web area, required for pairs of intermediate stiffeners of 36 ksi yield stress steel.)[a]

$F_y = 36$ ksi

Slenderness Ratios h/t: Web Depth to Web Thickness

h/t	Aspect Ratios 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
60										14.5	14.5	14.5	14.5	14.5
70							14.5	14.5	14.5	14.4	14.2	13.8	13.6	13.0
80					14.5	14.5	14.0	13.4	13.0	12.6	12.4	12.2 *0.3*	12.0 *0.4*	11.4
90				14.5	14.3	13.4	12.5	12.2 *0.6*	11.9 *0.9*	11.8 *1.1*	11.6 *1.2*	11.3 *1.2*	11.1 *1.2*	10.1
100			14.5	13.9	12.8	12.3 *0.5*	11.9 *1.4*	11.6 *1.8*	11.3 *2.0*	11.1 *2.1*	10.9 *2.2*	10.3 *2.3*	10.0 *2.1*	8.3
110		14.5	13.8	12.6	12.2 *0.9*	11.9 *1.8*	11.5 *2.5*	11.0 *3.1*	10.5 *3.5*	10.1 *3.6*	9.8 *3.6*	9.2 *3.4*	8.8 *3.1*	6.9
120		14.3	12.7	12.2 *1.1*	11.8 *2.1*	11.5 *2.8*	10.8 *4.1*	10.2 *4.7*	9.8 *4.9*	9.4 *4.9*	9.0 *4.7*	8.4 *4.3*	8.0 *3.8*	5.8
130	14.5	13.2	12.2 *0.9*	11.9 *2.2*	11.5 *3.2*	11.0 *4.5*	10.3 *5.6*	9.7 *5.9*	9.2 *6.0*	8.8 *5.8*	8.4 *5.6*	7.8 *5.0*	7.3 *4.4*	4.9
140	14.2	12.4 *0.3*	12.0 *1.9*	11.6 *3.2*	11.0 *4.8*	10.5 *5.9*	9.8 *6.7*	9.2 *6.9*	8.7 *6.8*	8.3 *6.6*	7.9 *6.3*	7.2 *5.5*	6.8 *4.9*	4.2
150	13.2	12.2 *1.2*	11.8 *2.8*	11.2 *4.7*	10.6 *6.1*	10.1 *7.0*	9.4 *7.6*	8.8 *7.7*	8.3 *7.5*	7.9 *7.2*	7.5 *6.8*	6.8 *6.0*	6.3 *5.2*	3.7
160	12.4	12.0 *2.1*	11.5 *4.1*	10.9 *6.0*	10.3 *7.2*	9.8 *8.0*	9.1 *8.4*	8.5 *8.3*	8.0 *8.1*	7.6 *7.7*	7.2 *7.3*	6.5 *6.3*		3.2
170	12.3 *0.9*	11.8 *2.8*	11.2 *5.3*	10.6 *7.0*	10.1 *8.1*	9.6 *8.7*	8.9 *9.0*	8.3 *8.9*	7.7 *8.5*	7.3 *8.1*	6.9 *7.7*			2.9
180	12.1 *1.6*	11.6 *4.0*	10.9 *6.3*	10.4 *7.9*	9.9 *8.8*	9.4 *9.4*	8.7 *9.6*	8.1 *9.3*	7.5 *8.9*	7.1 *8.5*	6.7 *8.0*			2.6
200	11.9 *2.9*	11.2 *6.0*	10.5 *8.0*	10.0 *9.2*	9.5 *10.0*	9.1 *10.4*	8.3 *10.4*	7.7 *10.0*	7.2 *9.5*					2.1
220	11.5 *4.8*	10.8 *7.5*	10.3 *9.2*	9.7 *10.2*	9.3 *10.8*	8.8 *11.1*	8.1 *11.0*	7.5 *10.6*						1.7
240	11.2 *6.2*	10.6 *8.6*	10.0 *10.1*	9.5 *11.0*	9.1 *11.5*	8.6 *11.7*								1.4
260	11.0 *7.3*	10.4 *9.5*	9.9 *10.8*	9.4 *11.6*	8.9 *12.0*	8.5 *12.1*								1.2
280	10.8 *8.2*	10.2 *10.2*	9.7 *11.4*	9.2 *12.1*										
300	10.7 *9.0*	10.1 *10.8*	9.6 *11.8*											
320	10.5 *9.5*	10.0 *11.2*												

[a] For single angle stiffeners, multiply by 1.8; for single plate stiffeners, multiply by 2.4.

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

ALLOWABLE SHEAR STRESS (KSI) IN WEBS OF PLATE GIRDERS
For 50 ksi Yield Stress Steel, Tension Field Action Included

(*Italic* values indicate gross area, as percent of web area, required for pairs of intermediate stiffeners of 50 ksi yield stress steel.)[a]

h/t	Aspect Ratios 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
50										20.0	20.0	20.0	20.0	20.0
60							20.0	20.0	20.0	19.9	19.5	18.9	18.6	17.9
70					20.0	20.0	18.9	18.0	17.4	17.1 *0.2*	16.9 *0.4*	16.6 *0.5*	16.3 *0.6*	15.3
80			20.0	20.0	18.9	17.8	17.0 *0.6*	16.6 *1.1*	16.2 *1.4*	15.9 *1.6*	15.7 *1.6*	15.2 *1.6*	14.9 *1.5*	13.0
90			19.9	18.1	17.1 *0.4*	16.7 *1.3*	16.2 *2.1*	15.7 *2.5*	15.1 *2.8*	14.6 *3.1*	14.2 *3.1*	13.4 *3.0*	12.8 *2.8*	10.3
100		20.0	17.9	17.0 *0.8*	16.5 *1.9*	16.1 *2.6*	15.2 *3.7*	14.4 *4.4*	13.8 *4.6*	13.2 *4.6*	12.8 *4.5*	11.9 *4.1*	11.3 *3.7*	8.3
110	20.0	18.3	17.0 *0.9*	16.5 *2.1*	16.0 *3.2*	15.3 *4.5*	14.3 *5.5*	13.4 *5.9*	12.8 *5.9*	12.2 *5.8*	11.7 *5.6*	10.8 *5.0*	10.2 *4.4*	6.9
120	19.5	17.2 *0.4*	16.6 *2.0*	16.0 *3.4*	15.2 *5.0*	14.5 *6.1*	13.5 *6.9*	12.7 *7.0*	12.0 *6.9*	11.4 *6.7*	10.9 *6.4*	10.0 *5.6*	9.3 *4.9*	5.8
130	18.0	16.8 *1.5*	16.3 *3.1*	15.4 *5.1*	14.6 *6.5*	14.0 *7.4*	12.9 *7.9*	12.1 *7.9*	11.4 *7.7*	10.8 *7.4*	10.3 *7.0*	9.3 *6.1*	8.6 *5.3*	4.9
140	17.2 *0.5*	16.6 *2.4*	15.7 *4.7*	14.9 *6.5*	14.2 *7.7*	13.5 *8.4*	12.5 *8.7*	11.6 *8.6*	10.9 *8.3*	10.3 *7.9*	9.8 *7.5*	8.8 *6.5*	8.1 *5.7*	4.2
150	16.9 *1.4*	16.2 *3.6*	15.3 *6.0*	14.5 *7.6*	13.8 *8.6*	13.1 *9.2*	12.1 *9.4*	11.3 *9.2*	10.5 *8.8*	9.9 *8.3*	9.4 *7.9*	8.4 *6.8*	7.7 *5.9*	3.7
160	16.7 *2.1*	15.8 *4.9*	14.9 *7.1*	14.2 *8.5*	13.5 *9.3*	12.8 *9.8*	11.8 *9.9*	11.0 *9.7*	10.2 *9.2*	9.6 *8.7*	9.1 *8.2*	8.0 *7.1*		3.2
170	16.5 *2.9*	15.5 *6.0*	14.6 *8.0*	13.9 *9.2*	13.2 *10.0*	12.6 *10.4*	11.6 *10.4*	10.7 *10.0*	10.0 *9.5*	9.4 *9.0*	8.8 *8.5*			2.9
180	16.2 *4.1*	15.2 *6.9*	14.4 *8.8*	13.7 *9.9*	13.0 *10.5*	12.4 *10.8*	11.4 *10.8*	10.5 *10.4*	9.8 *9.8*	9.1 *9.3*	8.6 *8.7*			2.6
200	15.7 *5.9*	14.8 *8.4*	14.0 *9.9*	13.3 *10.8*	12.6 *11.3*	12.0 *11.6*	11.0 *11.4*	10.2 *10.9*	9.4 *10.3*					2.1
220	15.3 *7.3*	14.4 *9.5*	13.7 *10.8*	13.0 *11.6*	12.4 *12.0*	11.8 *12.1*	10.8 *11.8*	9.9 *11.2*						1.7
240	15.0 *8.3*	14.2 *10.3*	13.5 *11.5*	12.8 *12.1*	12.2 *12.4*	11.6 *12.5*								1.4
260	14.8 *9.2*	14.0 *10.9*	13.3 *12.0*	12.7 *12.5*	12.0 *12.8*	11.5 *12.8*								
280	14.6 *9.8*	13.9 *11.4*	13.2 *12.4*	12.5 *12.9*										

Slenderness Ratios h/t: Web Depth to Web Thickness

[a] For areas of other intermediate stiffeners, multiply *italic* values by appropriate factor:

Stiffener Steel Grade	Pairs of Stiffeners	Single Angle Stiffeners	Single Plate Stiffeners
$F_y = 50$ ksi	1.0	1.8	2.4
$F_y = 36$ ksi	1.4	2.5	3.3

Note: Girders so proportioned that the computed shear is less than that given in right-hand column do not require intermediate stiffeners.

TABLE 12
VALUES OF C_h
For Determining Maximum Allowable Bending Stress in Hybrid Girders
(by equation $F'_b = C_h F_b$)

$\dfrac{A_w}{A_f}$	α = Ratio F_y (web) to F_y (flange)						
	0.9	0.8	0.7	0.6	0.5	0.4	0.3
0.3	1.00	1.00	0.99	0.99	0.99	0.98	0.97
0.4	1.00	1.00	0.99	0.99	0.98	0.97	0.96
0.5	1.00	1.00	0.99	0.98	0.98	0.97	0.96
0.6	1.00	0.99	0.99	0.98	0.98	0.96	0.95
0.7	1.00	0.99	0.99	0.98	0.97	0.95	0.94
0.8	1.00	0.99	0.99	0.98	0.97	0.95	0.93
0.9	1.00	0.99	0.98	0.97	0.96	0.94	0.93
1.0	1.00	0.99	0.98	0.97	0.96	0.94	0.92
1.1	1.00	0.99	0.98	0.97	0.96	0.93	0.91
1.2	1.00	0.99	0.98	0.97	0.95	0.93	0.91
1.3	1.00	0.99	0.98	0.96	0.95	0.92	0.90
1.4	1.00	0.99	0.98	0.96	0.95	0.92	0.89
1.5	1.00	0.99	0.98	0.96	0.94	0.91	0.89
1.6	1.00	0.99	0.97	0.96	0.94	0.91	0.88
1.7	1.00	0.99	0.97	0.95	0.94	0.90	0.88
1.8	1.00	0.99	0.97	0.95	0.93	0.90	0.87
1.9	1.00	0.99	0.97	0.95	0.93	0.89	0.86
2.0	1.00	0.99	0.97	0.95	0.93	0.89	0.86
2.2	1.00	0.99	0.97	0.94	0.92	0.88	0.85
2.4	1.00	0.98	0.97	0.94	0.92	0.88	0.84
2.6	1.00	0.98	0.96	0.94	0.91	0.87	0.83
2.8	1.00	0.98	0.96	0.93	0.91	0.86	0.82
3.0	1.00	0.98	0.96	0.93	0.90	0.86	0.81

Note 1:
$$F'_b = C_h F_b = \left[\frac{12 + (A_w/A_f)(3\alpha - \alpha^3)}{12 + 2(A_w/A_f)} \right] F_b \qquad (1.10\text{-}6)$$

Note 2: The lower value using Formulas (1.10-5) and (1.10-6) governs for the flanges of a hybrid girder.

APPENDIX B

Fatigue

SECTION B1 LOADING CONDITIONS; TYPE AND LOCATION OF MATERIAL

In the design of members and connections subject to repeated variation of live load stress, 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 as in Table B1.

The type and location of material shall be categorized as in Table B2.

SECTION B2 ALLOWABLE STRESSES

The maximum stress shall not exceed the basic allowable stress provided in Sects. 1.5 and 1.6 of this Specification, and the maximum range of stress shall not exceed that given in Table B3.

SECTION B3 PROVISIONS FOR MECHANICAL FASTENERS

B3.1 Range in tensile stress in properly tightened A325 or A490 bolts need not be considered, but the maximum computed stress, including prying action, shall not exceed the values given in Table 1.5.2.1, subject to the following stipulations:

1. Connections subject to more than 20,000 cycles, but not more than 500,000 cycles, of direct tension may be designed for the stress produced by the sum of applied and prying loads if the prying load does not exceed 10 percent of the externally applied load. If the prying force exceeds 10 percent, the allowable tensile stress given in Table 1.5.2.1 shall be reduced 40 percent, applicable to the external load alone.

2. Connections subject to more than 500,000 cycles of direct tension may be designed for the stress produced by the sum of applied and prying loads if the prying load does not exceed 5 percent of the externally applied load. If the prying force exceeds 5 percent, the allowable tensile stress given in Table 1.5.2.1 shall be reduced 50 percent, applicable to the external load alone.

B3.2 The use of other bolts and threaded parts subjected to tensile fatigue loading is not recommended.

B3.3 Rivets, bolts, and threaded parts subjected to cyclic loading in shear may be designed for the bearing-type shear stresses given in Table 1.5.2.1 insofar as the fatigue strength of the fasteners themselves is concerned.

TABLE B1

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 ten applications every day for 25 years.

[c] Approximately equivalent to fifty applications every day for 25 years.

[d] Approximately equivalent to two hundred applications every day for 25 years.

TABLE B2

General Condition	Situation	Kind of Stress[a]	Stress Category. (See Table B3)	Illustrative Example Nos. (See Fig. B1)[b]
Plain material	Base metal with rolled or cleaned surfaces.	T or Rev.	A	1,2
Built-up members	Base metal and weld metal in members, without attachments, built-up of plates or shapes connected by continuous full- or partial-penetration groove welds or continuous fillet welds parallel to the direction of applied stress.	T or Rev.	B	3,4,5,6
	Calculated flexural stress, f_b, in base metal at toe of welds on girder webs or flanges adjacent to welded transverse stiffeners.	T or Rev.	C	7
	Base metal at end of partial-length welded cover plates having square or tapered ends, with or without welds across the ends.	T or Rev.	E	5
Mechanically fastened connections	Base metal at gross section of high-strength-bolted friction-type connections, except connections subject to stress reversal and 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 high-strength bolted bearing connections.	T or Rev.	B	8,9
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.	T or Rev.	E	17,18,20
	Weld metal of continuous or intermittent longitudinal or transverse fillet welds.	S	F	5,17,18,21

[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.

TABLE B2 (*continued*)

General Condition	Situation	Kind of Stress[a]	Stress Category. (See Table B3)	Illustrative Example Nos. (See Fig. B1)[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 Table 9.25.3 of AWS D1.1-77.	T or Rev.	B	10
	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 Table 9.25.3 of AWS D1.1-77.	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 and/or weld soundness is not established by radiographic or ultrasonic inspection in accordance with the requirements of Table 9.25.3 of AWS D1.1-77.	T or Rev.	C	10,11,12,13
	Weld metal of partial-penetration transverse groove welds, based on effective throat area of the weld or welds.	T or Rev.	F	16
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
Attachments	Base metal at detail of any length attached by groove welds subject to transverse and/or longitudinal loading, when the detail embodies a transition radius, R, 2 inches or greater, with the weld termination ground smooth:			
	$R \geq 24$ in.	T or Rev.	B	14
	24 in. $> R \geq 6$ in.	T or Rev.	C	14
	6 in. $> R \geq 2$ in.	T or Rev.	D	14

TABLE B2 (*continued*)

General Condition	Situation	Kind of Stress[a]	Stress Category. (See Table B3)	Illustrative Example Nos. (See Fig. B1)[b]
Attachments (cont'd)	Base metal at detail attached by groove welds or fillet welds subject to longitudinal loading, with transition radius, if any, less than 2 inches: .			
	2 in. $< a \le 12b$ or 4 in.	T or Rev.	D	15
	$a > 12b$ or 4 in.	T or Rev.	E	15,23,24, 25,26

	where
	  a = detail dimension parallel to the direction of stress
	  b = detail dimension normal to the direction of stress and the surface of the base metal			

	Base metal at a detail of any length attached by fillet welds or partial-penetration groove welds in the direction parallel to the stress, when the detail embodies a transition radius, R, 2 inches or greater, with weld termination ground smooth:			
	$R \ge 24$ in.	T or Rev.	B	19
	24 in > $R \ge 6$ in.	T or Rev.	C	19
	6 in. > $R \ge 2$ in.	T or Rev.	D	19
	Base metal at a detail attached by groove welds or fillet welds, where the detail dimension parallel to the direction of stress, a, is less than 2 in.	T or Rev.	C	23,24,25
	Base metal at a stud-type shear connector attached by fillet weld.	T or Rev.	C	22
	Shear stress on nominal area of stud-type shear connectors.	S	F	22

TABLE B3
ALLOWABLE RANGE OF STRESS (F_{sr}), KSI

Category (From Table B2)	Loading Condition 1 F_{sr1}	Loading Condition 2 F_{sr2}	Loading Condition 3 F_{sr3}	Loading Condition 4 F_{sr4}
A	60	36	24	24
B	45	27.5	18	16
C	32	19	13	10[a]
D	27	16	10	7
E	21	12.5	8	5
F	15	12	9	8

[a] Flexural stress range of 12 ksi permitted at toe of stiffener welds on webs or flanges.
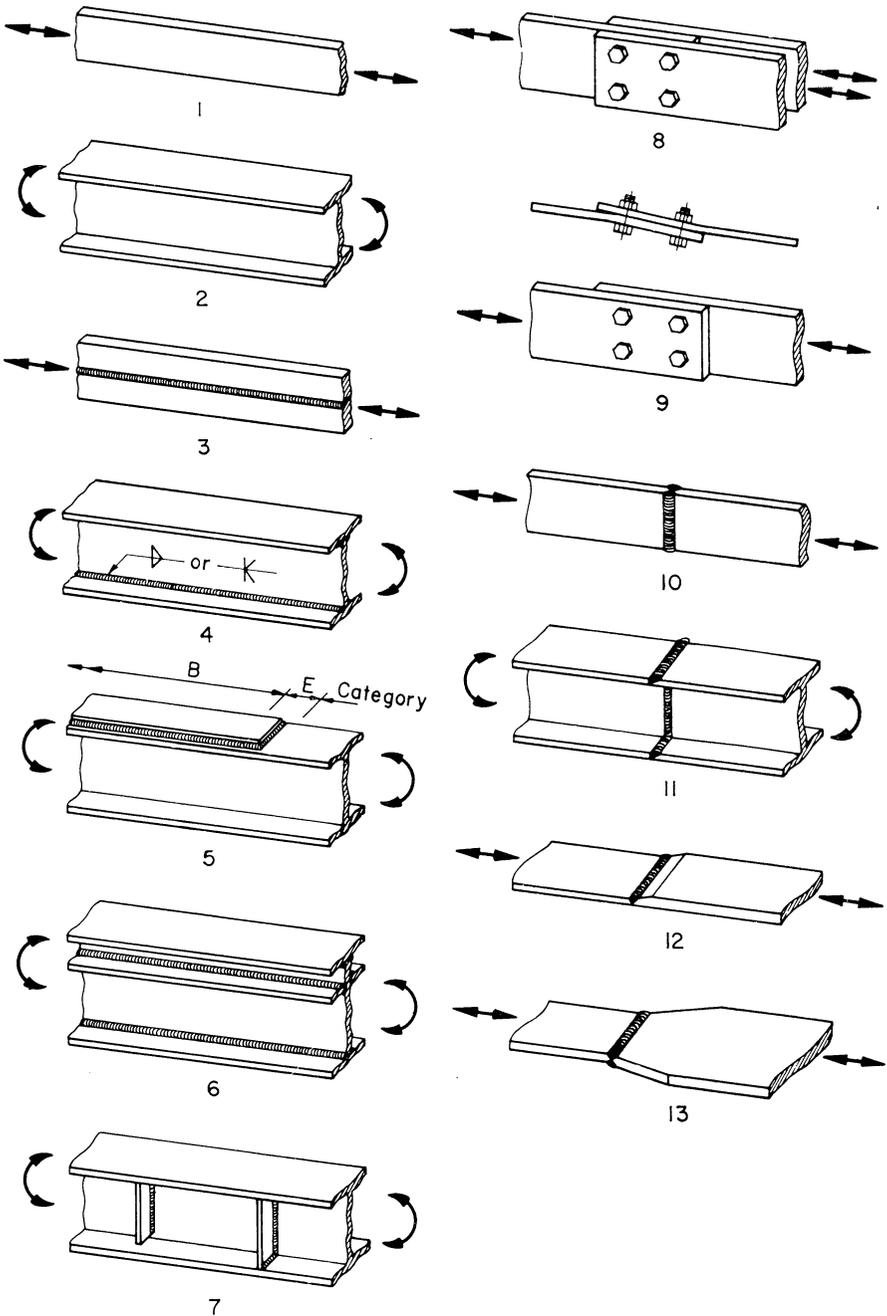
Fig. B1.   Illustrative examples
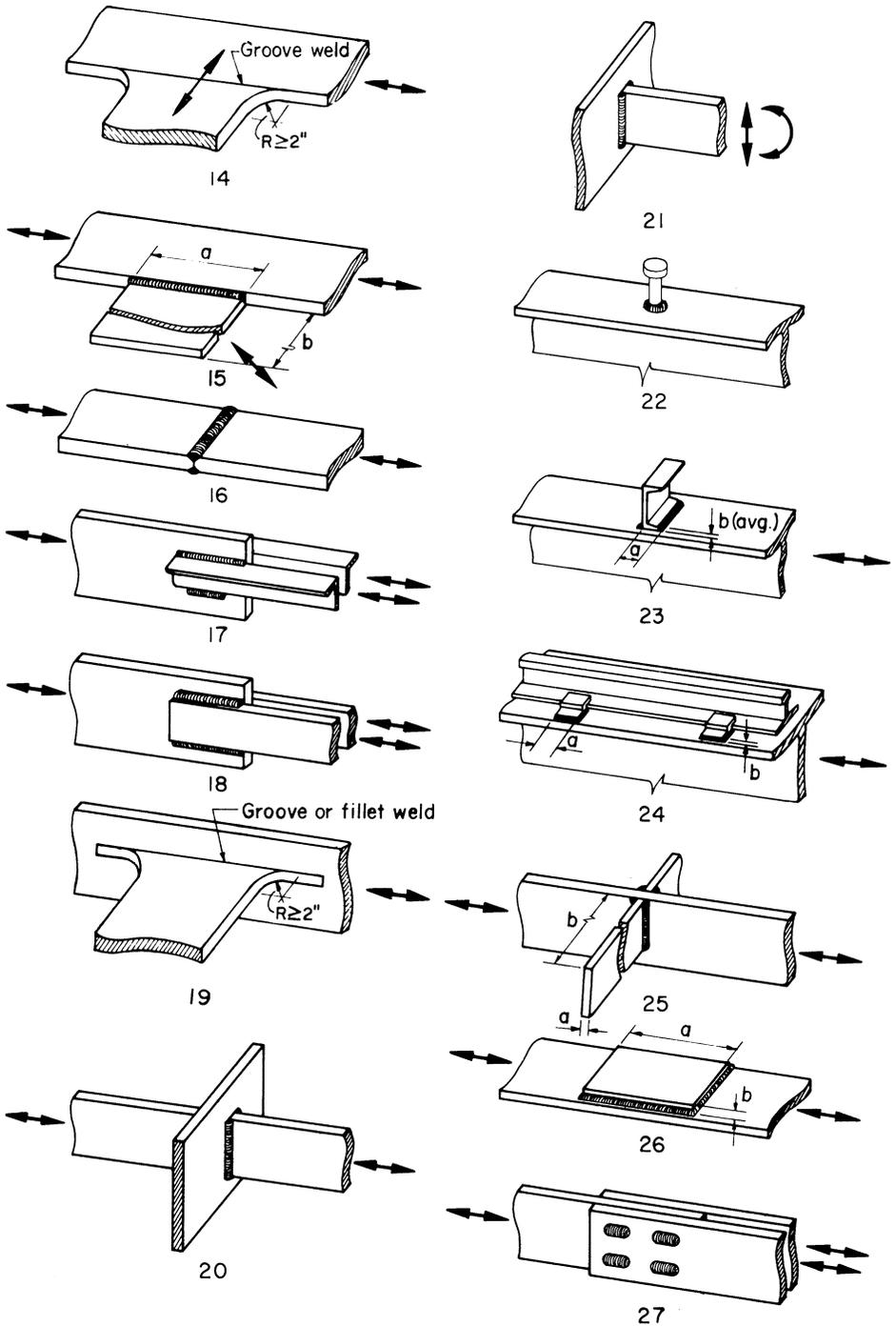
Fig. B1.   Illustrative examples (continued)

APPENDIX C

Slender Compression Elements

SECTION C1   GENERAL

Axially loaded members and flexural members containing elements subject to compression and having a width-thickness ratio in excess of the applicable limit given in Sect. 1.9 shall be proportioned to meet the requirements of this Appendix.

SECTION C2   STRESS REDUCTION FACTOR—UNSTIFFENED COMPRESSION ELEMENTS

Except as hereinafter provided, stress on unstiffened compression elements whose width-thickness ratio exceeds the applicable limit given in Sect. 1.9.1.2 shall be subject to a reduction factor Q_s.   The value of Q_s shall be determined by Formulas (C2-1) through (C2-6), as applicable, where b is the width of the unstiffened element as defined in Sect. 1.9.1.1.   When such elements comprise the compression flange of a flexural member, the maximum allowable bending stress shall not exceed $0.6F_y Q_s$ nor the applicable value as provided in Sect. 1.5.1.4.5.   The allowable stress of axially loaded compression members shall be modified by the appropriate reduction factor Q_s , as provided in Sect. C5.

For single angles:

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

$$Q_s = 1.340 - 0.00447(b/t)\sqrt{F_y} \tag{C2-1}$$

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

$$Q_s = 15{,}500/[F_y(b/t)^2] \tag{C2-2}$$

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

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

$$Q_s = 1.415 - 0.00437(b/t)\sqrt{F_y} \tag{C2-3}$$

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

$$Q_s = 20{,}000/[F_y(b/t)^2] \tag{C2-4}$$

For stems of tees:

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

$$Q_s = 1.908 - 0.00715(b/t)\sqrt{F_y} \tag{C2-5}$$

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

$$Q_s = 20{,}000/[F_y(b/t)^2] \tag{C2-6}$$

However, unstiffened elements of channels and tees whose proportions exceed the limits of Sect. 1.9.1.2 shall conform to the limits given in Table C1.

TABLE C1

LIMITING PROPORTIONS FOR CHANNELS AND TEES

Shape	Ratio of full flange width to profile depth	Ratio of flange thickness to web or stem thickness
Built-up or rolled channels	≤0.25	≤3.0
	≤0.50	≤2.0
Built-up tees	≥0.50	≥1.25
Rolled tees	≥0.50	≥1.10

SECTION C3 STIFFENED COMPRESSION ELEMENTS

When the width-thickness ratio of a uniformly compressed stiffened element (except perforated cover plates) exceeds the applicable limit given in Sect. 1.9.2.2, a reduced effective width, b_e, shall be used in computing the flexural design properties of the section containing the element and the permissible axial stress, except that the ratio b_e/t need not be taken as less than the applicable value permitted in Sect. 1.9.2.2.

For the flanges of square and rectangular sections of uniform thickness:

$$b_e = \frac{253t}{\sqrt{f}}\left[1 - \frac{50.3}{(b/t)\sqrt{f}}\right] \le b \qquad \text{(C3-1)}$$

For other uniformly compressed elements:

$$b_e = \frac{253t}{\sqrt{f}}\left[1 - \frac{44.3}{(b/t)\sqrt{f}}\right] \le b \qquad \text{(C3-2)}$$

where

b = actual width of a stiffened compression element, as defined in Sect. 1.9.2.1, inches

t = its thickness, inches

f = computed compressive stress in the stiffened elements, based on the design properties as specified in Sect. C4, kips per square inch. If un-stiffened 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 $F_a Q_s$ or $F_b Q_s$, as applicable.

When the allowable stresses are increased due to wind or seismic loading in accordance with the provisions of Sect. 1.5.6, the effective width, b_e, shall be determined on the basis of 0.75 times the stress caused by wind or seismic loading acting alone or in combination with the design dead and live loading.

The allowable stress for axially loaded circular tubular members not meeting the requirements of Sect. 1.9.2.3, but having a diameter-to-thickness ratio of less than $13,000/F_y$, shall not exceed the smaller value determined by Sect. 1.5.1.3 nor

$$F_a = \frac{662}{D/t} + 0.40F_y \qquad \text{(C3-3)}$$

where D = outside diameter

t = wall thickness

SECTION C4   DESIGN PROPERTIES

Properties of sections shall be determined in accordance with conventional methods, using the full cross section of the member except as follows:

In computing the moment of inertia and section modulus of flexural members with respect to the axis of bending under consideration, the effective width of stiffened compression elements parallel to the axis of bending and having a width-thickness ratio in excess of the applicable limit given in Sect. 1.9.2.2 shall be used rather than the actual width, and the axis of bending shall be located accordingly, except that, for sections otherwise symmetrical, the properties may conservatively and more easily be computed using a corresponding effective area on the tension side of the neutral axis as well.   That portion of the area which is neglected in arriving at the effective area shall be located at and symmetrically about the center line of the stiffened element to which it applies.

The stress, f_a , due to axial loading and the radius of gyration, r, shall be computed on the basis of actual cross-sectional area.   However, the allowable axial stress, F_a , as provided in Sect. C5, shall be subject to the form factor

$$Q_a = \frac{\text{effective area}}{\text{actual area}}$$

where the effective area is equal to the actual area less $\Sigma(b - b_e)t$.

SECTION C5   AXIALLY LOADED COMPRESSION MEMBERS

The allowable stress for axially loaded compression members containing unstiffened or stiffened elements shall not exceed

$$F_a = \frac{Q_s Q_a \left[1 - \dfrac{(Kl/r)^2}{2C'_c{}^2} \right] F_y}{\dfrac{5}{3} + \dfrac{3(Kl/r)}{8C'_c} - \dfrac{(Kl/r)^3}{8C'_c{}^3}} \tag{C5-1}$$

where

$$C'_c = \sqrt{\frac{2\pi^2 E}{Q_s Q_a F_y}}$$

when the largest effective slenderness ratio of any unbraced segment of the member is less than C'_c , nor the value given by Formula (1.5-2) or (1.5-3) when Kl/r exceeds C'_c or l/r exceeds 120, as applicable.

SECTION C6   COMBINED AXIAL AND FLEXURAL STRESS

In applying the provisions of Sect. 1.6 to members subject to combined axial and flexural stress and containing stiffened elements whose width-thickness ratio exceeds the applicable limit given in Sect. 1.9, the stresses F_a , f_{bx} , and f_{by} shall be calculated on the basis of the section properties as provided in Sects. C4 and C5, as applicable.   The allowable bending stress, F_b , for members containing unstiffened elements whose width-thickness ratio exceeds the applicable limit given in Sect. 1.9 shall be the smaller value, $0.6F_yQ_s$ or that provided in Sect. 1.5.1.4.5.

APPENDIX D

Web-Tapered Members

SECTION D1 GENERAL

The design of tapered members meeting the requirements herein shall be governed by the provisions of Part 1, except as modified by this Appendix.

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

1. It shall possess at least one axis of symmetry which shall be perpendicular to the plane of bending if moments are present.

2. The flanges shall be of equal and constant area.

3. The depth shall vary linearly as

$$d_o \left(1 + \gamma \frac{z}{l} \right)$$

where

d_o = depth at smaller end of member, inches
d_L = depth at larger end of member, inches
γ = $(d_L - d_o)/d_o \leq$ the smaller of $0.268l/d_o$ or 6.0
z = distance from the smaller end of member, inches
l = length of member, inches

SECTION D2 ALLOWABLE STRESSES—COMPRESSION

On the gross section of axially loaded tapered compression members, the axial compressive stress, in kips per square inch, shall not exceed the following:

When the effective slenderness ratio, S is less than C_c:

$$F_{a\gamma} = \frac{\left(1.0 - \dfrac{S^2}{2C_c{}^2} \right) F_y}{\dfrac{5}{3} + \dfrac{3S}{8C_c} - \dfrac{S^3}{8C_c{}^3}} \tag{D2-1}$$

When the effective slenderness ratio, S, exceeds C_c:

$$F_{a\gamma} = \frac{12\pi^2 E}{23 S^2} \tag{D2-2}$$

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*
l = actual unbraced length of member, inches
r_{ox} = strong axis radius of gyration at the smaller end of a tapered member, inches
r_{oy} = weak axis radius of gyration at the smaller end of a tapered member, inches

SECTION D3 ALLOWABLE STRESSES—BENDING**

Tension and compression stresses on extreme fibers of tapered flexural members, in kips per square inch, shall not exceed the following values:

---

* See Commentary Sect. D2.
** See Commentary Sect. D3.

$$F_{b\gamma} = \frac{2}{3}\left[1.0 - \frac{F_y}{6B\sqrt{F_{s\gamma}{}^2 + F_{w\gamma}{}^2}}\right]F_y \leq 0.60F_y \qquad \text{(D3-1)}$$

unless $F_{b\gamma} \leq F_y/3$, in which case

$$F_{b\gamma} = B\sqrt{F_{s\gamma}{}^2 + F_{w\gamma}{}^2} \qquad \text{(D3-2)}$$

In the above Formulas,

$$F_{s\gamma} = \frac{12 \times 10^3}{h_s l d_o / A_f} \quad \text{and} \quad F_{w\gamma} = \frac{170 \times 10^3}{(h_w l / r_{To})^2}$$

where

h_s = factor equal to $1.0 + 0.0230\gamma\sqrt{l d_o / A_f}$
h_w = factor equal to $1.0 + 0.00385\gamma\sqrt{l / r_{To}}$
l = distance between cross sections braced against twist or lateral displacement of the compression flange, inches
r_{To} = radius of gyration of a section at the smaller end, considering only the compression flange plus $\frac{1}{3}$ of the compression web area, taken about an axis in the plane of the web, inches
A_f = area of the compression flange, square inches
γ = tapering ratio, equal to $(d_l - d_o)/d_o$
d_o = depth at smaller end of unbraced segment, inches
d_l = depth at larger end of unbraced segment, inches

and where B is determined as follows:

1. 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:*

$$B = 1.0 + 0.37\left[1.0 + \frac{M_1}{M_2}\right] + 0.50\gamma\left[1.0 + \frac{M_1}{M_2}\right] \geq 1.0$$

2. 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:**

$$B = 1.0 + 0.58\left[1.0 + \frac{f_{b1}}{f_{b2}}\right] - 0.70\gamma\left[1.0 + \frac{f_{b1}}{f_{b2}}\right] \geq 1.0$$

3. When the largest computed bending stress, f_{b2}, occurs at the small 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:**

$$B = 1.0 + 0.55\left[1.0 + \frac{f_{b1}}{f_{b2}}\right] + 2.2\gamma\left[1.0 + \frac{f_{b1}}{f_{b2}}\right] \geq 1.0$$

In the foregoing, $\gamma = (d_l - d_o)/d_o$, calculated for the unbraced length that contains the maximum computed bending stress.

---

  * 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 $f_{b1}/f_{b2} \neq 0$.

4. When the computed bending stress at the smaller end of a tapered member or segment thereof is equal to zero:

$$B = \frac{1.75}{1.0 + 0.25\sqrt{\gamma}}$$

where $\gamma = (d_l - d_o)/d_o$, calculated for the unbraced length adjacent to the point of zero bending stress.

SECTION D4 COMBINED STRESSES

Tapered members and unbraced segments thereof subjected to both axial compression and bending stresses shall be proportioned to satisfy the following requirements:

$$\left(\frac{f_{ao}}{F_{a\gamma}}\right) + \frac{C'_m}{\left(1 - \frac{f_{ao}}{F'_{e\gamma}}\right)} \left(\frac{f_{bl}}{F_{b\gamma}}\right) \leq 1.0 \qquad \text{(D4-1a)}$$

and

$$\frac{f_a}{0.6F_y} + \frac{f_b}{F_{b\gamma}} \leq 1.0 \qquad \text{(D4-1b)}$$

When $f_{ao}/F_{a\gamma} \leq 0.15$, Formula (D4-2) may be used in lieu of Formulas (D4-1a) and (D4-1b).

$$\left(\frac{f_{ao}}{F_{a\gamma}}\right) + \left(\frac{f_{bl}}{F_{b\gamma}}\right) \leq 1.0 \qquad \text{(D4-2)}$$

where

$F_{a\gamma}$ = axial compressive stress permitted in the absence of bending moment, kips per square inch

$F_{b\gamma}$ = bending stress permitted in the absence of axial force, kips per square inch

$F'_{e\gamma}$ = Euler stress divided by factor of safety, kips per square inch; equal to

$$\frac{12\pi^2 E}{23(K_\gamma l_b/r_{bo})^2}$$

where l is the actual unbraced length in the plane of bending and r_{bo} is the corresponding radius of gyration at its smaller end

f_{ao} = computed axial stress at the smaller end of the member or unbraced segment thereof, as applicable, kips per square inch

f_{bl} = computed bending stress at the larger end of the member or unbraced segment thereof, as applicable, kips per square inch

C'_m = coefficient applied to bending term in interaction formula

$$= 1.0 + 0.1 \left(\frac{f_{ao}}{F'_{e\gamma}}\right) + 0.3 \left(\frac{f_{ao}}{F'_{e\gamma}}\right)^2$$

when the member is subjected to end moments which cause single curvature bending and approximately equal computed bending stresses at the ends

$$= 1.0 - 0.9 \left(\frac{f_{ao}}{F'_{e\gamma}}\right) + 0.6 \left(\frac{f_{ao}}{F'_{e\gamma}}\right)^2$$

when the computed bending stress at the smaller end of the unbraced length is equal to zero

When $Kl/r \geq C_c$ and combined stresses are checked incrementally along the length, f_{ao} may be replaced by f_a, and f_{bl} may be replaced by f_b, in Formulas (D4-1a) and (D4-2).

APPENDIX E

Allowable Shear Stresses in Friction-type Connections

SECTION E1 DEFINITIONS

This Specification recognizes nine classes of commercially practical surface conditions (Classes A to I) in friction-type connections, each having distinctive slip-resistant characteristics. These nine classifications, together with corresponding recommended working values, F_v, for friction-type connections assembled with A325 or A490 bolts, are briefly identified in Table E1. The several classes shall conform to the following provisions:

1.   Class A, B, and C surfaces shall be free of oil, paint, lacquer, or other coatings. Contact surfaces for coated joints shall conform to the appropriate conditions as listed below.

2.   Class D, Hot-dip Galvanized and Roughened, shall have the contact surfaces scored by wire brushing or blasting after galvanizing and prior to assembly.

3.   Classes E and F, Blast Cleaned Zinc Rich Paint, shall have blast cleaned contact surfaces coated with organic or inorganic zinc rich paint, as defined in the Steel Structures Painting Council's Paint System Specification SSPC-PS 12.00, *Guide to Zinc-Rich Coating Systems.*

4.   Classes G and H, Blast Cleaned Metallized Zinc or Aluminum, shall have metal applied to the contact surfaces in accordance with the American Welding Society's *Recommended Practice for Metallizing with Aluminum and Zinc for Protection of Iron and Steel,* C2.2, except that subsequent sealing treatments, described in Section IV therein, shall not be used.

5.   Class I, Vinyl Wash, shall have the contact surfaces coated in accordance with the provisions of the Steel Structures Painting Council's Pretreatment Specification SSPC-PT 3, *Basic Zinc Chromate—Vinyl Butyral Washcoat.*

SECTION E2 USE OF HIGHER WORKING STRESSES

Subject to the approval of the responsible engineer, when the contact surfaces of friction-type connections assembled with A325 or A490 bolts meet the provisions of Sect. E1, the allowable working values given in Table E1 may be substituted in lieu of those given Table 1.5.2.1. However, the value thus obtained shall not exceed that specified in Table 1.5.2.1 for a bearing-type connection having bolts of the same size and thread length.

TABLE E1

ALLOWABLE SHEAR STRESSES, KSI,[a] BASED UPON SURFACE
CONDITION OF BOLTED PARTS IN FRICTION-TYPE CONNECTIONS

Class	Surface Condition of Bolted Parts	Standard Holes		Oversized Holes and Short-slotted Holes		Long-slotted Holes	
		A325	A490	A325	A490	A325	A490
A	Clean mill scale	17.5	22.0	15.0	19.0	12.5	16.0
B	Blast-cleaned carbon and low alloy steel	27.5	34.5	23.5	29.5	19.5	24.0
C	Blast-cleaned quenched and tempered steel	19.0	23.5	16.0	20.0	13.5	16.5
D	Hot-dip galvanized and roughened[b]	21.5	27.0	18.5	23.0	15.0	19.0
E	Blast-cleaned, organic zinc rich paint	21.0	26.0	18.0	22.0	14.5	18.0
F	Blast-cleaned, inorganic zinc rich paint	29.5	37.0	25.0	31.5	20.5	26.0
G	Blast-cleaned, metallized with zinc	29.5	37.0	25.0	31.5	20.5	26.0
H	Blast-cleaned, metallized with aluminum	30.0	37.5	25.5	32.0	21.0	26.5
I	Vinyl wash	16.5	20.5	14.0	17.5	11.5	14.5

[a] Values from this table are applicable *only* when they do not exceed the lowest appropriate allowable working stresses for *bearing-type* connections, taking into account the position of threads relative to shear planes and, if required, the 20% reduction due to joint length. (See Table 1.5.2.1.)

[b] If loads causing actual stresses in excess of one-half the tabulated allowable stresses are sustained over a long period of time (e.g., gravity), slip into bearing may occur. If such slip would be severely detrimental, these increased working stresses are not recommended.

Commentary

ON THE SPECIFICATION FOR THE DESIGN, FABRICATION AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS (NOVEMBER 1, 1978)

INTRODUCTION

This Commentary provides information on the basis and limitations of various provisions of the 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 and applying the Documents. Such a recitation could not be made complete and would make the documents unduly lengthy, burdensome, 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.

SECTION 1.2 TYPES OF CONSTRUCTION

In order that adequate instructions can be issued to the shop and erection personnel, the basic assumptions underlying the design must be thoroughly understood by all concerned. As heretofore, these assumptions are classified under three separate but generally recognized types of construction.

For better clarity, the provisions covering tier buildings of Type 2 construction designed for wind loading were reworded in the 1969 Specification, but without change in intent. Justification for these provisions has been discussed by Disque[1] and others.

SECTION 1.3 LOADS AND FORCES

The Specification does not presume to establish the loading requirements for which structures should be designed. In most cases these are adequately covered in the applicable local building codes. Where such is not the case, the generally recognized standards of the American National Standards Institute are recommended as the basis for design.

1.3.3 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 increase in out-of-straightness) will not develop or will be negligible during the short duration of impact.

Ed. Note: See Commentary Appendix for List of References.

SECTION 1.4   MATERIAL

The grades of structural steel approved for use under the Specification, covered by ASTM standard specifications, extend to a yield stress of 100 kips per square inch. 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. When requested to do so, the fabricator must make affidavit that all steel specified has been provided in accordance with the plans and Specification.

In keeping with the inclusion of steels of several strength grades, a number of corresponding ASTM standards for cast steel forgings and other appurtenant materials such as rivets, bolts, and welding electrodes are also included.

Provisions of the Specification are based upon providing a factor of safety against reaching yield stress in primary connected material at working loads. 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 this Commentary adequate information to impart full understanding relative to all factors which might merit consideration in the selection and specification of materials for unique or specially demanding applications. In such situations 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 situation, for example, is the design of highly restrained welded connections.[2] 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 Specification; however, it should be noted that Grade B is intended for pipe flange bolting and Grade 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 kips per square inch, of the weld metal and the final two digits indicate the type coating; however, in the case of mild steel electrodes for submerged arc welding (AWS A5.17), the first one or two digits times 10 indicates the nominal tensile strength classification, while the final

digit or digits times −10 indicates 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.

SECTION 1.5 ALLOWABLE STRESSES*

1.5.1. Structural Steel

Where provisions are given in terms of F_y or F_u together with numerical values, it should be noted that, throughout the Specification, all stresses including the applicable value of F_y or F_u are expressed in kips per square inch.

For ready reference, numerical values are presented in Appendix A for several of the yield stress and ultimate strength levels of the materials in Sect. 1.4.1.

1.5.1.1 Tension

It has been observed that 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.

To prevent failure of a member loaded in tension, Sect. 1.5.1.1 has imposed a factor of 1.67 against yielding of the entire member and 2.0 against fracture of its weakest effective net area.

It is clear that the portion of the member occupied by the net area at fastener holes has a negligible length relative to the total length of the member; thus, yielding of the net area at fastener holes does not constitute a limit state of practical significance. For the very rare case where holes or slots, other than rivet or bolt holes, are located in a tension member, it is conceivable that they could have an appreciable length in the direction of the tensile force. The failure limit states of general yielding on the gross area and fracture on the reduced area are still the principal limit states of concern. However, when the length of the reduced area exceeds the member depth or constitutes an appreciable portion of the member's length, yielding of the net area may become a serviceability limit state meriting special consideration and exercise of engineering judgment.

The mode of failure is dependent upon the ratio of effective net area to gross area and the mechanical properties of the steel. The boundary between these modes, according to the provisions of Sect. 1.5.1.1, is defined by the equation $A_e/A_g = 0.6F_y/0.5F_u$. When $A_e/A_g \geq F_y/0.833F_u$, general yielding of the member will be the failure mode. When $A_e/A_g < F_y/0.833F_u$, fracture at the weakest net area will be the failure mode.

---

* Appendix D covers design provisions for frames consisting of members which are linearly tapered in the plane of their web. While allowable stress provisions for tapered members are basically similar to those provided in this Section, certain modifications are required to account for the effect of the taper. This has resulted in the use of special notations, often defined by algebraic expressions not applicable to prismatic members. Since the use of tapered members is somewhat limited, these notations and the design provisions in which they appear are omitted from Sect. 1.5 and are covered only in Appendix D.

In the case of short fittings used to transfer tensile force, an upper limit of 0.85 is placed on the ratio A_e/A_g. See Sect. 1.14.2.3.

The working stress at the net area at pin holes is based upon research[4] and experience with eye bars.

1.5.1.2 Shear

While the shear yield stress of structural steel has been variously estimated as between $\frac{1}{2}$ and $\frac{5}{8}$ of the tension and compression yield stress and is frequently taken as $F_y/\sqrt{3}$, it will be noted that the permissible working value is given as $\frac{2}{3}$ the recommended basic allowable tensile stress, substantially as it has been since the first edition of the AISC Specification, published in 1923. This apparent reduction in factor of safety is justified by the minor consequences of shear yielding, as compared with those associated with tension and compression yielding, and by the effect of strain hardening.

Although the allowable stress of $0.40F_y$ may be applied over the full area of the beam web, judgment should be used in cases where a connection length is considerably less than the depth of the beam.

The webs of rolled shapes are all of such thickness that shear is seldom the criterion for design. However, the web shear stresses are generally high within the boundaries of the rigid connection of two or more members whose webs lie in a common plane. Such webs should be reinforced when the web thickness is less than

$$\frac{32M}{A_{bc}F_y}$$

where M is the algebraic sum of clockwise and counter-clockwise moments, in kip-feet, applied on opposite sides of the connection boundary, and A_{bc} is the planar area of the connection web, expressed in square inches. This expression is derived for a rigid connection with beam depths of d_b and a column depth of d_c. The moment is represented as a force couple in the beam flanges at an assumed distance of $0.95d_b$, and the resulting web shear stress within the connection is limited to $0.40F_y$. This results in

$$0.40F_y = \frac{12M}{0.95d_b d_c t_{min}}$$

or

$$t_{min} = \frac{32M}{A_{bc}F_y}$$

Recent tests[68] conducted at the University of Toronto have shown that high-strength-bolted beam end connections which subject a coped web to high bearing stresses may cause a tearing failure mode wherein a portion of the beam web tears out along the perimeter of the holes. The tests demonstrated that the failure load can be reliably predicted using an analytical model which combines ultimate shear strength of the net section subject to shear stress with the ultimate tensile strength of the net section subject to tensile stress. As provided in Sect. 1.5.1.2.2, the allowable stress of $0.30F_u$ may be conservatively applied to the entire rupture surface. Alternatively, the tension and shear areas may be treated separately, and the connection allowable capacity in the tearing failure mode taken as follows:

$$0.30A_v F_u + 0.50A_t F_u$$

where A_v and A_t are the net shear and net tension areas, respectively (see Fig. C1.5.1.2).
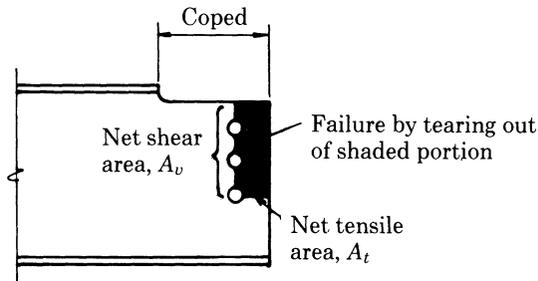
Fig. C1.5.1.2

There may be similar connections, such as thin bolted gusset plates in double shear, where this type of failure could occur. Such situations should be investigated.

1.5.1.3   Compression*

1.5.1.3.1   Formulas (1.5-1) and (1.5-2) are founded upon the basic column strength estimate suggested by the Structural Stability Research Council (formerly the Column Research Council).** This estimate assumes that the upper limit of elastic buckling failure is defined by an average column stress equal to ½ of yield stress. The slenderness ratio C_c, corresponding to this limit, can be expressed in terms of the yield stress of a given grade of structural steel as

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$$

A variable factor of safety has been applied to the column strength estimate to obtain allowable working stresses. For very short columns, this factor has been taken as equal to, or only slightly greater than, that required for members axially loaded in tension, and can be justified by the insensitivity of such members to accidental eccentricities. For longer columns, entering the Euler slenderness range, the factor is increased 15 percent, to approximately the value provided in the AISC Specification since it was first published.

To provide a smooth transition between these limits, the factor of safety has been arbitrarily defined by the algebraic equivalent of a quarter sine curve whose abscissas are the ratio of given Kl/r values to the limiting value C_c, and whose ordinates vary from 5/3 when Kl/r equals 0 to 23/12 when Kl/r equals C_c.

Tables giving the permissible stress for columns and other compression members of 36 and 50 kips per square inch structural steels are included in Appendix A of the Specification for the convenience of the designer.

1.5.1.3.2   Formula (1.5-2), covering slender columns (Kl/r greater than C_c) which fail by elastic buckling, is based upon a constant factor of safety of 23/12 with respect to the elastic (Euler) column strength.

---

* For tapered members, also see Commentary Sect. D2.

** Ref. 5, Eqs. (3.23) and (3.27).

1.5.1.3.3 By dividing the values obtained from Formulas (1.5-1) and (1.5-2) by the factor $[1.6 - (l/200r)]$ when l/r exceeds 120, to obtain Formula (1.5-3), substantially the same allowable stresses are still recommended for bracing and secondary members as those formerly given by the Rankine-Gordon formula which, until 1961, had been included in the AISC Specification since its first adoption.

The more liberal working stress for this type of member was justified in part by the relative unimportance of such members and in part by the greater effectiveness of end restraint likely to be present at their ends.

Since Formula (1.5-3) does take advantage of end restraint, the full unbraced length of the member (rather than a reduced effective length, assuming $K < 1.0$) should always be used, and the formula should be restricted to members which are more or less fixed against rotation and translation at braced points.

1.5.1.4 Bending*

1.5.1.4.1 When flexural members, loaded to produce bending about their strong axis, are proportioned in accordance with the provisions of Sects. 1.9.1.2 and 1.9.2.2, and are adequately braced to prevent the lateral displacement of the compression flange, they provide bending resistance equal at least to the product of their section modulus and yield stress, even when the width-thickness ratio of compressed elements of their profile is such that local buckling may be imminent. Lateral buckling of members bent about their strong axis may be prevented by bracing which either restrains the compression flange against lateral displacement or restrains the cross section against twisting which would induce bending about the weaker axis. Members bent solely about their minor axis, and members having approximately the same strength about both axes, do not buckle laterally and therefore may be stressed to the full allowable bending stress consistent with the width-thickness proportions of their compression elements without bracing.

Research in plastic design has demonstrated that local buckling will not occur in homogeneous sections meeting the requirements of subparagraphs 1 to 5, inclusive, of Sect. 1.5.1.4.1 before the full plastic moment is reached. Practically all W and S shapes of A36 steel and a large proportion of these shapes having a yield stress of 50 kips per square inch meet these provisions and are termed "compact" sections. It is obvious that the possibility of overload failure in bending of such rolled shapes must involve a higher level of stress (computed on the basis of M/S) than members having more slender compression elements. Since the shape factor of W and S beams is generally in excess of 1.12, the allowable bending stress for such members has been raised 10 percent from $0.60F_y$ to $0.66F_y$.

The further provision permitting the arbitrary redistribution of 10 percent of the moment at points of support, due to gravity loading, gives partial recognition to the philosophy of plastic design. Subject to the restrictions provided in Sect. 1.5.1.4.1, continuous framing consisting of compact members may safely be proportioned on the basis of the working stress provisions of Part 1 of the Specification when the moments, before redistribution, are determined on the basis of an elastic analysis. Figure C1.5.1.4 illustrates the application of this provision by comparing calculated moment diagrams with the diagrams as altered by this provision.

Supplement No. 3 (1974) to the 1969 Specification added subparagraph 6, an unsupported length criteria for compact tubular members with rectangular cross sections. The equation recognizes the effect of moment gradient, and tests[6] have shown it to be conservative.

---

*For tapered members, also see Commentary Sect. D3.
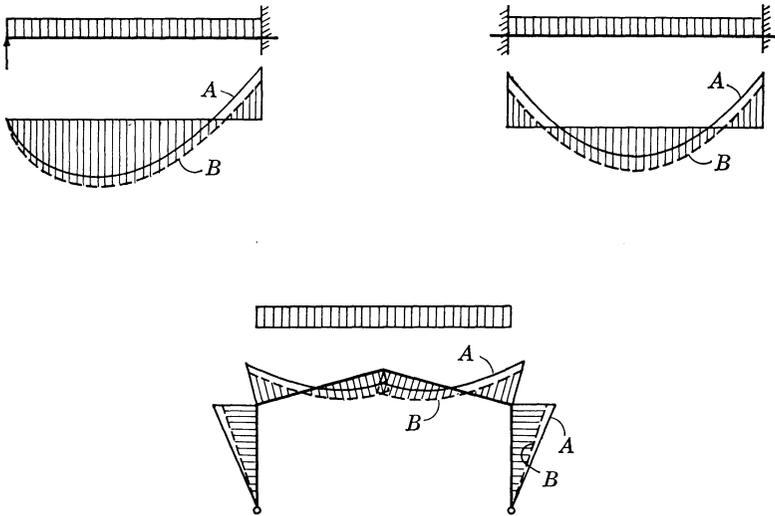
A = Actual moment diagram
B = Modified diagram corresponding to 10 percent moment reduction allowance at
    interior supports

Fig. C1.5.1.4

Subparagraph 7 now permits circular members to be compact and is based on research at the University of Wisconsin—Milwaukee.[6]

In order to assure maximum advantage of moment redistribution, designs should be executed in accordance with the rules for plastic design given in Part 2. However, for many cases commonly encountered, the provisions of Sect. 1.5.1.4.1 afford approximately the same overall economy.

1.5.1.4.2 Formula (1.5-5a) avoids an abrupt transition between an allowable bending stress of $0.66F_y$ when the half-flange width-to-thickness ratio of laterally supported compression flanges exceeds $65/\sqrt{F_y}$, and $0.60F_y$ when this ratio is no more than $95/\sqrt{F_y}$. The assured hinge rotation capacity in this range is too small to permit redistribution of computed moment.

1.5.1.4.3 The 25 percent increase in allowable bending stress for compact sections and solid rectangular bars bent about their weak axis, as well as for square and rectangular bars, is based upon the favorable shape factor present when these sections are bent about their weaker axis, and the fact that, in this position, they are not subject to lateral-torsional buckling. While the plastic bending strength of these shapes, bent in this direction, is considerably more than 25 percent in excess of their elastic bending strength, full advantage is not taken of this fact in order to provide elastic behavior at service loading.

Formula (1.5-5b), like Formula (1.5-5a), is a transition formula between the high allowable bending stress of $0.75F_y$ at $b_f/2t_f = 65/\sqrt{F_y}$ and the lower stress of $0.60F_y$ at $b_f/2t_f = 95/\sqrt{F_y}$.

1.5.1.4.4 Box-type members are torsionally very stiff.* The critical flexural stress due to lateral-torsional buckling, for the compression flange of a box-type beam loaded in the plane of its minor axis so as to bend about its major axis, can be obtained using Formula (1.5-1) with an equivalent slenderness ratio, by the expression

$$\left(\frac{l}{r}\right)_{equiv} = \sqrt{\frac{5.1 l S_x}{\sqrt{J I_y}}}$$

where l is the distance between points of lateral support and S_x, I_y and J are, respectively, the major axis section modulus, minor axis moment of inertia, and the torsional constant of the beam cross section. It can be shown that, when $d < 10b$ and $l/b < 2500/F_y$, the allowable compression flange stress indicated by the above equation will approximate $0.60F_y$. Beyond this limit, deflection rather than stress is likely to be the design criterion.

1.5.1.4.5 The allowable bending stress for all other flexural members is given as $0.60F_y$, provided the member is braced laterally at relatively close intervals $(l/b_f \leq 76/\sqrt{F_y})$.

Members bent about their major axis and having an axis of symmetry in the plane of loading may be adequately braced laterally at greater intervals if the maximum bending stress is reduced sufficiently to prevent premature buckling of the compression flange. Mathematical expressions affording an exact estimate of the buckling strength of such members, which take into account their torsional rigidity about their longitudinal axis (St. Venant torsion) as well as the bending stiffness of their compression flange between points of lateral support (warping torsion), are too complex for general design office use. Furthermore, their accuracy is dependent upon the validity of assumptions regarding restraint at points of lateral support and conditions of loading which, at best, can be no more than engineering judgments.

The combination of Formulas (1.5-6a) or (1.5-6b) and (1.5-7) provides a reasonable design criterion in convenient form. Formulas (1.5-6a) and (1.5-6b) are based on the assumption that only the bending stiffness of the compression flange will prevent the lateral displacement of that element between bracing points.

Formula (1.5-7) is a convenient approximation which assumes the presence of both lateral bending resistance and St. Venant torsional resistance. Its agreement with more exact expressions for the buckling strength of intermittently braced flexural members** is closest for homogeneous sections having substantial resistance to St. Venant torsion, identifiable in the case of doubly-symmetrical sections by a relatively low d/A_f ratio. Due to the difference between flange and web yield strength of a hybrid girder, it is desirable to base the lateral buckling resistance solely on warping torsion of the flange. Hence, use of Formula (1.5-7) is not permitted for such members.

For some sections having a compression flange area distinctly smaller than the tension flange area, Formula (1.5-7) may be unconservative; for this reason, its use is limited to sections whose compression flange area is at least as great as the tension flange. In plate girders, which usually have a much higher d/A_f ratio than rolled W shapes, Formula (1.5-7) may err grossly on the conservative side. For such members, the larger stress permitted by Formula (1.5-6a), and at times

---

* Ref. 5, Sect. 6.2.
** Ibid., Eq. (6.8).

by Formula (1.5-6b), affords the better estimate of buckling strength. While these latter formulas underestimate the buckling strength somewhat because they ignore the St. Venant torsional rigidity of the profile, this rigidity for such sections is relatively small and the margin of overconservatism, therefore, is likewise small.

It should be noted that Formula (1.5-7) is written for the case of elastic buckling. A transition is not provided for this formula in the inelastic stress range because, when actual conditions of load application and variation in bending moment are considered, any unconservative error without the transition must be small.

Singly-symmetrical, built-up, I-shape members, such as some crane girders, often have an increased compression flange area in order to resist bending due to lateral loading acting in conjunction with the vertical loads. Such members usually can be proportioned for the full permissible bending stress when the stress is produced by the combined vertical and horizontal loading. Where the failure mode of a singly-symmetrical I-shape member having a larger compression than tension flange would be by lateral buckling, the permissible bending stress can be obtained by using Formula (1.5-6a) or (1.5-7).

Through the introduction of the modifier* C_b, some liberalization in stress is permissible when there is moment gradient over the unbraced length, except that C_b must be taken as unity when computing F_{bx} for use in Formula (1.6-1a) for frames braced against joint translation.

Formulas (1.5-6a) and (1.5-6b) may be refined to include both St. Venant and warping torsion by substituting a derived value for r_T. The equivalent radius of gyration, r_{Tequiv}, can be obtained by equating the appropriate expression giving the critical elastic bending stress for the compression flange of a beam** with that of an axially loaded column.[†]

For the case of a doubly-symmetrical I-shape beam,

$$(r_{Tequiv})^2 = \frac{I_y}{2S_x} \sqrt{d^2 + \frac{0.156 l^2 J}{I_y}}$$

where I_y is the minor axis moment of inertia of the member, S_x is its major axis section modulus, and

$$J = \frac{2 b_f l_f^3}{3} + \frac{d t^3}{3}$$

1.5.1.5 Bearing

1.5.1.5.1 As used throughout the 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. The recommended bearing stress on pins is not the same as for bolts and rivets. The lower value, $\frac{9}{10}$ of the yield stress of the part containing the pin hole, provides a safeguard against instability of the plate beyond the hole[4] and high bearing stress concentration which might interfere with operation of the pin, but which is of no concern with rivets and bolts.

---

* Ref. 5, Eq. (6.13).

** Ibid., Eq. (6.9b), (6.21), (6.22), or (6.23).

† Ibid., Eq. (4.2).

1.5.1.5.3 Bearing values are provided, not as a protection to the fastener, because it needs no such protection, but for the protection of the connected parts. Therefore, the same bearing value applies to joints assembled by rivets or by bolts, regardless of fastener shear strength or the presence or absence of threads in the bearing area.

It should be noted that the value for bearing stress, $1.5F_u$, is the *maximum* allowable value. As explained under Sects. 1.16.4 and 1.16.5 of this Commentary, this maximum value is permitted only if the end distance and intermediate spacing of fasteners, measured in the direction of applied force, are adequate to prevent failure by splitting of a connected part parallel to the line of force at a load less than required to cause transverse fracture through the net area of the part.

While the possibility of a friction-type shear connection slipping into bearing under anticipated service conditions is extremely remote, such connections should comply with the provisions of Sects. 1.5.2.2 and 1.16.4, in order to insure the usual minimum factor of safety of 2.0 against complete connection failure.

1.5.2   Rivets, Bolts, and Threaded Parts

The numerous revisions dealing with provisions for mechanical fasteners are based upon an extended review and reexamination of the large body of data growing out of voluminous research, both here and abroad, concerned with the behavior of mechanically fastened connections, which has been completed in the past two decades. In order to consolidate and organize this material and, for the convenience of the engineering profession, to present concise, rational and well balanced conclusions within the covers of a single volume, the Research Council on Riveted and Bolted Structural Joints in 1970 sponsored the preparation of *Guide to Design Criteria for Bolted and Riveted Joints,*[7] which was published in 1974. (In subsequent references this publication will be noted as the "Guide.")

The Guide has since provided the background for a revision of the *Specification for Structural Joints Using ASTM A325 or A490 Bolts* of the Research Council on Riveted and Bolted Structural Joints, approved April 26, 1978. It likewise has been the basis for the revision of AISC Specification provisions concerning mechanically fastened structural connections.

At the outset, the Guide notes a distinction between a factor of safety adequate to prevent loss of usefulness of a structure, member, or connection, and one needed to insure against complete failure of these entities.* In the latter case, it notes that, under the long-standing misconception of "balanced design", when the weakest element of a joint has a factor of safety of 2.0, other elements may be grossly overdesigned,** with attendant loss in economy.

The balanced design concept may have been valid when there was but one grade of structural steel and but one grade of fastener. However, it has lost its meaning with today's multiplicity of both fastener and connected material strengths.

Based on the earlier criteria, the weakest component in some of the largest and most important joints of existing structures have a factor of safety no greater than 2.0, yet they have proven with time to be entirely satisfactory. The Guide has adopted this value as basic with respect to failure, increasing it somewhat in rounding off to even working stress values or, as in the case of slip-resistance, reducing it somewhat when impairment of usefulness alone is at stake. With con-

---

* Ref. 7, p. 17.
** Ibid., p. 125.

siderable accumulation of data now available as to the effectiveness of joint components under various loading conditions, probabilistic methods of statistical analysis have been used in determining the critical stress to which the factor of safety should be applied.*

1.5.2.1 Tension

As in earlier editions, permissible stresses for rivets are given in terms applicable to the nominal cross-sectional area of the rivet before driving. For convenience in the proportioning of high-strength-bolted connections, permissible stresses for bolts and threaded items are given in terms applicable to their nominal body area, i.e., the area of the threaded part based on its major diameter.

Except as provided in Appendix Sect. B3, any additional fastener tension resulting from prying action due to distortion of the connection details should be added to the stress calculated directly from the applied tension in proportioning fasteners for an applied tensile force, using the specified working stresses. Depending upon the relative stiffness of the fasteners and the connection material, this prying action may be negligible or it may be a substantial part of the total tension in the fasteners.**

1.5.2.1 Shear

Mechanically fastened connections which transmit load by means of shear in their fasteners are categorized as either "friction-type" or "bearing-type". The former depend upon sufficiently high clamping force to prevent slip of the connected parts under anticipated service conditions. The latter depend upon contact of the fasteners against the sides of their holes to transfer the load from one connected part to another.

The amount of clamping force developed by shrinkage of a rivet after cooling and by A307 bolts is unpredictable and generally insufficient to prevent complete slippage at the permissible working stress. Riveted connections and connections made with A307 and A449 bolts for shear are treated as bearing-type. The high clamping force produced by properly tightened A325 and A490 bolts is sufficient to assure that slip will not occur at full allowable stress in friction-type connections and probably will not occur at working loads in bearing-type connections.

Given a prescribed minimum clamping force provided by pretensioning of the bolts, the force required to produce slip is greatly influenced by the conditions of the contact surfaces of the connected parts. In earlier editions of the Specification, the allowable stress for fasteners in friction-type connections (for convenience treated as shear stress) was based upon observed performance when these surfaces consisted of tight mill scale, which offered the least slip resistance of any commercially practical untreated surface condition. Specifications in other countries, however, capitalized on the superior slip resistance of surfaces subject to treatments such as, for example, blast cleaning.

With the phenomenal increase in the use of high-strength bolted connections, much experience and test data concerning the efficiency of various surface preparations in resisting slip have been accumulated. As a result, it has been possible, through the use of statistical analyses,† to determine, with a high degree of reliability, working stresses applicable to several commercially practical surface

---

* Ref. 7, p. 18.
** Ibid., pp. 269–274, and Ref. 8.
† Ibid., pp. 75–83, 208, and 209.

preparations providing the same resistance to slip as that produced by tight mill scale.* However, the cost of surface preparation relative to the saving in bolts due to the increase in working stress varies and may be high; therefore, the requiring of special surface treatment to reduce the number of fasteners might increase cost. On the other hand, if special surface preparations are required for other reasons, the use of higher allowable stresses, and hence fewer fasteners, is fully justified. For this reason the provisions for their use have been presented in Appendix E, subject to the approval of the designer.

The limitation placed on the use of Class D, hot-dip galvanized surface condition, by footnote b in Table E1, is based upon the observed tendency for these surfaces to slip somewhat under long-sustained full load. Under random loading, however, they tend to lock into a stable state.

The working values given in Table 1.5.2.1 for friction-type and bearing-type shear connections are, with only minor modifications based on reliability analysis of existing data, equivalent to those in previous editions of the AISC Specification for use with A325 and A490 bolts in standard or slotted holes with tight mill scale surfaces.

The requirement of footnote f in Table 1.5.2.1, which calls for a 20 percent reduction in allowable fastener shear stress, as noted in the Guide,** is based upon tests on butt-type splice specimens where all connected parts were loaded in tension. This footnote provision would not apply to connection angles at the ends of plate girders which transmit the girder reaction to the supporting member by means of shear in the connection angles. Nor would the distance between extreme fasteners in tension members connected at opposite edges of a gusset plate govern; instead, the length of the connection for each tension member would control the design.

While ASTM Standard A449 calls for the same mechanical properties as those specified for A325 bolts, the quality control provisions specified for the latter type are more stringent. Hence, A449 bolts are not permitted in friction-type connections.

Bearing-type connections are intended for use where service conditions are such that cyclic loading approaching complete stress reversal will not occur, and deformation of the structural frame or a component thereof, due to slip of the connection into bearing, can be tolerated. The allowable stresses in this case are based upon a factor of safety of 2.0 or more, which over a long period has been found to be adequate. This is substantially higher than that which is basic to the design of the connected members.

The efficiency of threaded fasteners in resisting shear in bearing-type connections is reduced when the threading extends into the shear plane between the connected parts. Except in the case of A307 bolts, two allowable shear stress values are given: one when threading is excluded from the shear plane and one when it is not. In selecting appropriate allowable shear stresses, it was deemed an unwarranted refinement to make a distinction between threads in a single shear plane and threads in two planes (double shear of an enclosed part). Therefore, the allowable stresses were established on the conservative assumption of threads in two planes. Because it is not customary to control this feature in the case of A307 bolts, and because the length of threading on A307 and A449 bolts is greater than on A325 and A490 bolts, it is assumed that threading may extend into the

---

* The analyses in the Guide (Ref. 7) are based upon pretension produced by the turn-of-nut method of installation, which is somewhat greater than that provided by calibrated wrench installation.   Values in Table E1 (Appendix E) are based on pretension by a calibrated wrench and, hence, are slightly lower.
** Ref. 7, pp. 84–96.

shear plane and the allowable shear value, applicable to the gross area, is reduced accordingly.

1.5.3   Welds

As in the past, the allowable working stresses for statically loaded full-penetration welds are the same as those permitted for the base metal, provided the mechanical properties of the electrodes used are such as to match or exceed those of the weakest grade of base metal being joined.

On the basis of physical tests,[9] the allowable stress on fillet welds deposited on "matching" base metal, or on steel having mechanical properties higher than those specified for such base metal, has been given in terms of the nominal tensile strength* of the weld metal since the 1969 edition of the Specification.

As in the past, the same working value is given to a transverse as to a longitudinal weld, even though the force that the former can resist is substantially greater than that of the latter.   In the case of tension on the throat of partial-penetration groove welds normal to their axis (more nearly analogous to that of transverse than longitudinal fillets), the working stress is conservatively taken the same as for fillet welds.

When partial-penetration groove welds are so disposed that they are stressed in tension parallel to the longitudinal axis of the groove, or primarily in compression or bearing, they may be proportioned to resist such stress at the same unit value permitted in the base metal.

1.5.4   Cast Steel and Steel Forgings

In keeping with the inclusion of high strength low-alloy steels, the Specification recognizes high strength steel castings.   Allowable stresses are expressed in terms of the specified minimum yield stress for castings.

1.5.5   Masonry Bearing

In Supplement No. 3 to the Specification (1974), higher allowable stresses were adopted for bearing on concrete.   These new provisions were derived from ACI Standard 318-71[10] ultimate strength criteria, using a load factor of 1.7 applied to both live and dead load.   These provisions are more conservative than the ACI ultimate strength provisions, wherein a load factor of 1.4 is permitted for dead load.

SECTION 1.6   COMBINED STRESSES

1.6.1   Axial Compression and Bending

The application of moment along the unbraced length of axially loaded members, with its attendant axial displacement in the plane of bending, generates a secondary moment equal to the product of resulting eccentricity and the applied axial load, which is not reflected in the computed stress f_b.   To provide for this added moment in the design of members subject to combined axial and bending stress, Formula (1.6-1a) requires that f_b be amplified by the factor

$$\frac{1}{\left(1 - \frac{f_a}{F'_e}\right)}$$

---

* See Commentary Sect. 1.4.

Depending upon the shape of the applied moment diagram (and, hence, the critical location and magnitude of the induced eccentricity), this factor may overestimate the extent of the secondary moment. To take care of this condition the amplification factor is modified, as required, by a reduction factor C_m.

When bending occurs about both the x- and y-axes, the bending stress calculated about each axis is adjusted by the value of C_m and F'_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 stress permitted for bending about that axis, with due regard to the unbraced length of compression flange where this is a factor.

When the computed axial stress is no greater than 15 percent of the permissible axial stress, the influence of

$$\frac{C_m}{\left(1 - \dfrac{f_a}{F'_e}\right)}$$

is generally small and may be neglected, as provided in Formula (1.6-2). However, its use in Formula (1.6-1a) is not intended to permit a value of f_b greater than F_b when the value of C_m and f_a are both small.

Depending upon the slenderness ratio of the given unbraced length of a member in the plane of bending, the combined stress computed at one or both ends of this length may exceed the combined stress at all intermediate points where lateral displacement is created by the applied moments. The limiting value of the combined stress in this case is established by Formula (1.6-1b).

The classification of members subject to combined axial compression and bending stresses is dependent upon two conditions: the stability against sidesway of the frame of which they are an integral part, and the presence or absence of transverse loading between points of support in the plane of bending.

Note that f_b is defined as the computed bending stress *at the point under consideration.* In the absence of transverse loading between points of support, f_b is computed from the larger of the moments at these points of support. When intermediate transverse loading is present, the larger moment at one of the two supported points is used to compute f_b for use in Formula (1.6-1b). However, to investigate the possibility of buckling failure, the maximum moment between points of support is used to compute f_b for use in Formula (1.6-1a).

Three categories are to be considered in computing values of C_m.

Category 1 covers columns in frames.subject to sidesway, i.e., frames which depend upon the bending stiffness of their several members for overall lateral stability. For determining the value of F_a and F'_e, the effective length of such members, as discussed hereinafter under Sect. 1.8, is never less than the actual length, unbraced in the plane of bending, and may be greater than this length. The actual length is used in computing moments. For this case the value of C_m can be taken as

$$C_m = 1 - 0.18 f_a / F'_e$$

However, under the combination of compression stress and bending stress most affected by the amplification factor, a value of 0.15 can be substituted for $0.18 f_a / F'_e$. Hence, a constant value of 0.85 is recommended for C_m here.*

Category 2 applies to columns not subject to transverse loading in frames where sidesway is prevented. For determining the value of F_a and F'_e, the effective length of such members is never greater than the actual unbraced length and may be somewhat less. The actual length is used in computing moments.

---

* See Commentary Sect. 1.8 for cases where C_m for unbraced frames 10 to 40 stories high may be computed as for braced frames.

For this category, the greatest eccentricity, and hence the greatest amplification, occurs when the end moments, M_1 and $-M_2{}^*$ are numerically equal and cause single curvature. It is least when they are numerically equal and of a direction to cause reverse curvature.

To properly evaluate the relationship between end moment and amplified moment, the concept of an equivalent moment, M_e, to be used in lieu of the numerically smaller end moment, has been suggested. M_e can be defined as the value of equal end moments of opposite signs which would cause failure at the same concurrent axial load as would the given unequal end moments. Then, M_e/M_2 can be written** in terms of $\pm M_1/M_2$ as

$$\frac{M_e}{M_2} = C_m = \sqrt{0.3\left(\frac{M_1}{M_2}\right)^2 - 0.4\left(\pm\frac{M_1}{M_2}\right) + 0.3}$$

It has been noted[11] that the simpler formulation

$$C_m = 0.6 - 0.4\left(\pm\frac{M_1}{M_2}\right) \geq 0.4$$

affords a good approximation to this expression. When M_1/M_2 is greater than 0.5, the combined axial and bending stress is usually limited by general yielding rather than by stability, in which case Formula (1.6-1b) would govern. Therefore, a tentatively selected column section should be tested by both Formulas (1.6-1a) and (1.6-1b).

Category 3 is exemplified by the compression chord of a truss subject to transverse loading between panel points, or by a simply supported column subjected to transverse loads between supports. For such cases, the value of C_m can be approximated using the equation

$$C_m = 1 + \psi \frac{f_a}{F'_e}$$

Values of ψ for several conditions of transverse loading and end restraint (simulating continuity at panel points) are given in Table C1.6.1, together with two cases of simply supported beam-columns. In the case of continuity at panel points, f_b is maximum at the restrained ends or end, and the value of C_m for usual f_a/F'_e ratios is only slightly less than unity (a value of 0.85 is suggested in the Specification in the final paragraph of Sect. 1.6.1). For determinate (simply supported) beam-columns, f_b is maximum at or near midspan, depending upon the pattern of transverse loading. For this case

$$\psi = \frac{\pi^2 \delta_o EI}{M_o L^2} - 1$$

where

δ_o = maximum deflection due to transverse loading
M_o = maximum moment between supports due to transverse loading

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, δ_o should include the deflection between supports produced by this moment.

---

* The sign convention for moments here and in Sect. 1.6 is that generally used in frame analysis. It should not be confused with the beam sign convention used in many textbooks. Moments are considered positive when acting clockwise about a fixed point, negative when acting counter-clockwise.

** Ref. 5, p. 202.

TABLE C1.6.1
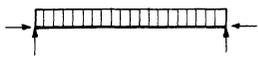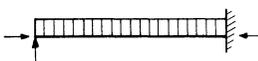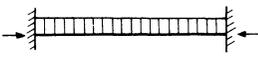
Case	ψ	C_m
	0	1.0
	-0.4	$1 - 0.4\dfrac{f_a}{F'_e}$
	-0.4	$1 - 0.4\dfrac{f_a}{F'_e}$
	-0.2	$1 - 0.2\dfrac{f_a}{F'_e}$
$\leftarrow L/2 \rightarrow$	-0.3	$1 - 0.3\dfrac{f_a}{F'_e}$
	-0.2	$1 - 0.2\dfrac{f_a}{F'_e}$
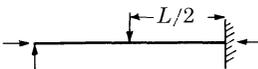
It should be noted that, for amplified end moments in indeterminate members, stress alone is critical and is controlled by Formula (1.6-1b). For determinate members, where the amplified bending stress is maximum between supports, buckling-type failure is also of concern.

Note that F_a is governed by the maximum slenderness ratio, regardless of the plane of bending. F'_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.

1.6.2 Axial Tension and Bending

Contrary to the behavior in compression members, axial tension tends to *reduce* the bending stress because the secondary moment, which is the product of the deflection and the axial tension, is opposite in sense to the applied moment; thus, the secondary moment diminishes, rather than amplifies, the primary moment.

R = numerical coefficient

$$f_t = \frac{F_t}{F_v} \sqrt{F_v{}^2 - f_v{}^2}$$
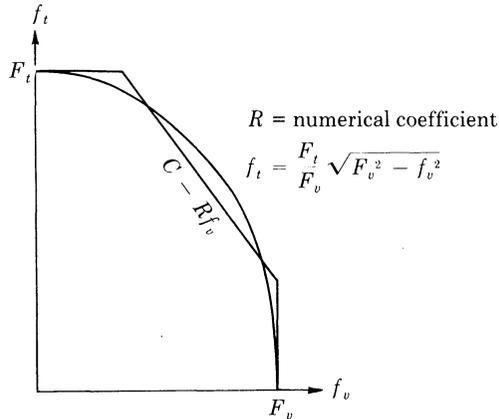
Fig. C1.6.3

1.6.3   Shear and Tension

Tests have shown that the strength of bearing-type fasteners subject to combined shear and tension resulting from externally applied forces can be closely defined by an ellipse constructed with major and minor axis half-lengths equal, respectively, to the tension-type and shear-type stresses given in Sect. 1.5.2.*

Such a curve can be replaced, with only minor deviations, by three straight lines, as shown in Fig. C1.6.3. This latter representation offers the advantage that no modification of either type stress is required in the presence of fairly large magnitudes of the other type. Therefore, it is the one upon which the linear formulas in Sect. 1.6.3, giving F_t as a function of f_v for bearing-type connections, are based. In no case is the "unconservative" deviation more than 5 percent, nor the "conservative" deviation more than about 10 percent.

In the case of friction-type connections subject to combined tension and shear at the contact surface common to a beam connection and the supporting member, where the fastener tension, f_t, is produced by moment in the plane of the beam web, the shear component may be neglected in proportioning the fasteners for tension, because the shear component assigned to the fasteners subject to direct tensile stress is picked up by the increase in compressive force on the compression side of the beam axis, resulting in no actual shear force on the fasteners in tension.

However, when a friction-type connection must resist an axially applied tensile force, the clamping force is reduced and F_v must be reduced in proportion to the loss of pretension.

SECTION 1.7   MEMBERS AND CONNECTIONS SUBJECT TO REPEATED VARIATION OF STRESS (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 B.

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

---

* Ref. 7, pp. 33–35, 53, and 54.

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 B 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 $1\frac{1}{2}$ times the applicable value given in Table B3 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 "compression", formerly designated by "C", has been removed from the column headed by "Kind of Stress" in Table B2 of Appendix B. 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.

Recent extensive test programs[12,13] 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 kips per square inch do not exhibit significantly different fatigue strength for given welded details fabricated in the same manner.

The use of a constant stress *range*, which can be read directly from a table for a particular category and loading condition,* greatly simplifies designs involving fatigue, when compared with designs based on maximum or minimum allowable stress obtained from fatigue strength formulas on the basis of a stress *ratio*.

In the previous editions of the AISC Specification, provisions dealing with the fatigue of fasteners were limited to notes interspersed throughout the Specification where applicable. In this edition, these provisions are assembled in a new section of Appendix B, Sect. B3, which is devoted to the topic of fatigue prevention for mechanical fasteners.

Tests have uncovered dramatic differences in fatigue life, not completely predictable from the various published formulas for estimating the actual magnitude of prying force.** To limit the uncertainties regarding prying action on the fatigue behavior of these bolts, the tensile stresses given in Table 1.5.2.1 are approved for use under extended cyclic loading only if the prying force, included

---

* See Table B3, Appendix B.
** Ref. 7, p. 267

in the design tensile force, is small. When this cannot be assured, the allowable 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 B.

SECTION 1.8   STABILITY AND SLENDERNESS RATIOS

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[5] 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.

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 each story, in the columns and beams of the lateral load resisting systems. 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, an effect which should be accounted for in frame design is the $P\Delta$ effect. Similarly, increases in axial forces occur in the members of the bracing systems of braced frames; however, such effects are usually less significant.

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. While the concept is completely valid, its practical implementation involves several assumptions of idealized conditions which will be mentioned later.

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. C1.8.1), the effective length of these columns will exceed their 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 non-existent are tabulated in Table C1.8.1. Also shown are suggested design values recommended by the Structural Stability Research Council (formerly the Column Research Council) for use when
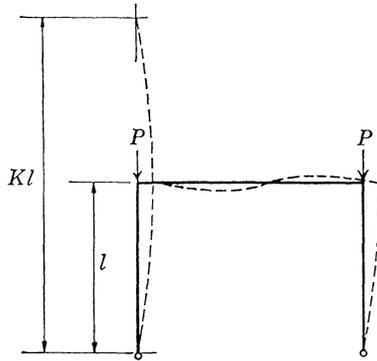
Fig. C1.8.1

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 C1.8.1 were truly pinned, K would actually exceed 2.0 for a frame such as that pictured in Fig. C1.8.1, 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[14] 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).

TABLE C1.8.1

	(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				

    While ordinarily the existence of masonry walls provides enough lateral support for tier 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 in-plane 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 by means of which the effective length of the columns in a laterally unbraced frame can be estimated with sufficient accuracy. These range from simple interpolation between the idealized cases shown in Table C1.8.1 to very complex analytical procedures. Once a trial selection of framing members has been made, the use of the alignment chart in Fig. C1.8.2 affords a fairly rapid method for determining adequate K-values.

---

* Ref. 15, pp. 260–265.

G_A	K	G_B

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.

Sidesway Uninhibited

Alignment Chart for Effective Length of Columns in Continuous Frames

Fig. C1.8.2

However, it should be noted that this alignment chart is based upon assumptions of idealized conditions which seldom exist in real structures.* 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, overly conservative designs may result. There are design procedures available[16,17] which may be used in the calculation of G for use in Fig. C1.8.2 to give results more truly representative of conditions in real structures.

Research at Lehigh University[18,19] on the load-carrying capacity of regular rectangular rigid frames has shown that it is not always necessary to directly account for the $P\Delta$ effect for a certain class of adequately stiff rigid frames. In the research, second order analyses using different load sequences to failure were used to confirm the adequacy of alternate allowable stress design procedures. The loading sequences used in the second order analysis were:

1. Constant gravity load at a load factor of 1.0 while the lateral load was progressively increased.
2. Constant gravity load at a load factor of 1.3 while the lateral load was progressively increased.
3. Both the lateral and gravity loads were progressively increased proportionately.

The seven frames included in the study were 10 to 40 stories high and in-plane column slenderness ratios, h/r_x, ranged from 18 to 42. The live load, including partitions, varied from 40 to 100 psf and the dead load from 50 to 75 psf. A uniform wind load of 20 psf was specified throughout. All beams and column sections were compact. The axial load ratios, f_a/F_a and $f_a/0.60F_y$, were limited to not more than 0.75.

The results of the second order analyses showed that adequate strength and stability were assured under combined gravity and lateral loads or gravity load alone, when the rigid frames were designed by either a stress design procedure according to AISC Specification requirements or by a modified stress design procedure. The modified allowable stress design procedure incorporated a stiffness parameter** which assured adequate frame stiffness, while the effective length factor, K, was assumed to be unity in calculations of f_a and F'_e, and the coefficient C_m was computed as for a braced frame.

---

* Ref. 5, pp. 418–428.

** A design procedure based only upon a first order drift index may not assure frame stability.

Several other references* are available concerning alternatives to effective length factors for multistory frames under combined loads or gravity loads alone.

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 interaction formulas of Sect. 1.6.1. 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.[25]

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 as less than the distance between panel points, it is usual practice to take K as equal to 1.0,** 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.

The slenderness limitations recommended for tension members are not essential to the structural integrity of such members; they merely afford a degree of stiffness such that undesirable lateral movement ("slapping" or vibration) will be avoided. These limitations are not mandatory.

SECTION 1.9 WIDTH-THICKNESS RATIOS

When the width-thickness ratio of the compressed elements in a member does not exceed the applicable limit specified in Sect. 1.9.1.2, 1.9.2.2, or 1.9.2.3, no reduction in allowable stress is necessary in order to prevent local buckling. Appendix C provides a design procedure for those infrequent situations where width-thickness ratios in excess of the limits given in Sect. 1.9 are involved.

Formulas (C2-1) to (C2-6) are based upon the following expression[5] for critical buckling stress, σ_c, for a plate supported against lateral deflection along one or both edges, with or without torsional restraint along these edges and subject to in-plane compressive force:

$$\sigma_c = k\left[\frac{\pi^2 E\sqrt{\eta}}{12(1-\nu^2)(b/t)^2}\right]$$

where η is the ratio of the tangent modulus to the elastic modulus, E_t/E, and ν is Poisson's ratio. The assumption of nothing more than knife-edge lateral support applied along one edge of the unstiffened element under a uniformly distributed stress (the most critical case) would give a value of $k = 0.425$. Some increase in this value is warranted because of the torsional restraint provided by the supporting element and because of the difference between b, as defined in Sect. 1.9.1.2, and the theoretical width b.

---

* Refs. 16, 20, 21 (pp. 1643–1655), 22, 23, and 24.
** Ref. 5, pp. 75–77, for K less than unity in trusses.
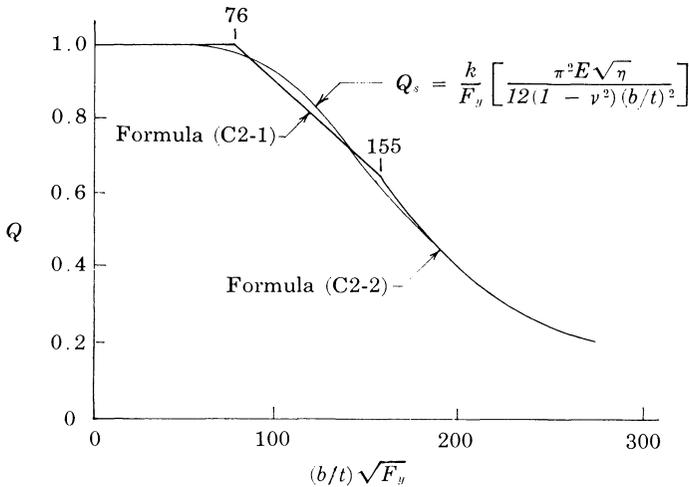
Fig. C1.9.1

In the interest of simplification, when $\sqrt{\eta} < 1.0$, a linear formula is substituted for the theoretical expression. Its agreement with the latter may be judged by the comparison shown in Fig. C1.9.1.

Formula (C2-5) recognizes that the torsional restraint characteristics of tees cut from rolled shapes might be of quite different proportions than those of tees formed by welding two plates together.

It has been shown[26] that singly-symmetrical members whose cross section consists of elements having large width-thickness ratios may fail by twisting under a smaller axial load than that associated with general column failure. Such is not generally the case with hot-rolled shapes. To guard against this type of failure, particularly when relatively thin-walled members are fabricated from plates, Table C1 in Appendix C places an upper limit on the proportions permissible for channels and tees.

With both edges parallel to the applied load supported against local buckling, stiffened compression elements can support a load producing an average stress, σ_c, greater than that given in the above expression for critical plate buckling stress. This is true even when k is taken as 4.0, applicable to the case where both edges are simply supported, or a value between 4.0 and 6.97, applicable when some torsional restraint is also provided along these edges.

A better estimate of the compressive strength of stiffened elements, based upon an "effective width" concept, was first proposed by von Karman.* This was later modified by Winter[28] to provide a transition between very slender elements and stockier elements shown by tests to be fully effective.

As modified, the ratio of effective width to actual width increases as the level of compressive stress applied to a stiffened element in a member is decreased, and takes the form

$$\frac{b_e}{t} = 1.9 \sqrt{\frac{E}{f}} \left[1 - \frac{C}{(b/t)} \sqrt{\frac{E}{f}} \right]$$

---

* Ref. 27, p. 53.

where f is the level of uniformly distributed stress to which the element would be subjected based upon the design of the member, and C is an arbitrary constant based on engineering judgment supported by observed test results.

Obviously, holding the effective width of a stiffened element to no greater value than given by the limits provided in Sect. 1.9.2.2 is unnecessarily conservative when the maximum uniformly distributed design stress is substantially less than $0.60F_y$, or when the ratio b/t is considerably in excess of the limit given in Sect. 1.9.2.2.

For the case of square and rectangular box sections, the sides of which, in their buckled condition, afford negligible torsional restraint for one another along their corner edges, the value of C reflected in Formula (C3-1) is higher than for the other case, thereby providing a slightly more conservative evaluation of effective width. For cases where appreciable torsional restraint is provided, as for example the web of an I-shape column, the value of C implicit in Formula (C3-2) is decreased slightly. As in earlier editions of the AISC Specification, for such cases no reduction from actual width is required when the width-thickness ratio does not exceed $253/\sqrt{F_y}$, and for greater widths the effective width may be taken as equal to $253t/\sqrt{F_y}$. If the actual width-thickness ratio is substantially greater than $253/\sqrt{F_y}$, however, a larger effective width can be obtained using Formula (C3-2) rather than the earlier provisions.

In computing the section modulus of a member subject to bending, the area of stiffened elements parallel to the axis of bending and subject to compressive stress must be based upon their effective, rather than actual, width. In computing the effective area of a member subject to axial loading, the effective, rather than actual, area of all stiffened elements must be used. However, the radius of gyration of the *actual* cross section together with the form factor Q_a may be used in determining the allowable axial stress. If the cross section contains an unstiffened element, the allowable axial stress must be modified by the reduction factor Q_s.

The classical theory of longitudinally compressed cylinders overestimates the actual buckling strength, often by 200 percent or more. Inevitable imperfections of shape and the axiality of load are responsible for the reduction in actual strength below theoretical strength. The limits of Sect. 1.9.2.3 are based upon test evidence,[6] rather than theoretical calculations, that local buckling will not occur if the D/t ratio is equal to or less than $3300/F_y$ when the applied stress is equal to F_y. When D/t exceeds $3300/F_y$, but is less than $13,000/F_y$, Formula (C3-3) provides for a reduction in allowable stress with a factor of safety against local buckling of at least 1.67. The Specification contains no recommendations for allowable stresses when D/t exceeds $13,000/F_y$.

SECTION 1.10 PLATE GIRDERS AND ROLLED BEAMS

1.10.1 Proportions

As in earlier editions of the Specification, it is provided that flexural members be proportioned to resist bending on the basis of the moment of inertia of their gross cross section, with the stipulation that holes in the flanges having an area in excess of 15 percent of the gross flange area must be deducted.[29] This provision includes the design of hybrid flexural members whose flanges are fabricated from a stronger grade of steel than that in their web. As in the case of flexural members having the same grade of steel throughout their cross section, their bending strength is defined by the product of the section modulus of the gross cross section multiplied by the allowable bending stress. On this basis, the stress in the web

at its junction with the flanges may even exceed the yield stress of the web material, but under strains controlled by the elastic state of stress in the stronger flanges. Numerous tests[30] have shown that, with only minor adjustment in the basic allowable bending stress as provided in Formula (1.10-5), the bending strength of a hybrid member is predictable within the same degree of accuracy as is that of a homogeneous member.

1.10.2   Web

The limiting web depth-thickness ratio to prevent vertical buckling of the compression flange into the web, before attainment of yield stress in the flange due to flexure, may be increased when transverse stiffeners are provided, spaced not more than $1\frac{1}{2}$ times the girder depth on centers.

The provision $h/t \leq 2000/\sqrt{F_y}$ is based upon tests* on both homogeneous and hybrid girders with flanges having a specified yield stress of 100 kips per square inch and a web of similar or weaker steel.

1.10.4   Flange Development

If a partial length cover plate is to function as an integral part of a beam or girder at the theoretical cutoff point beyond which it is not needed, it must be developed in an extension beyond this point by enough high-strength bolts, rivets, or welding to support its portion of the flexural stresses (i.e., the stresses which the plate would have received had it been extended the full length of the member). The cover plate force to be developed by the fasteners in the extension is equal to

$$\frac{MQ}{I}$$

where

$M =$ moment at theoretical cutoff

$Q =$ statical moment of cover plate area about neutral axis of coverplated section

$I =$ moment of inertia of coverplated section

When the nature of the loading is such as to produce fatigue, the fasteners must be proportioned in accordance with the provisions of Sect. 1.7.

In the case of welded cover plates, it is further provided that the amount of stress that may be carried by a partial length cover plate, at a distance a' in from its actual end, may not exceed the capacity of the terminal welds deposited along its edges and optionally across its end within this distance a'. If the moment, computed by equating MQ/I to the capacity of the welds in this distance, is less than the value at the theoretical cutoff point, either the size of the welds must be increased or the end of the cover plate must be extended to a point such that the moment on the member at the distance a' from the end of the cover plate is equal to that which the terminal welds will support.

1.10.5   Stiffeners

Unlike columns, which actually are on the verge of collapse as their buckling stage is approached, the panels of a plate girder web, bounded on all sides by the girder flanges or transverse stiffeners, are capable of carrying loads far in excess of their "web buckling" load. Upon reaching the theoretical buckling limit, very

---

* Ref. 30, p. 1412.

slight lateral displacements will have developed in the web. Nevertheless, they are of no structural significance, because other means are still present to assist in resisting further loading.

When transverse stiffeners are properly spaced and strong enough to act as compression struts, membrane stresses, due to shear forces greater than those associated with the theoretical buckling load, form diagonal tension fields. The resulting combination in effect provides a Pratt truss which, without producing yield stress in the steel, furnishes the capacity to resist applied shear forces unaccounted for by the linear buckling theory.

Analytical methods based upon this action have been developed[31,32] and corroborated in an extensive program of tests.[33] These methods form the basis for Formula (1.10-2). Use of tension field action is not counted upon when $0.60F_y/\sqrt{3} \leq F_v \leq 0.40F_y$, nor where $a/h > 3.0$. Pending further investigation, it is not recommended for hybrid girders.

When the computed average shear stress in the web is less than that permitted by Formula (1.10-1), intermediate stiffeners are not required, provided the depth of girders is limited to not more than 260 times the web thickness. Such girders do not depend upon tension field action.

In order to facilitate handling during fabrication and erection, when intermediate stiffeners are required the panel aspect ratio a/h is arbitrarily limited to not more than $[260/(h/t)]^2$, with a maximum spacing of 3 times the girder depth.

When required, the maximum permissible longitudinal spacing of intermediate stiffeners is dependent upon three parameters: a/h, h/t, and f_v. For the convenience of the designer, the relationship of these parameters with one another is presented in Tables 10 and 11, Appendix A, for 36 and 50 kips per square inch yield stress steels. Given the shear diagram produced by the design loads, and a desired depth of girder, it is only necessary to select a web thickness (with due regard for limitations placed on h/t ratios) such that the web shear stress will be equal to or less than the maximum permitted value. With the resulting value for h/t and the computed shear stress, the required aspect ratio a/h can be taken directly from the appropriate table. Comparison of the web and stiffener material required with two or three trial web thicknesses will quickly indicate the most economical combination.

Whenever stiffeners are required, they must satisfy minimum moment of inertia requirements, whether tension field action is counted upon or not. When the maximum allowable shear stress by Formula (1.10-2) is less than $0.35F_y$, the stiffeners must also satisfy a minimum cross-sectional area requirement. In Table 11, the required gross area of intermediate stiffeners, given as a percent of the web area, is shown in italics in the column headed by the aspect ratio and the line nearest to the selected h/t ratio. Stiffeners which will provide this area usually will be little, if any, larger than those generally called for. No stiffener areas are shown in Table 11 when the a/h and h/t ratios are small enough to permit a shear stress larger than $0.35F_y$, for which cases the allowable stress is governed by Formula (1.10-1). No stiffener areas are shown in Table 10, in which all allowable shear stresses, whether greater or less than $0.35F_y$, are governed by Formula (1.10-1), i.e., tension field action is not counted upon.

To provide adequate lateral support for the web, all stiffeners are required to have a moment of inertia at least equal to $(h/50)^4$. In many cases, however, this provision will be overshadowed by the gross area requirement. The amount of stiffener area necessary to develop the tension field, which is dependent upon the ratios a/h and h/t, is given by Formula (1.10-3). Larger gross areas are required for one-sided stiffeners than for pairs of stiffeners, because of the eccentric nature of their loading.

The amount of shear to be transferred between web and stiffeners is not affected by the eccentricity of loading, and generally is so small that it can be taken care of by the minimum amount of welding or riveting that might be desired. The specified formula

$$f_{vs} = h \sqrt{\left(\frac{F_y}{340}\right)^3}$$

affords a conservative estimate of required shear transfer under any condition of stress permitted by Formula (1.10-2). The shear transfer between web and stiffener due to tension field action and that due to a concentrated load or reaction in line with the stiffener are not additive. The stiffener need only be connected for the larger of the two shears.

1.10.6 Reduction in Flange Stress

In regions of maximum bending moment, a portion of a thin web may deflect enough laterally on the compression side of the neutral axis that it does not provide the full bending resistance assumed in proportioning the girder on the basis of its moment of inertia. The compression stress which the web would have resisted is therefore shifted to the compression flange. But because the relative bending strength of this flange is so much greater than that of the laterally displaced portion of the web, the resulting increase in flange stress is at most only a few percent. By reducing the allowable design stress in the compression flange from F_b to F'_b, as provided in Formula (1.10-5), sufficient bending capacity is provided in the flange to compensate for any loss of bending strength in the web due to its lateral displacement.

To compensate for the slight loss of bending resistance when portions of the web of a hybrid flexural member are strained beyond their yield stress limit, Formula (1.10-6)[30] provides for a reduced allowable flange bending stress applicable to both flanges. The extent of the reduction is dependent upon the ratios of web area to a flange area and of web yield stress to flange yield stress.

In order to avoid a more complicated formula, the area and grade of steel in both flanges are required to be the same. Since any reductions in bending strength due to buckling of the web on the compression side of the neutral axis is considerably less in the case of a hybrid girder than for a homogeneous member having the same cross section, it is not required that Formula (1.10-5) apply when the stress permitted by Formula (1.10-6) is less than that given for the former.

1.10.7 Combined Shear and Tension Stress

Unless a flexural member is designed on the basis of tension field action, no stress reduction is required due to the interaction of concurrent bending and shear stress.

It has been shown[34] that plate girder webs subject to tension field action can be proportioned on the basis of:

1. The allowable bending stress, F_b, when the concurrent shear stress, f_v, is not greater than 0.60 of the allowable shear stress, F_v, or
2. The allowable shear stress, F_v, when the concurrent bending stress, f_b, is not greater than 0.75 of the allowable bending stress, F_b.

Beyond these limits a linear interaction formula is provided in the Specification by Formula (1.10-7).

However, because the webs of homogeneous girders of A514 steel loaded to their full capacity in bending develop more waviness than less heavily stressed

girder webs of weaker grades of steel, use of tension field action is limited in the case of A514 steel webs to regions where the concurrent bending stress is no more than $0.75F_b$.

1.10.10   Web Crippling

1.10.10.1   Webs of beams and girders not protected by bearing stiffeners could fail by crippling at points of high stress concentration resulting from the application of concentrated loads or reactions. To guard against this, the stress at the toe of the flange fillet, assumed to be distributed longitudinally a distance no greater than the length of the bearing plus 1 or 2 times the k-distance of the flange, depending upon the location of the load, is limited by Formula (1.10-8) or (1.10-9) to $0.75F_y$.

1.10.10.2   As a safeguard against instability of relatively thin plate girder webs, a further limitation has been placed on the amount of load which can be applied directly to the girder flange between stiffeners. Concentrated loads light enough to meet the provisions of Sect. 1.10.10.1 are treated as if distributed by means of shear over the full panel length within which they occur (or the depth of girder if this is less than the panel length). Taken together with such other distributed loading as may be applied directly to the flange, the total load divided by the web thickness should not exceed the stress permitted by Formula (1.10-10) or (1.10-11). If the flange is prevented from rotation about its longitudinal axis by its contact with a rigid slab, Formula (1.10-10) will govern; otherwise, the more conservative Formula (1.10-11) is applicable.

These formulas are derived[35] from a consideration of the elastic buckling strength of the web plate subject to edge loading. The loading is resisted in part by column action and in part by a plate intermittently stiffened in the direction of applied loading.

The formulas are likely to be overconservative in the case of riveted girders, since they ignore any bending capacity the flange angles may have in spanning between adjacent stiffeners to support the loads.

1.10.11   Rotational Restraint at Points of Support

Slender beams and girders resting on top of columns and stayed laterally only in the plane of their top flanges may become unstable due to the flexibility of the column. Unless lateral support is provided for the bottom flange, either by bracing or continuity at the beam-to-column connection, lateral displacement at the top of the column, accompanied by rotation of the beam about its longitudinal axis, may lead to collapse of the framing.

SECTION 1.11   COMPOSITE CONSTRUCTION

1.11.1   Definition

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.

For composite beams with formed steel deck, studies[36,37] have demonstrated that the total slab thickness, including ribs, can be used in determining effective slab width.

1.11.2 Design Assumptions

Unless temporary shores are used, beams encased in concrete and interconnected only by means of natural bond must be proportioned to support all of the dead load, unassisted by the concrete, plus the superimposed live load in composite action, without exceeding the allowable bending stress for steel provided in Sect. 1.5.1.

Because the completely encased steel section is restrained from both local and lateral buckling, an allowable stress of $0.66F_y$, rather than $0.60F_y$, can be applied when the analysis is based on the properties of the transformed section. The alternate provision to be used in designs where a fully encased beam is proportioned, on the basis of the steel beam alone, to resist all loads at a stress not greater than $0.76F_y$, reflects a common engineering practice where it is desired to eliminate the calculation of composite section properties.

It is accepted practice[37] that when shear connectors are used to obtain composite action, this action may be assumed, within certain limits, in proportioning the beam for the moments created by the sum of live and dead loads, even for unshored construction. This liberalization is based upon an ultimate strength concept, although the provisions for proportioning of the member are based upon the elastic section modulus of the transformed cross section.

The flexural capacity of composite steel-concrete beams designed for complete composite action is the same for either lightweight or normal weight concrete, given the same area of concrete slab and concrete strength, but with the number of shear connectors appropriate to the type of concrete. The same concrete design stress level can be used for both types of concrete.

In order that the maximum bending stress in the steel beam, under service loading, will be well below the level of initial yielding, regardless of the ratio of live-load moment to dead-load moment, the section modulus of the composite cross section, referred to the bottom of the beam, for unshored construction, is limited in calculations to $(1.35 + 0.35M_L/M_D)$ times the section modulus of the bare beam.*

On the other hand, the requirement that flexural stress in the concrete slab, due to composite action, be computed on the basis of the transformed section modulus, referred to top of concrete, and limited to the generally accepted working stress limit, is necessary in order to avoid excessively conservative slab-to-beam proportions.

Research at Lehigh University** has shown that, for a given beam and concrete slab, the increase in bending strength intermediate between no composite action and full composite action is dependent upon the shear resistance developed between the steel and concrete, i.e., the number of shear connectors provided between these limits. Usually, it is not necessary, and occasionally it may not be feasible, to provide full composite action. Therefore, the Specification recognizes two conditions: full and partial composite action.

For the case where the total shear (V'_h) developed between steel and concrete each side of the point of maximum moment is less than V_h, Formula (1.11-1) can be used to derive an effective section modulus, S_{eff}, having a value less than the section modulus for fully effective composite action, S_{tr}, but more than that of

---

* Ref. 38, Eq. (3).

** Ref. 39, p. 91.

the steel beam alone. In the 1969 Specification, the obviously conservative straight line function of Formula (1.11-1) was adopted pending the results of research. The completed research[36] indicated that a parabolic function using $(V'_h/V_h)^{1/2}$ provided a good fit to the test results.

1.11.4 Shear Connectors

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 required 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 shear 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, either side of the point of maximum moment. The provisions of the Specification are based upon this concept of composite action.

In computing the section modulus 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, $\frac{1}{2}$ of the ultimate tensile strength of the reinforcement.

Studies have defined stud shear connection strength, Q_u, in terms of normal weight and lightweight aggregate concretes, as a function of both concrete modulus of elasticity and concrete strength:[40,41]

$$Q_u = 0.5A_s \sqrt{f'_c E_c}$$

where

A_s = cross-sectional area of stud, square inches
f'_c = concrete compressive strength, kips per square inch
E_c = concrete modulus of elasticity, kips per square inch

Tests[41] have shown that fully composite beams designed using the values in Tables 1.11.4 and/or 1.11.4A, as appropriate, and concrete meeting the requirements of Part 3, Chap. 4, "Concrete Quality", of ACI Standard 318-71, made with ASTM C33 or C330 aggregates, develop their full flexural capacity. For normal weight concrete, compressive strengths greater than 4.0 kips per square inch do not increase the shear capacity of the connectors, as is reflected in Table 1.11.4. For lightweight concrete, compressive strengths greater than 5.0 kips per square inch do not increase the shear capacity of the connectors. The reduction coefficients in Table 1.11.4A are applicable to both stud and channel shear connectors and provide comparable margins of safety.

When partial composite action is counted upon to provide flexural capacity, the restriction on the minimum value of V'_h is to prevent excessive slip, as well as substantial loss in beam stiffness. Studies indicate that Formulas (1.11-1) and (1.11-6) adequately reflect the reduction in strength and beam stiffness, respectively, when fewer connectors than required for full composite action are used.

Where adequate flexural capacity is provided by the steel beam alone, that is, composite action to any degree is not required for flexural strength, but where it is desired to provide interconnection between the steel frame and the concrete slab for other reasons, such as to increase frame stiffness or to take advantage of diaphragm action, the minimum requirement that V'_h be not less than $V_h/4$ does not apply.

The required shear connectors can generally be spaced uniformly between the points of maximum and zero moment.* However, certain loading patterns can produce a condition where closer connector spacing is required over a part of this distance.

For example, consider the case of a uniformly loaded simple beam also required to support two equal concentrated loads, symmetrically disposed about midspan, of such magnitude that the moment at the concentrated loads is only slightly less than the maximum moment at midspan. The number of shear connectors (N_2) required between each end of the beam and the adjacent concentrated load would be only slightly less than the number (N_1) required between each end and midspan.

Formula (1.11-7) is provided to determine the number of connectors, N_2, required between one of the concentrated loads and the nearest point of zero moment. It is based upon the following requirement:

$$\frac{N_2}{N_1} = \frac{S - S_s}{S_{eff} - S_s} = \frac{\left[\dfrac{S}{S_{eff}} \times \dfrac{S_{eff}}{S_s}\right] - 1}{\dfrac{S_{eff}}{S_s} - 1}$$

where

S = section modulus required at the concentrated load at which location moment equals M, inches^3

S_{eff} = section modulus required at M_{max} (equal to S_{tr} for fully composite case), inches^3

S_s = section modulus of steel beam, inches^3

N_1 = number of studs required from M_{max} to zero moment

N_2 = number of studs required from M to zero moment

M = moment at the concentrated load point

M_{max} = maximum moment in the beam

Noting that $S/S_{eff} = M/M_{max}$, and defining β as S_{eff}/S_s, the above equation is equivalent to Formula (1.11-7).

With the issuance of Supplement No. 3 to the 1969 AISC Specification, the requirement for 1-inch cover over the tops of studs was eliminated. Only the concrete surrounding the stud below the head contributes to the strength of the stud in resistance to shear. 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.

Stud welds 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 $2\frac{1}{2}$ times the flange thickness.

1.11.5 Composite Beams and Girders with Formed Steel Deck

The 6 diameter minimum center-to-center spacing of studs in the longitudinal direction is based upon observation of concrete shear failure surfaces in sectioned flat soffit concrete slab composite beams which had been tested to full ultimate strength. The reduction in connection capacity of more closely spaced shear studs within the ribs of formed steel decks is accounted for by the parameter $0.85/\sqrt{N_r}$ in Formula (1.11-8).

---

* Ref. 39, p. 91.

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 per square foot, special precautions and procedures recommended by the stud manufacturer should be followed.

Figure C1.11.5 is a graphic presentation of the terminology used in Sect. 1.11.5.
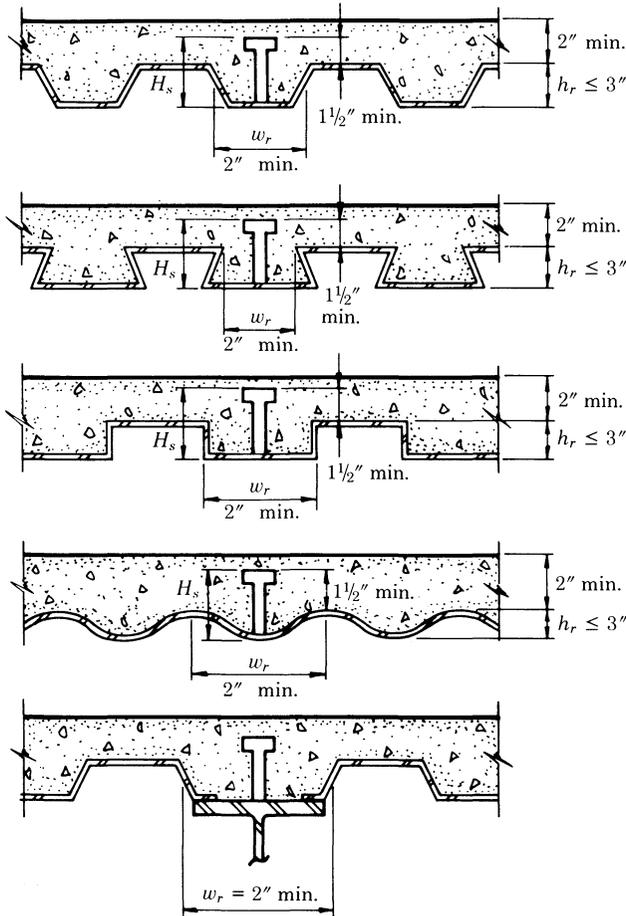
Fig. C1.11.5

The design rules which have been added for composite construction with formed steel deck are based upon a study[36] at Lehigh University of all available test results. The limiting parameters listed in Sect. 1.11.5.1 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: $3/4$-in. diam. x 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

Based upon all tests, the strength of stud connectors in flat soffit composite slab beams, determined in previous test programs,[41] when multiplied by values computed from Formula (1.11-8), reasonably approximates 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. Hence, Formula (1.11-8) provides a reasonable reduction factor to be applied to the allowable design stresses in Tables 1.11.4 and 1.11.4A.

For the case where ribs run parallel to the beam, limited testing[36] 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 (1.11-9), has been suggested in view of lack of test data.

The Lehigh study[36] also indicated that Formula (1.11-1) for effective section modulus and Formula (1.11-6) 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, in effect creating 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.

SECTION 1.13   DEFLECTIONS, VIBRATION, AND PONDING

1.13.1   Deflections

Although deformation, rather than stress, is sometimes the criterion of satisfactory design, there is no single scale by which the limit of tolerable deflection can be defined. Where limitations on flexibility are desirable, they are often dictated by the nature of collateral building components, such as plastered walls and ceilings, rather than by considerations of human comfort and safety. The admissible amount of movement varies with the type of component.

The most satisfactory solution must rest upon the sound judgment of qualified engineers. As a guide, the following rules are suggested:

1. The depth of fully stressed beams and girders in floors should, if practicable, be not less than $(F_y/800)$ times the span. If members of less depth are used, the unit stress in bending should be decreased in the same ratio as the depth is decreased from that recommended above.

2. The depth of fully stressed roof purlins should, if practicable, be not less than $(F_y/1000)$ times the span, except in the case of flat roofs.

1.13.2 Vibration

Where human comfort is the criterion for limiting motion, as in the case of perceptible vibrations, the limit of tolerable amplitude is dependent on both the frequency of the vibration and the damping effect provided by components of the construction. At best, the evaluation of these criteria is highly subjective, although mathematical models[42] do exist which may be useful. When such vibrations are caused by running machinery, they should be isolated by effective damping devices or by the use of independent foundations.

The depth of a steel beam supporting large open floor areas free of partitions or other sources of damping should not be less than $\frac{1}{20}$ of the span, in order to minimize perceptible transient vibration due to pedestrian traffic.

1.13.3 Ponding

As used in the 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.

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 and, from this, the contribution that the deflection each of these members makes to the total ponding deflection can be expressed[43] as

$$\Delta_w = \frac{\alpha_p \Delta_o \left[1 + \frac{\pi}{4} \alpha_s + \frac{\pi}{4} \rho (1 + \alpha_s) \right]}{1 - \frac{\pi}{4} \alpha_p \alpha_s}$$

for the primary member, and

$$\delta_w = \frac{\alpha_s \delta_o \left[1 + \frac{\pi^3}{32} \alpha_p + \frac{\pi^2}{8\rho} (1 + \alpha_p) + 0.185 \alpha_s \alpha_p \right]}{1 - \frac{\pi}{4} \alpha_p \alpha_s}$$

for the secondary member. 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 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.
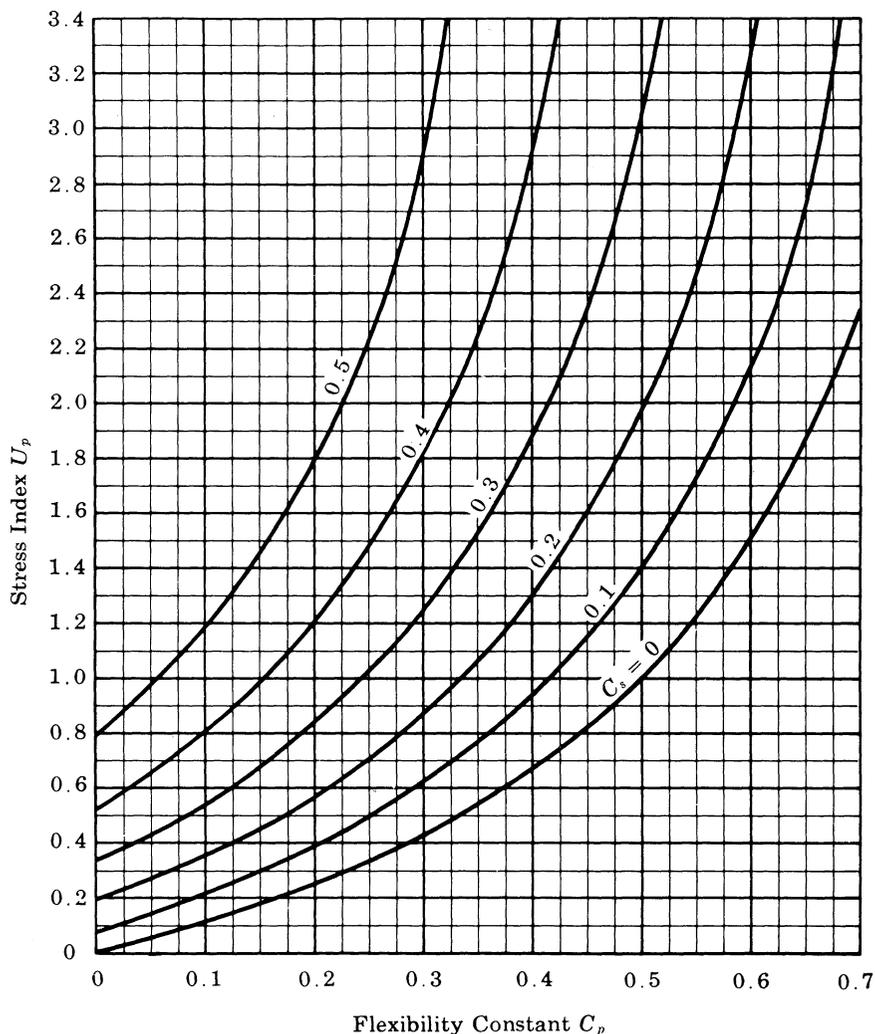
Fig. C1.13.3.1

Noting that elastic deflection is directly proportional to stress, and providing a factor of safety of 1.25 with respect to stress due to ponding, the admissible amount of ponding deflection 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. C1.13.3.1 and C1.13.3.2 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 Specification provision that $C_p + 0.9C_s \leq 0.25$.
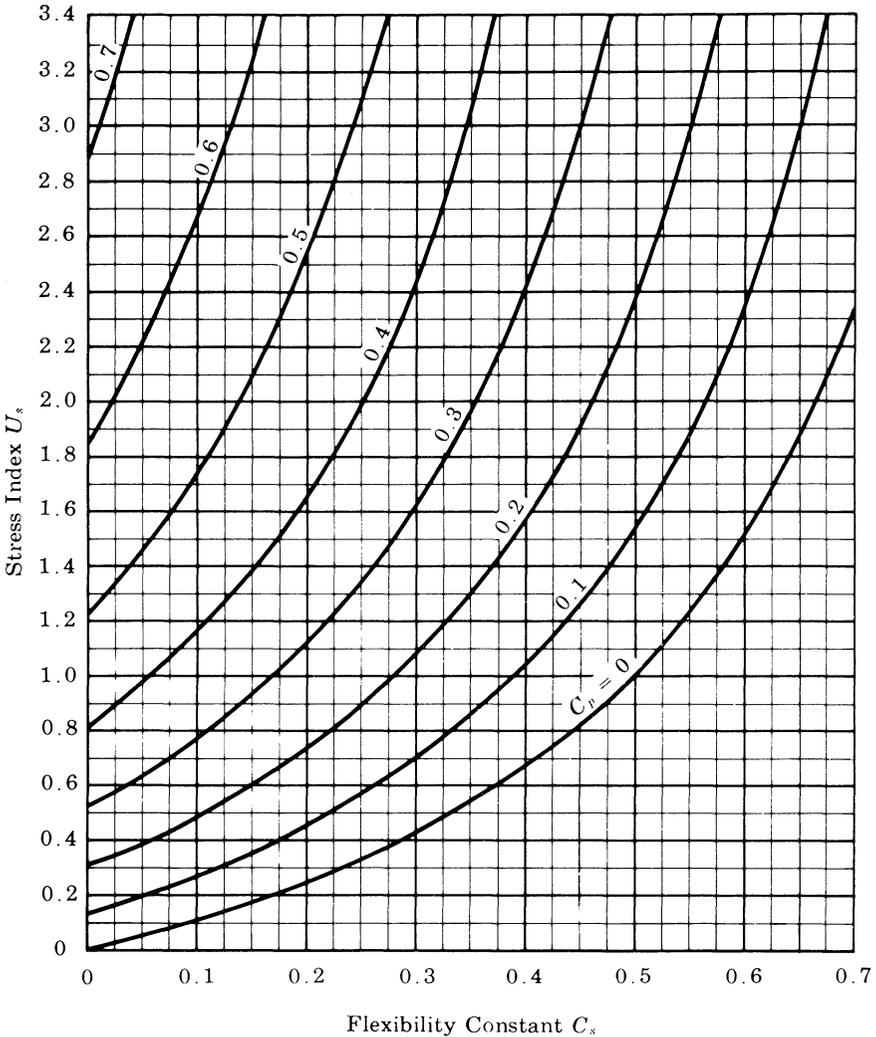
Fig. C1.13.3.2

Given any combination of primary and secondary framing, the stress index is computed as

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_p \qquad \text{for the primary member}$$

$$U_s = \left(\frac{0.8F_y - f_o}{f_o}\right)_s \qquad \text{for the secondary member}$$

where f_o, in each case, is the computed bending stress 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. C1.13.3.1 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, thence, 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 enter Fig. C1.13.3.2. 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 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 to be checked. This can be done using Fig. C1.13.3.1 or C1.13.3.2 with the following computed values:

U_p, the stress index for the supporting beam

U_s, the stress index for the roof deck

C_p, the flexibility constant for the supporting beams

C_s, the 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.

SECTION 1.14 GROSS AND NET AREAS

1.14.2.2 When failure by fracture occurs in the net area (defined as in Sect. 1.14.2.1) of tension members other than flat plates or bars, the failure load divided by the net area is generally less than the coupon tensile strength of the steel, unless

all of the segments comprising the profile are connected so as to provide a uniformly distributed transfer of stress. This is attributable to a concentration of shear stress in the vicinity of the connection, often referred to as *shear lag*. When, for example, an angle is connected by only one leg, an increase in the distance $\bar{x}$, from the centroid of the angle profile to the shear plane of the connection, will result in shear lag, everything else being equal. On the other hand, as the length of the connection, l, is increased, the intensity of shear lag is diminished. The situation can be expressed empirically as

$$C_t \approx 1 - \bar{x}/l$$

Munse and Chesson have shown,* 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 ±10 percent. For any given profile and connected elements, $\bar{x}$ is a fixed geometric property. Length l, however, is dependent upon the number of fasteners 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 used. The values of C_t, given as reduction coefficients in Sect. 1.14.2.2, are reasonably lower bounds for the profile types and connection means described, based upon the use of the above expression.

1.14.2.3 The restriction that the net area shall in no case be considered as comprising more than 85 percent of the gross area is now limited to relatively short fittings, such as splice plates, gusset plates, or beam-to-column fittings.

1.14.5 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 Specification are based upon standards evolved from long experience with forged eyebars. Through extensive destructive testing, they have been found to provide balanced designs when these members are thermally cut instead of forged. The somewhat more conservative rules for pin-connected members of non-uniform 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 kips per square inch, in order to eliminate any possibility of their "dishing" under the higher working stress for which they may be designed.

SECTION 1.15 CONNECTIONS

1.15.3 Placement of Rivets, Bolts, and Welds

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 upon the static strength of such members. Tests[46] have shown that similar practice is warranted in the case of welded members in statically loaded structures. However, the fatigue life of single angles, loaded in tension or compression, has been shown to be very short.[47]

---

* Refs. 44, 45 (pp. 114–123), and 7 (pp. 146–159).
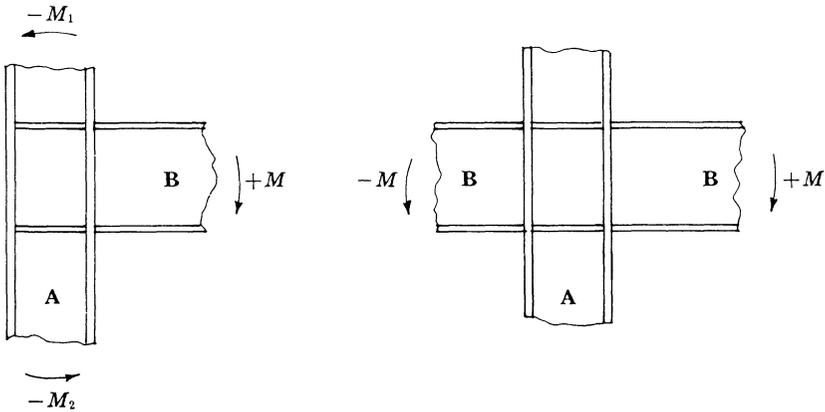
Fig. C1.15.5

1.15.5 Restrained Members

Whether or not transverse stiffeners are required on the web of a member opposite the flanges of members rigidly connected to its flanges, as in Fig. C1.15.5.5, depends upon the proportions of these members. Formula (1.15-1), giving the required area of stiffeners when stiffeners are needed, is based on tests[48] supporting the concept that, in the absence of transverse stiffeners, the web and flange thickness of member A should be such that these elements will not yield inelastically under concentrated forces delivered by member B.

Formula (1.15-2) limits the slenderness ratio of an unstiffened web of the supporting member, in order to avoid possibility of its buckling. Formula (1.15-3) limits the bending stress in the flange of the supporting member.

When Formula (1.15-2) and/or Formula (1.15-3) indicate the need for stiffeners, the required area of stiffeners is not given. However, minimum stiffener dimensions are given in Sect. 1.15.5 and their width-to-thickness ratio must satisfy Sect. 1.9.1.

In previous editions of the Specification, Formulas (1.15-1), (1.15-2), and (1.15-3) were based on the maximum force that could be delivered by the supporting member flanges. This was conservative for connection details that limited the maximum force to less than $A_f F_y$, and for members that were oversized from the standpoint of stresses, in order to satisfy a deflection limitation or to increase the stiffness of a structure. In this Specification, the provisions are still conservative in that they require stiffeners to be proportioned on the basis of a force greater than the actual force; that is, the formulas are based on the actual force delivered multiplied by a load factor. In calculations, the actual force times the load factor need not exceed the area of the flange or connection plate delivering the force times the yield strength of the material.

1.15.6 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 as a friction-type 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 required.

1.15.10   Rivets and Bolts in Combination with Welds

The sharing of stress between rivets and A307 bolts in a single group of fasteners is not recommended. High-strength bolts used in bearing-type connections should not be required to share shear stress with welds. High-strength bolts used in friction-type connections, however, because of the rigidity of the connection, may be proportioned to function in conjunction with welds in resisting the transfer of stress across faying surfaces. Because the welds, if installed prior to final tightening of the bolts, might interfere with the development of the high contact pressure between faying surfaces that is counted upon in friction-type connections, it is advisable that the welds be made after the bolts are tightened. At the location of the fasteners, the heat of welding the connected parts will not alter the mechanical properties of the fasteners.

In making alterations to existing structures, it is assumed that whatever slip is likely to occur in riveted connections or high-strength bolted bearing-type 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.

SECTION 1.16   RIVETS AND BOLTS

1.16.1   High-Strength Bolts

Provision for the limited use of A449 bolts, in lieu of A325 bolts, is predicated on the fact that the provisions of ASTM A449 concerning quality control are less stringent that those contained in ASTM A325. These bolts differ from A325 bolts only as to reduced size of head and increased length of threading.

1.16.3   Long Grips

Provisions requiring a decrease in calculated stress for rivets having long grips (by arbitrarily increasing the required number an amount in proportion to the grip length) are not required for high-strength bolts. Tests[49] 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.

1.16.4   Minimum Spacing
1.16.5   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 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, even beyond the range of interest:

$$\frac{F_{pcr}}{F_u} = \frac{l_e}{d}$$

---

* Ref. 7, pp. 109–112, 137.

where

F_{pcr} = critical bearing stress

F_u = tensile strength of the connected material

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), inches

d = diameter of a fastener, inches

The above equation, modified by a safety factor of 2, is the basis for Formulas (1.16-1) and (1.16-2) in the Specification.

Along a line of transmitted force, the required spacing center-to-center of standard holes is found from Formula (1.16-1). For oversized and slotted holes, this spacing is increased by an increment C_1, given in Table 1.16.4.2, providing the same clear distance between holes as for standard holes.

The required edge distance in the direction of stress is found from Formula (1.16-2) as the distance from the center of a standard hole to the edge of a connected part. For oversized and slotted holes, this distance is increased by an increment C_2, given in Table 1.16.5.4, providing the same clear distance from the edge of the hole as for a standard hole.

The provisions of Sect. 1.16.4 are concerned with l_e as hole spacing, whereas Sect. 1.16.5 is concerned with l_e as edge distance in the direction of stress, and Sect. 1.5.1.5.3 establishes a maximum allowable bearing stress. Spacing and/or edge distance may be increased to provide for a required bearing stress, or bearing force may be reduced to satisfy a spacing and/or edge distance limitation. Thus, the Specification provides for adjustment of spacing and/or edge distance in the direction of stress in terms of calculated bearing stress or vice versa, rather than providing a single criterion.

It has long been known[50] that the critical bearing stress of a single fastener connection is more dependent upon a given edge distance than multi-fastener connections. For this reason, previous editions of the AISC Specification have required longer edge distances (in the direction of stress) for connections having no more than two fasteners in the line of transmitted force than required for those having more than two.

Recent tests performed at Lehigh University have shown that when the capacity of a connection designed on the basis of the present higher allowable stresses is dependent upon bearing, rather than tension on the effective net area or shear in the fasteners, the critical bearing stress is significantly affected by reduction of the end distance, even with three fasteners in line.

Although in beam end connections the reaction force is essentially parallel to the free end of the beam web, effects of fastener eccentricity may cause force vectors of uncertain magnitude and direction.[68] The requirement for minimum edge distances from the center of the fastener holes to the free end of the web, determined by use of Formula (1.16-3), has been introduced to assure that tearout through the end will not occur, while eliminating the necessity of an analysis which accounts for the effects of fastener eccentricity. The available test data indicate that the uncoped end connections performed slightly better than predicted, considering the added end distance restriction. However, pending further research on the behavior of end connections, the same restrictions have been applied to both coped and uncoped beams.

The distinction between Formulas (1.16-2) and (1.16-3) is illustrated in Fig. C1.16.5. Formula (1.16-3) is applicable only to the ends of high-strength-bolted beam connections, whereas Formula (1.16-2) applies to all high-strength-bolted connections where a force is transmitted toward a free edge.

Edge distance by
Formula (1.16-3)

Edge distance by
Formula (1.16-2)
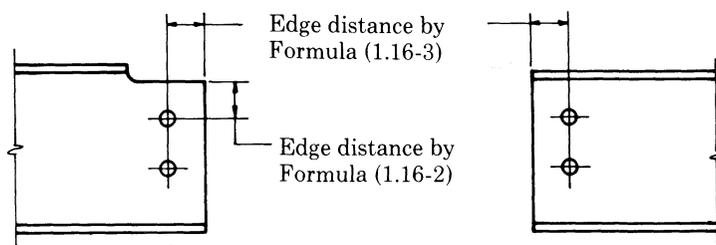
Fig. C1.16.5

SECTION 1.17 WELDS

With the adoption of the 1969 edition of the AISC Specification, a number of welding provisions were introduced to cover welding processes, techniques, and workmanship requirements relative to which the AWS Code (then entitled *Code for Welding in Building Construction,* D1.1-69) was silent at that time. Subsequently, provisions similar to those adopted by AISC have been incorporated in the AWS *Structural Welding Code,* D1.1-77. In this edition of the Specification, the requirements of the AWS Code have been adopted by reference, with four exceptions, and most requirements governing welding workmanship have been deleted. For convenience of the designer, provisions for allowable design stresses and proportioning of welds have been retained, even though the AISC and AWS provisions are consistent.

The provisions of the AWS *Structural Welding Code* to which exception is taken in the AISC Specification are as follows:

1. Section 2.3.2.4 of the AWS Code and Sect. 1.14.6.2 of the AISC Specification both define the effective throat of fillet welds as the shortest distance from the root to the face of the diagrammatic weld. However, for fillet welds made by the submerged arc process, Sect. 1.14.6.2 additionally recognizes the deep penetration that is provided by this automatic process at the root of the weld beyond the limits of the diagrammatic weld.

2. Section 2.5 of the AWS Code prohibits the use of partial-penetration welds subject to cyclic tension normal to the longitudinal axis of the weld, whereas the AISC Specification, Appendix B, recognizes partial-penetration welds subject to fatigue loading, but only at the same severely limited stress ranges of Category F that are appropriate to fillet welds.

3. Section 8.13.1.2 of the AWS Code provides criteria for the flatness of girder webs, which are arbitrary and based upon a concern for possible cyclic secondary stresses resulting from breathing action of thin girder webs subject to fatigue loading. The AISC Specification does not include such criteria, because lateral deflection or out-of-flatness of webs of girders subject to static loading is of no structural significance. If architectural appearance of exposed girders is of importance, then tolerances based upon specific consideration of architectural requirements, rather than tolerances based upon unrelated consideration of fatigue effects, should be provided in the project specification.

4. Section 9 of the AWS Welding Code is applicable to bridges, which are outside the scope of the AISC Specification. Therefore, no comparable provisions are included in the AISC Specification.

As in earlier editions, the Specification accepts, without further procedure qualification, numerous weld and joint details executed in accordance with the provisions of the AWS Code. Other welding procedures may be used, provided they are qualified to the satisfaction of the designer and the building code authority and are executed in accordance with the provisions of the AWS Code.

SECTION 1.18 BUILT-UP MEMBERS

Requirements for detailing of built-up members, which cannot be stated in terms of calculated stress, are based upon judgment, tempered by experience.

The longitudinal spacing of fasteners connecting components of built-up compression members must be so limited that buckling of segments between adjacent fasteners would not occur at less load than that required to develop the ultimate strength of the member as a whole. However, maximum fastener spacing less than that necessary to prevent local buckling may be needed to ensure a close fit-up over the entire faying surface of components designed to be in contact with one another.

Provisions based on this latter consideration, like those giving maximum spacing of stitch fasteners for separated components of built-up tension members, are of little structural significance. Hence, some latitude is warranted in relating them to the given dimensions of a particular member.

The provisions governing the proportioning of perforated cover plates are based upon extensive experimental research.[51]

SECTION 1.19 CAMBER

The cambering of flexural members, to eliminate the appearance of sagging or to match the elevation of adjacent building components when the member is loaded, 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 of camber produced by any of these methods can be controlled to a remarkable degree, it must be realized that some tolerance, to cover workmanship error and permanent change due to handling, is inevitable.

SECTION 1.20 EXPANSION

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 having masonry wall enclosures than where the walls consist of 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.

SECTION 1.22   ANCHOR BOLTS

Shear at the base of a column resisted by bearing of the column base details against the anchor bolts is seldom, if ever, critical.   Even considering the lowest conceivable slip coefficient, the vertical load on a column is generally more than sufficient to result in the transfer of any likely amount of shear from column base to foundation by frictional resistance, so that the anchor bolts usually experience only tensile stress.   Generally, the largest tensile force for which anchor bolts should be designed is that produced by bending moment at the column base, at times augmented by uplift caused by the overturning tendency of a building under lateral load.

Hence, the use of oversized holes required to accommodate the tolerance in setting anchor bolts cast in concrete, permitted in Sect. 1.23.4.1, is in no way detrimental to the integrity of the supported structure.

SECTION 1.23   FABRICATION

1.23.1   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 1100°F for A514 steel, as contrasted with 1200°F for other steels.

1.23.4   Riveted and Bolted Construction—Holes

A new section has been added in this edition of the Specification, providing rules for the use of oversized and slotted holes paralleling the provisions which have been in the RCRBSJ specification[52] since 1972, extended to include A307 bolts, which are outside the scope of the high-strength bolt specifications.

1.23.5   Riveted and High-Strength-Bolted Construction—Assembling

Even when used in bearing-type shear connections, A325 and A490 bolts are required to be tightened to 0.7 of their tensile strength.   This may be done either by the turn-of-nut method,[52] by a calibrated wrench, or by use of direct tension indicators.   Since fewer fasteners and stiffer connected parts are involved than is generally the case with A307 bolts, the greater clamping force is recommended in order to ensure solid seating of the connected parts.   However, because the performance of bolts in bearing is not dependent upon an assured minimum level of high pretension, thorough inspection requirements to assure full and complete compliance with pretightening criteria is not warranted.   This is especially true regarding the arbitration inspection requirements of Sect. 6(d) of the RCRBSJ specification.[52]   Visual evidence of solid seating of the connected parts, and of wrench impacting to assure that the nut has been tightened sufficiently to prevent it from loosening and falling off, is adequate.

SECTION 1.24   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.   Where such leakage is not eliminated, the presence or absence of a shop coat is of minor influence.[53]

The Specification does not define the type of paint to be used when a shop coat is required. Conditions of exposure and individual preferences with regard to finish paint are factors which have a bearing on the selection of the proper primer. Hence, a single formulation would not suffice.*

SECTION 1.25   ERECTION

1.25.4   Fit of Column Compression Joints

Tests at the University of California in Berkeley[55] 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 $1/16$-inch were not shimmed; gaps of $1/4$-inch 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 $1/4$-inch.

The criteria for fit of column compression joints are equally applicable to joints at column splices and joints between columns and base plates.

SECTION 2.1   SCOPE

The Specification recognizes three categories of profiles, classified according to the ability to resist local buckling of elements of the cross section when subject to compressive stress. These categories are: (1) non-compact, (2) compact, and (3) plastic design. The elements of *non-compact* sections (Sect. 1.9) will not buckle locally when subject to elastic limit strains. Elements of *compact* sections (Sect. 1.5.1.4.1) are proportioned so that the cross section may be strained in bending to the degree necessary to achieve full plastification of the cross section; however, the reserve for inelastic strains is adequate only to achieve modest re-distribution of moments. The elements of *plastic design* sections (Sect. 2.7) are proportioned so that they will not only achieve full plastification of the cross section, but will remain stable while being bent through an appreciable angle at a constant plastic moment up to the point where strain hardening is initiated. Thus, plastic design cross sections are capable of providing the hinge rotations that are counted upon in the plastic method of analysis.

The superior bending strength of compact sections is recognized in Part 1 of the Specification by increasing the allowable bending stress to $0.66F_y$ and by permitting 10% redistribution of moment. By the same token, the logical load factor for plastically designed beams is given by the equation

$$F = \frac{F_y}{0.66F_y} \times \text{(shape factor)}$$

For such shapes listed in the AISC *Steel Construction Manual,* the variation of shape factor is from 1.10 to 1.23, with a mode of 1.12. Then, the corresponding load factor must vary from 1.67 to 1.86, with a mode of 1.70. Such a load factor is consistent and in better balance with that inherent in the allowable working stresses for tension members and deep plate girders.

Research[56] on the ultimate strength of heavily loaded columns subjected to concurrent bending moments has provided data which justifies a load factor, for such members, that is the same as that provided for members subject to bending

---

* For a comprehensive treatment of the subject, see Ref. 54.

only, namely 1.7. Consistent with the $\frac{1}{3}$ increase in allowable stress permitted in Part 1 of the Specification, the load factor to be used in designing for gravity loading combined with wind or seismic loading is 1.3.

Based on continuing research at Lehigh University on multistory framing,[57,58] application of the Specification provisions includes the complete design of braced and unbraced planar frames in high-rise buildings. Systematic procedures for application of plastic design in proportioning the members of such frames have been developed[59,60] and are available in the current literature.

SECTION 2.2 STRUCTURAL STEEL

Research testing has demonstrated the suitability of all of the steels listed in this section for use in plastic design.*

SECTION 2.3 BASIS FOR MAXIMUM STRENGTH DETERMINATION

While resistance to wind and seismic loading can be provided in moderate height buildings by means of concrete or masonry shear walls, which also provide for overall frame stability at factored gravity loading, taller building frames must provide this resistance acting alone. This can be achieved in one of two ways: either by a system of bracing or by a moment-resisting frame.

For one- and two-story unbraced frames with Type 1 construction throughout, where the column axial loads are generally modest, the frame instability effect is small and $P\Delta$ effects** may be safely ignored. However, where such frames are designed with a mixture of rigid connections and simple or semi-rigid connections (Type 2 and Type 3 construction), it may be necessary to consider the frame instability effect ($P\Delta$). In this case, stability is dependent upon a reduced number of rigid connections and the effect of frame drift may be a significant consideration in the design.

2.3.1 Stability of Braced Frames

The limitation on axial force of $0.85P_y$ was inserted as a simple means of compensation for three possible effects:†

1. Loss of stiffness due to residual stress
2. Effect of secondary ($P\Delta$) moments on the vertical bracing system
3. Lateral torsional buckling effect

SECTION 2.4 COLUMNS

Formulas (2.4-2) and (2.4-3)‡ will be recognized as similar in type to Formulas (1.6-1a) and (1.6-1b) in Part 1, except that they are written in terms of factored loads and moments, instead of allowable stresses at service loading. As in the case of Formulas (1.6-1a) and (1.6-1b), P_{cr} is computed on the basis of l/r_x or l/r_y, whichever is larger, for any given unbraced length.¶

---

* Ref. 61 and Sect. 5.1 of Ref. 62.

** See Commentary Sect. 1.8 for discussion of $P\Delta$ effects.

† Ref. 62, Chapter 10.

‡ Ref. 57, Eqs. (4.6) and (4.7).

¶ Ibid., p. 4.24.

A column is considered to be fully braced if the slenderness ratio l/r_y between the braced points is less than or equal to that specified in Sect. 2.9. When the unbraced length ratio of a member bent about its strong axis exceeds the limit specified in Sect. 2.9, the rotation capacity of the member may be impaired, due to the combined influence of lateral and torsional deformation, to such an extent that plastic hinge action within the member cannot be counted upon. However, if the computed value of M is small enough so that the limitations of Formulas (2.4-2) and (2.4-3) are met, the member will be strong enough to function at a joint where the required hinge action is provided in another member entering the joint. An assumed reduction in moment-resisting capacity is provided by using the value M_m, computed from Formula (2.4-4), in Formula (2.4-2).

Formula (2.4-4) was developed empirically* on the basis of test observations and provides an estimate of the critical lateral buckling moment, in the absence of axial load, for the case where $M_1/M_2 = -1.0$ (single curvature bending). For other values of M_1/M_2, adjustment is provided by using the appropriate C_m value as defined in Sect. 1.6.1.

Formula (2.4-4) is to be used only in connection with Formula (2.4-2).

Space frames containing plastically designed planar rigid frames are assumed to be supported against sidesway normal to these frames. Depending upon other conditions of restraint, the basis for determination of proper values for P_{cr} and P_e and M_m, for a plastically designed column oriented to resist bending about its strong axis, is outlined in Table C2.4.1. In each case l is the distance between points of lateral support corresponding to r_x or r_y, as applicable. When K is indicated, its value is governed by the provisions of Sect. 1.8.3 of the Specification.

TABLE C2.4.1

	Braced Planar Frames	One- and Two-Story Unbraced Planar Frames
P_{cr}	Use larger ratio, $\dfrac{l}{r_y}$ or $\dfrac{l}{r_x}$	[1]Use larger ratio, $\dfrac{l}{r_y}$ or $\dfrac{Kl}{r_x}$
P_e	Use l/r_x	[1]Use Kl/r_x
M_m	Use l/r_y	Use l/r_y

[1] Webs of columns assumed to be in plane of frame.

SECTION 2.5 SHEAR

Using the von Mises criterion, the average stress at which an unreinforced web would be fully yielded in pure shear can be expressed as $F_y/\sqrt{3}$. It has been observed** that the plastic bending strength of an I-shape beam is not appreciably reduced until shear yielding occurs over the full effective depth, which may be taken as the distance between the centroids of its flanges (approximately 0.95 times its actual depth). Thus,

$$V_u = \frac{F_y}{\sqrt{3}} \times 0.95\, dt = 0.55 F_y dt$$

* Ref. 57, p. 4.26.
** Ref. 62, Sect. 6.1.

Shear stresses are generally high within the boundaries of a rigid connection of two or more members whose webs lie in a common plane. Assuming the moment $+M$, in Fig. C2.5.1, expressed in kip-feet, to be resisted by a force couple acting at the centroid of the beam flanges, the shear, in kips, produced in beam-to-column connection web $abcd$ can be computed as

$$V = \frac{+12M}{0.95d_b}$$

when $V = V_u = 0.55F_y d_c t$

$$\text{Req'd } t = \frac{12M}{0.95d_b \times 0.55F_y d_c} = \frac{23M}{A_{bc}F_y}$$

where A_{bc} is the planar area $abcd$ and F_y is expressed in kips per square inch. If the thickness of the web panel is less than that given by this formula, the deficiency may be compensated for by a pair of diagonal stiffeners or by a reinforcing plate in contact with the web panel and welded around its boundary to the column flanges and horizontal stiffeners.
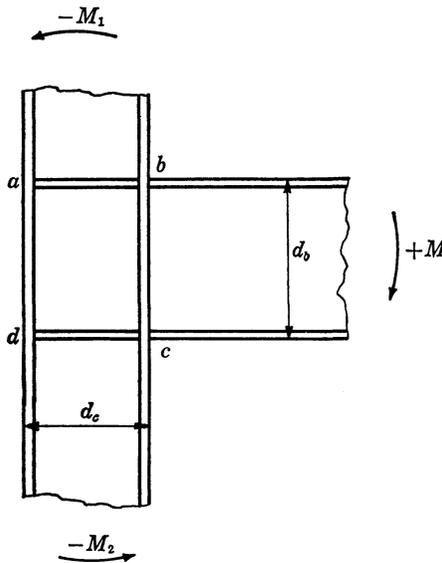
Fig. C2.5.1

SECTION 2.6   WEB CRIPPLING

Usually stiffeners are needed, as at ab and dc in Fig. C2.5.1, in line with the flanges of a beam rigidly connected to the flange of a second member so located that their webs lie in the same plane, in order to prevent crippling of the web of the latter opposite the compression flange of the former. A stiffener may also be required opposite the tension flange, in order to protect the weld joining the two flanges; otherwise the stress in the weld might be too great in the region of the beam web, due to lack of bending stiffness in the flange to which the beam is connected. Since their design is based upon equating the plastic resisting capacity

of the supporting member to the plastic moment delivered by the supported member, Formulas (1.15-1), (1.15-2), and (1.15-3) are equally applicable to allowable stress design and plastic design.

When stiffeners are required, as an alternative to the usual pair of horizontal plates, vertical plates parallel to but separated from the web, as shown in Fig. C2.6.1, may prove advantageous.
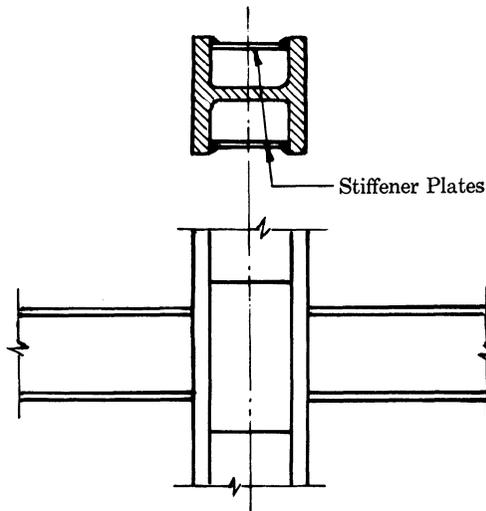
Fig. C2.6.1

SECTION 2.7   MINIMUM THICKNESS (WIDTH-THICKNESS RATIOS)

Research* has shown that the limiting flange and web width-thickness ratios, below which ample plastic hinge rotations could be relied upon without reduction in the M_p-value due to local buckling, are not exactly proportional to $1/\sqrt{F_y}$, although the discrepancy using such a relationship, within the range of yield stress presently permitted by the Specification, is not large.   Expressions including other pertinent factors are complex and involve use of mechanical properties that have not been clearly defined.   Tabular values for limiting flange width-thickness ratios are given in the Specification for the approved grades of steel.

No change in basic philosophy was involved in extending the earlier expresssion for limiting web depth-thickness ratio to stronger steels.   Formulas (2.7-1a) and (2.7-1b) were derived, with minor adjustments for better correlation with observed test results, by multiplying Formula (25) of the 1963 Specification by the factor $\sqrt{36/F_y}$, in order to cover the accepted range in yield point stress. Formula (2.7-1a) is identical to Formula (1.5-4) in Part 1 of the 1969 Specification, except that it is written in terms of factored loads instead of allowable stresses at service loading.   Formula (1.5-4) was liberalized in 1974 and redesignated as Formula (1.5-4a).   However, this liberalization was not extended to plastic design sections, which require greater rotational capacity than compact sections.

---

* Ref. 62, Sect. 6.2.

SECTION 2.8   CONNECTIONS

Connections located outside of regions where hinges would have formed at ultimate load can be treated in the same manner that similar connections in frames designed in accordance with the provisions of Part 1 would be treated. Since the moments and forces to be resisted will be those corresponding to the factored loading, the permissible stresses to be used in proportioning parts of the connections can be taken as 1.7 times those given in Sects 1.5 and 1.6.

The same procedure is valid in proportioning connections located in the region of a plastic hinge. Connections required to resist moments and forces due to wind and earthquake loads combined with gravity loading factored to 1.3, and proportioned on the basis of limiting stresses equal to 1.7 times those given in Sects. 1.5 and 1.6, provide a balance between frame strength and connection strength, provided they are adequate to resist gravity loading alone, factored to 1.7.

The width-thickness ratio and unbraced length of all parts of the connection that would be subject to compression stresses in the region of a hinge must meet the requirements given in Part 2, and sheared edges and punched holes must not be used in portions of the connection subject to tension.

When a haunched connection is proportioned elastically for the moments that would exist within its length, the continuous frame can be analyzed as a mechanism having a hinge at the small end of the haunch, rather than at the intersection point between connected members,* with some attendant economy.

Tests[63] have shown that splices assembled with high-strength bolts are capable of developing the M_p-value of the gross cross section of the connected part. It has also been demonstrated[64] that beam-to-column connections involving use of welded or mechanically fastened fittings, instead of full-penetration groove welds matching the full member cross section, not only are capable of developing the M_p-value of the member, but that the resulting hinge rotation can be reversed several times without failure.

SECTION 2.9   LATERAL BRACING

Portions of members that would be required to rotate inelastically as a plastic hinge, in reducing a continuous frame to a mechanism at ultimate load, need more bracing than similar parts of a continuous frame designed in accordance with the elastic theory. Not only must they reach yield point at a load factor of 1.7, they must also strain inelastically to provide the necessary hinge rotation. This is not true at the last hinge to form, since the factored load is assumed to have been reached when this hinge starts to rotate. When bending takes place about the strong axis, any W-shape member tends to buckle out of the plane of bending. It is for this reason that lateral bracing is needed. The same tendency exists with highly stressed members in elastically designed frames, and in portions of plastically designed frames outside of the hinge areas, but here the problem is less severe since hinge rotation is not involved.

The Specification provisions governing unbraced length are based upon research on members with moment gradients.**

In keeping with similar usage of the parameter M/M_p in Sect. 1.6 of the Specification, the sign convention adopted in Formulas (2.9-1a) and (2.9-1b) is that generally found to be more convenient in frame analysis, namely that clockwise moments about a fixed point are positive and counterclockwise moments are negative.

---

* Ref. 62, Chapter 8.
** Ibid., Sect. 6.3, and Ref. 65, p. 390.

APPENDIX B—FATIGUE*

APPENDIX C—SLENDER COMPRESSION ELEMENTS**

APPENDIX D—TAPERED MEMBERS

The provisions contained in Appendix D cover 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 D, see the appropriate portions of Part 1 of the Specification and Commentary.

SECTION D2 ALLOWABLE STRESSES—COMPRESSION

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.[66] This factor, which is used to determine the value of S in Formulas (D2-1) and (D2-2), 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. CD1.5.1. From Formulas (D2-1) and (D2-2), the critical load P_{cr} can be expressed as $\pi^2 EI_o/(K_\gamma'l)^2$. The value of K_γ can be obtained by interpolation, using the appropriate chart (Figs. CD1.5.2 to CD1.5.17) 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. Such an approach is well documented.

SECTION D3 ALLOWABLE STRESSES—BENDING

The development of the allowable 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 with that of the smaller end of the tapered beam.[66] This has led to the modified length factors h_s and h_w in Formulas (D3-1) and (D3-2).

Formulas (D3-1) and (D3-2) 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 Section D3 usually apply; however, it is to be noted that they apply

---

* See Commentary Sect. 1.7 for discussion of Fatigue Provisions of Appendix B.
** See Commentary Sect. 1.9 for discussion of Appendix C provisions for Slender Compression Elements.

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 for B is unity. The value of B should also be taken as unity when computing the value of $F_{b\gamma}$ to be used in Formula (D4-1a), since the effect of moment gradient is provided for by the factor C_m. The background material is given in WRC Bulletin No. 192.[67]

Thus, it is to be noted that in these charts the values of K_γ represent the combined effects of end restraints and tapering. For the case $\gamma = 0$, K_γ becomes K, which can also be determined from the alignment chart for effective length of columns in continuous frames (Fig. C1.8.2). For cases when the restraining beams are also tapered, the procedure used in WRC Bulletin No. 173[66] can be followed, or appropriate estimation of K_γ can be made based on these charts.

$$G_T = \frac{b_T I_0}{\ell I_T}$$

$$G_B = \frac{b_B I_0}{\ell I_B}$$

$$G_T = \frac{I_0}{\ell} \sum \frac{b_T}{I_T}$$

$$G_B = \frac{I_0}{\ell} \sum \frac{b_B}{I_B}$$
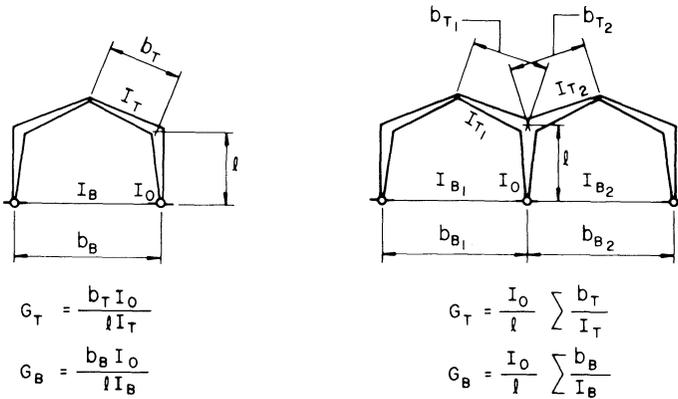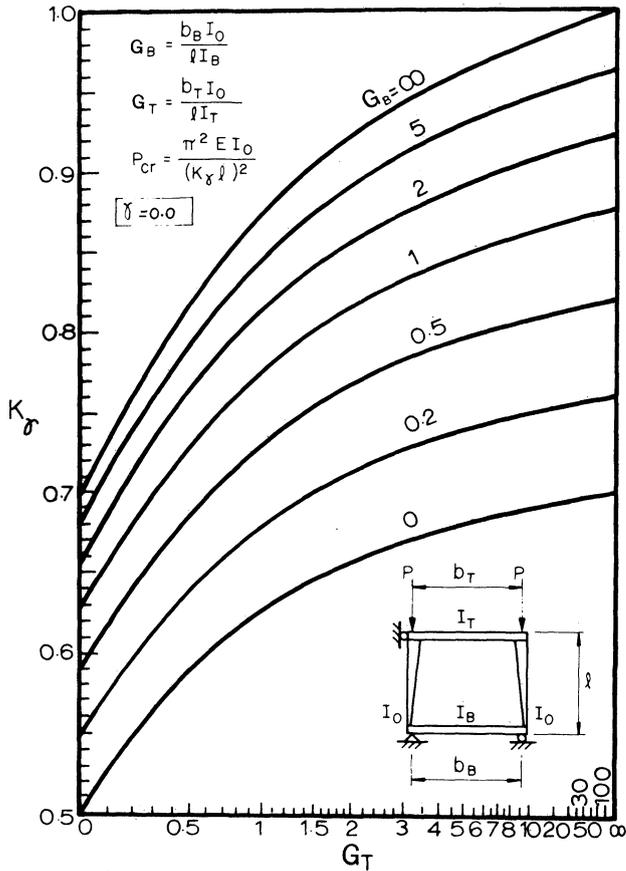
Fig. CD1.5.1

Fig. CD1.5.2. Effective length factors for tapered columns: sidesway prevented ($\gamma = 0$)
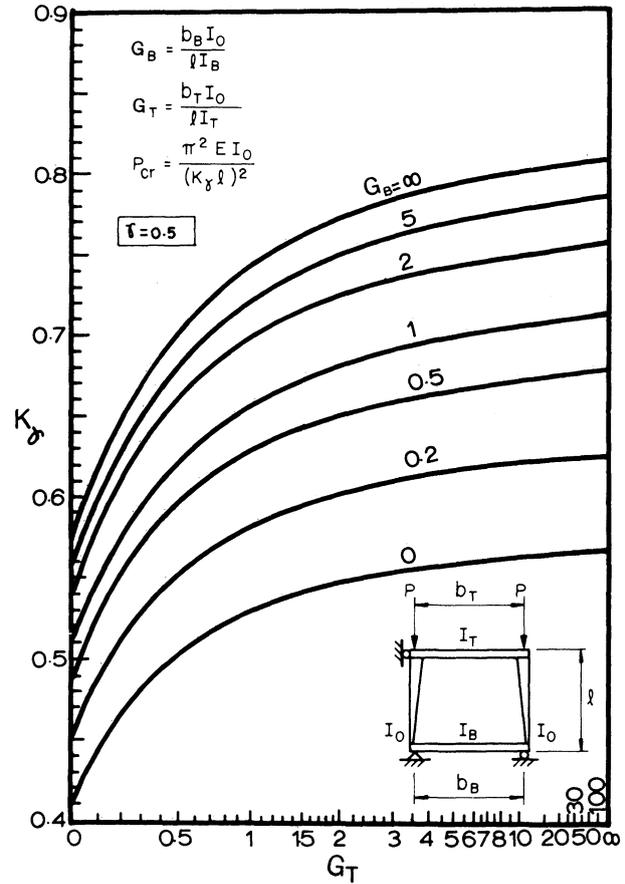
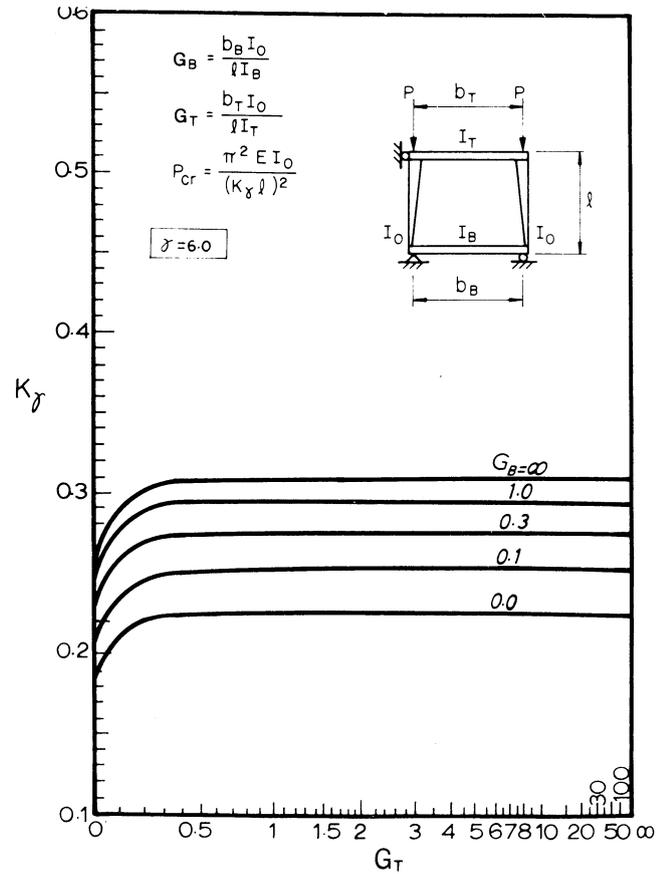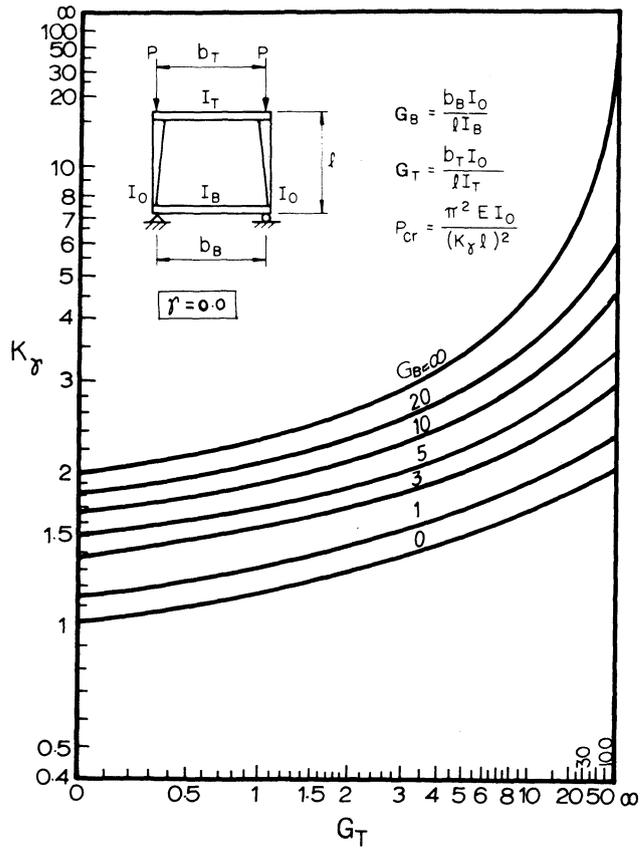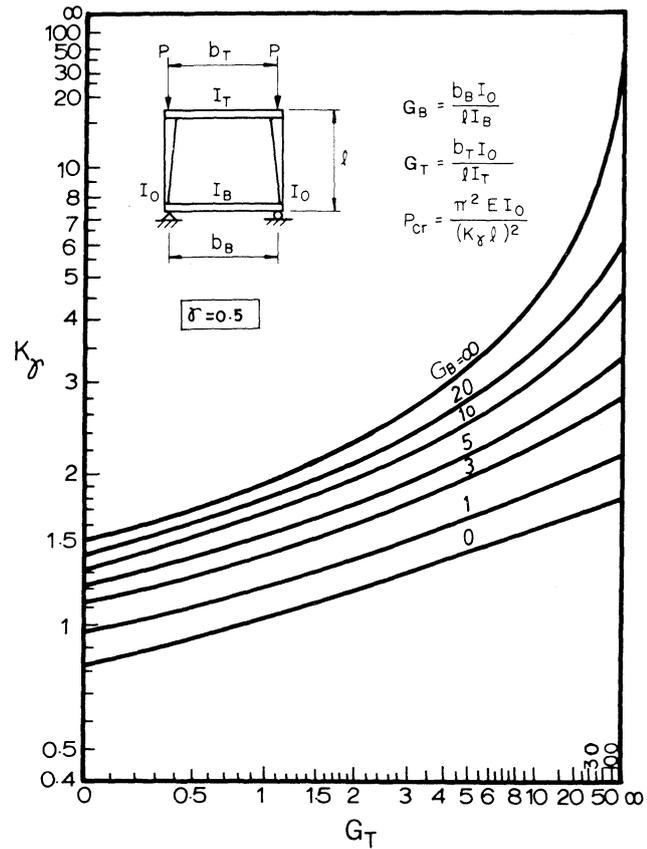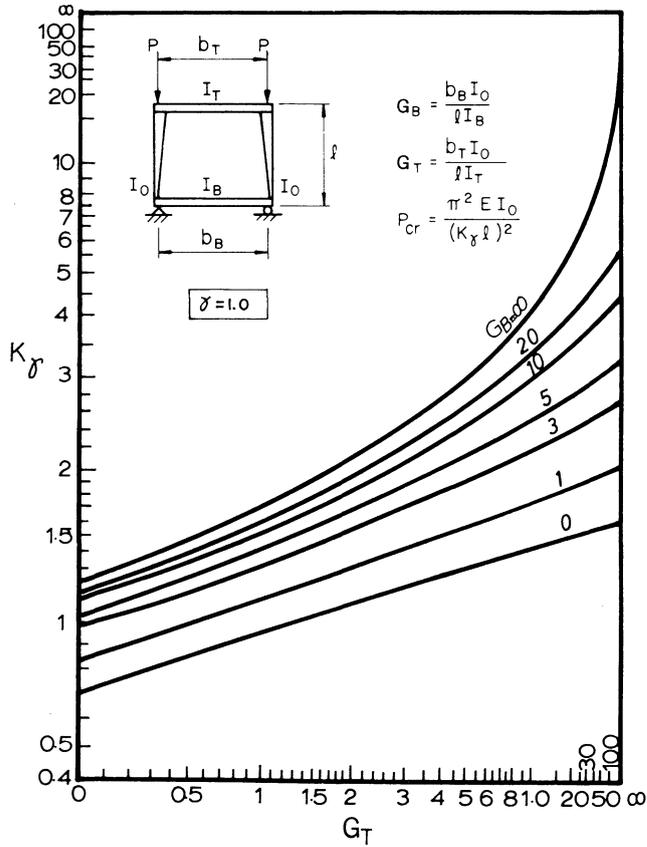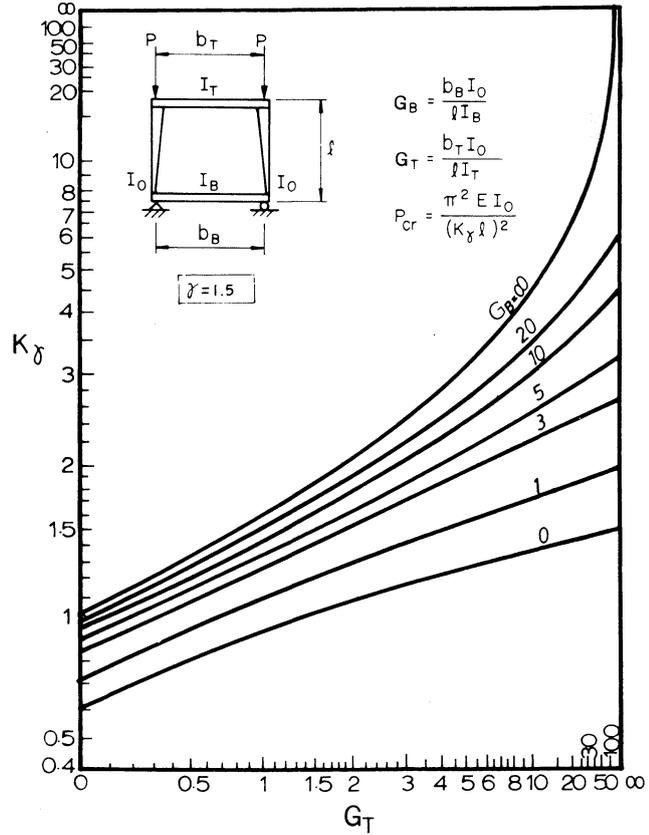**Fig. CD1.5.3.** Effective length factors for tapered columns: sidesway prevented ($\gamma = 0.5$)
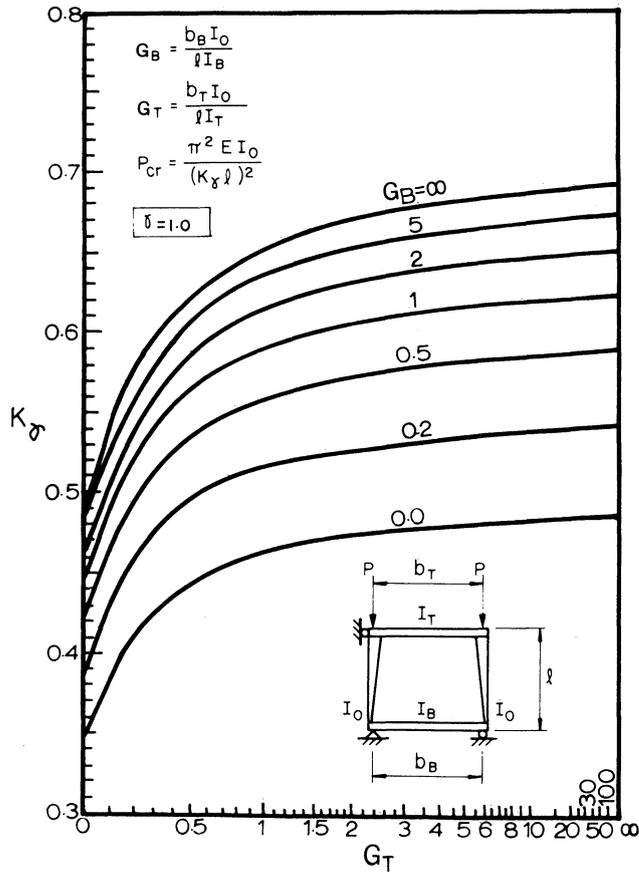
Fig. CD1.5.4. **Effective length factors for tapered columns: sidesway prevented ($\gamma = 1.0$)**
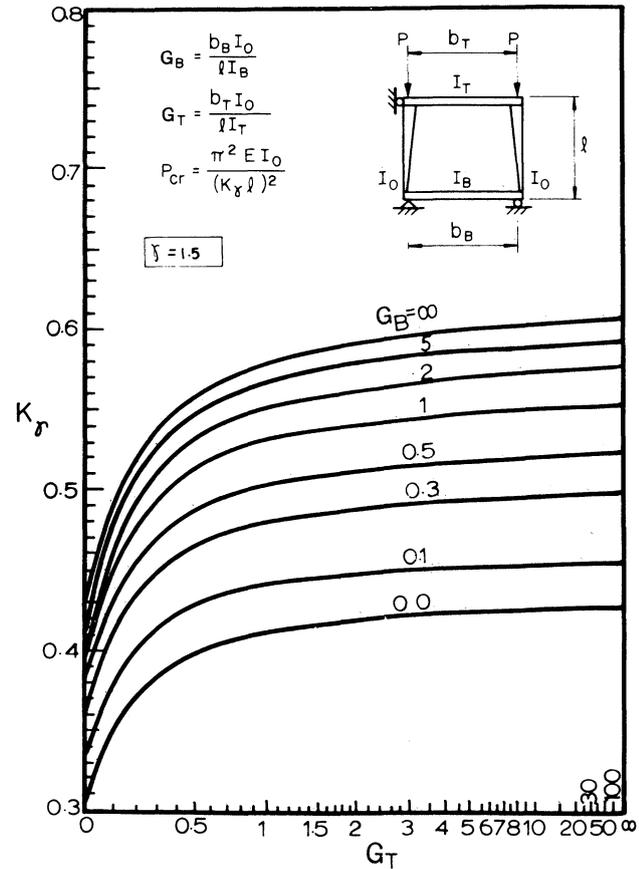
Fig. CD1.5.5. **Effective length factors for tapered columns: sidesway prevented ($\gamma = 1.5$)**
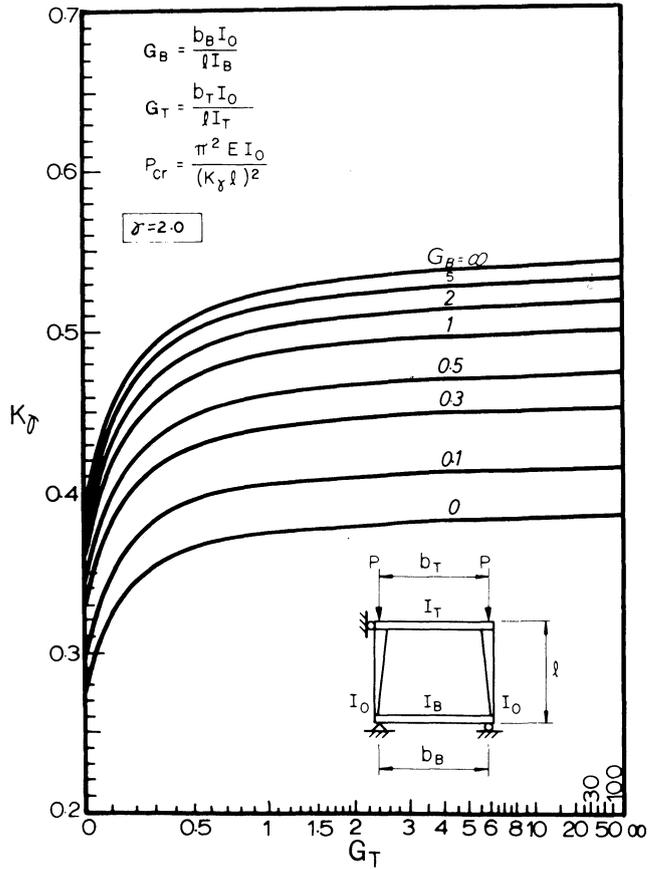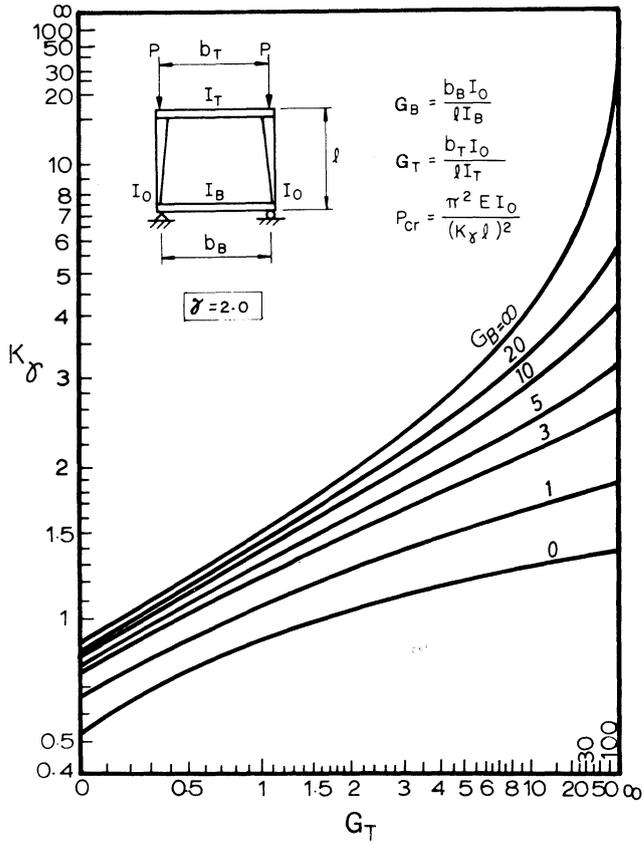
Fig. CD1.5.6. Effective length factors for tapered columns: sidesway prevented ($\gamma = 2.0$)
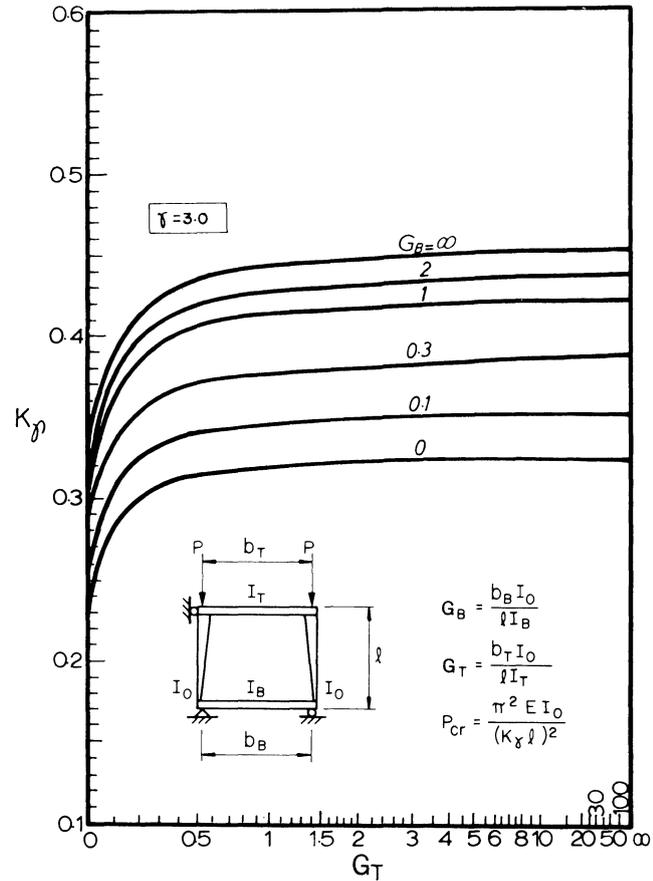
Fig. CD1.5.7. Effective length factors for tapered columns: sidesway prevented ($\gamma = 3.0$)
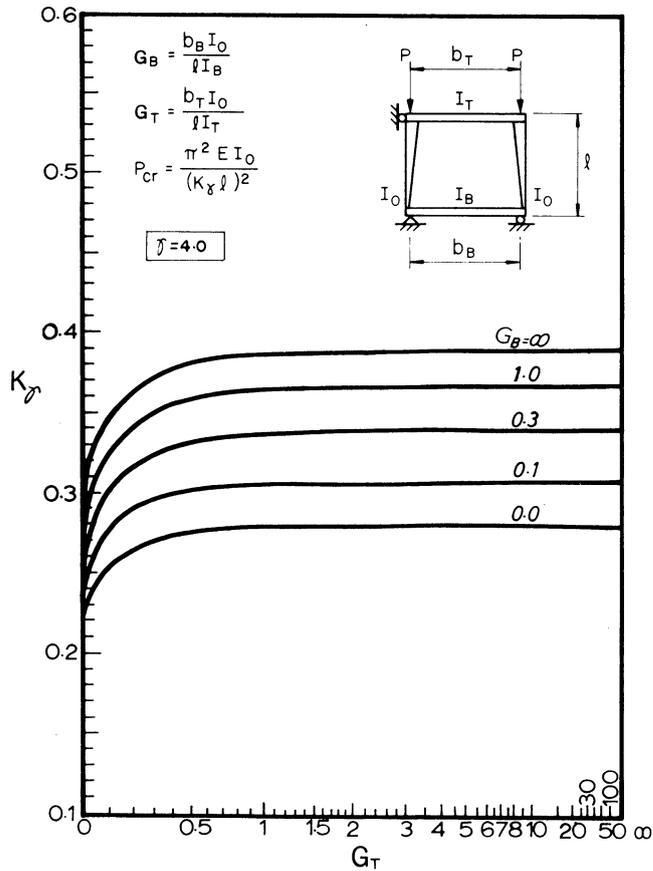
Fig. CD1.5.8. Effective length factors for tapered columns: sidesway prevented ($\gamma = 4.0$)

Fig. CD1.5.9. Effective length factors for tapered columns: sidesway prevented ($\gamma = 6.0$)

Fig. CD1.5.10. **Effective length factors for tapered columns: sidesway permitted ($\gamma = 0$)**

Fig. CD1.5.11. **Effective length factors for tapered columns: sidesway permitted ($\gamma = 0.5$)**

Fig. CD1.5.12. Effective length factors for tapered columns: sidesway permitted ($\gamma = 1.0$)

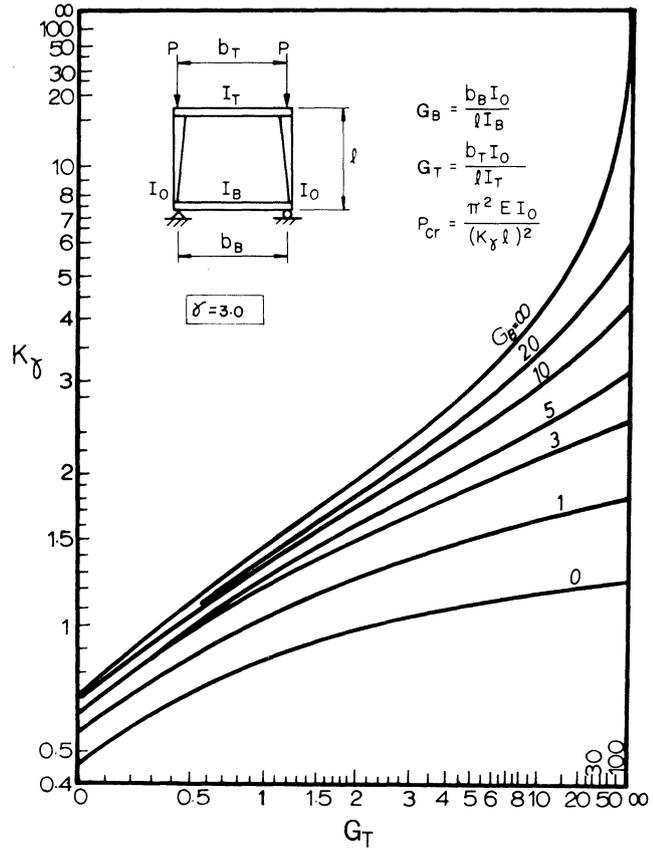**Fig. CD1.5.13.** Effective length factors for tapered columns: sidesway permitted ($\gamma = 1.5$)
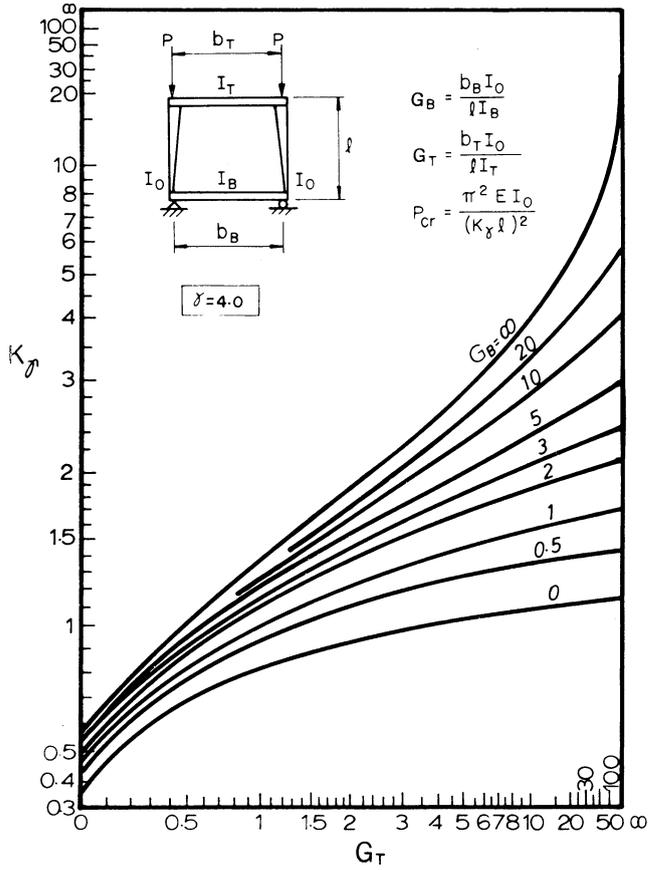
Fig. CD1.5.14. Effective length factors for tapered columns: sidesway permitted ($\gamma = 2.0$)
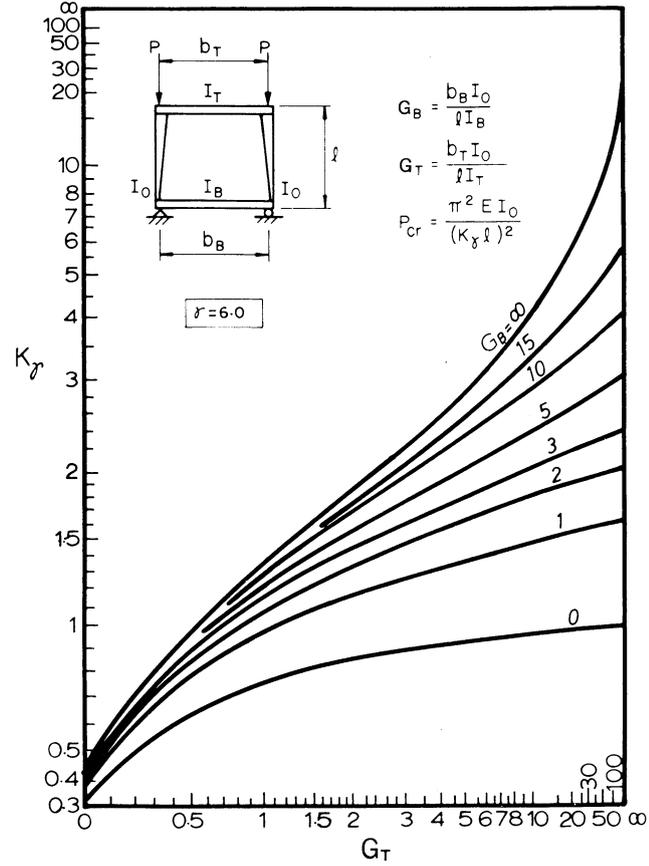

Fig. CD1.5.15. Effective length factors for tapered columns: sidesway permitted ($\gamma = 3.0$)

Fig. CD1.5.16. Effective length factors for tapered columns: sidesway permitted ($\gamma = 4.0$)

Fig. CD1.5.17. Effective length factors for tapered columns: sidesway permitted ($\gamma = 6.0$)

COMMENTARY APPENDIX

LIST OF REFERENCES

1. *Disque, R. O.* Wind Connections with Simple Framing *AISC Engineering Journal, Vol. 1, No. 3, July, 1964.*
2. *American Institute of Steel Construction* Commentary on Highly Restrained Welded Connections *AISC Engineering Journal, Vol. 10, No. 3, 3rd Quarter, 1973.*
3. *Rolfe, S. T.* Fracture and Fatigue Control in Steel Structures *AISC Engineering Journal, Vol. 14, No. 1, 1st Quarter, 1977.*
4. *Johnston, B. G.* Pin-Connected Plate Links *1939 ASCE Transactions.*
5. *Johnston, Bruce G. (Ed.)* Structural Stability Research Council Guide to Stability Design Criteria for Metal Structures *Third Edition, John Wiley & Sons, 1976.*
6. *Sherman, D. R.* Tentative Criteria for Structural Applications of Steel Tubing and Pipe *American Iron and Steel Institute, Washington, D.C., August, 1976.*
7. *Fisher, John W. and Struik, John H. A.* Guide to Design Criteria for Bolted and Riveted Joints *John Wiley & Sons, New York, 1974.*
8. *Krishnamurthy, N.* A Fresh Look at Bolted End-Plate Behavior and Design *AISC Engineering Journal, Vol. 15, No. 2, 2nd Quarter, 1978.*
9. *Higgins, T. R. and Preece, F. R.* Proposed Working Stresses for Fillet Welds in Building Construction *Welding Journal Research Supplement, Oct., 1968.*
10. Building Code Requirements for Reinforced Concrete *ACI 318-71, American Concrete Institute, Detroit, Mich., 1971.*
11. *Austin, W. J.* Strength and Design of Metal Beam-Columns *ASCE Journal of the Structural Division, April, 1961.*
12. *Fisher, J. W.; Frank, K. H.; Hirt, M. A.; and McNamee, B. M.* Effect of Weldments on the Fatigue Strength of Beams *National Cooperative Highway Research Program, Report 102, 1970.*
13. *Fisher, J. W.; Albrecht, P. A.; Yen, B. T.; Klingerman, D. J; and McNamee, B. M.* Fatigue Strength of Steel Beams With Welded Stiffeners and Attachments *National Cooperative Highway Research Program, Report 147, 1974.*
14. *Galambos, T. V.* Influence of Partial Base Fixity on Frame Stability *ASCE Journal of the Structural Division, May, 1960.*
15. *Bleich, F.* Buckling Strength of Metal Structures *McGraw-Hill Book Co., New York, 1952.*
16. *Yura, J. A.* The Effective Length of Columns in Unbraced Frames *AISC Engineering Journal, Vol. 8, No. 2, April, 1971.*
17. *Disque, R. O.* Inelastic K-Factor in Design *AISC Engineering Journal, 2nd Quarter, 1973.*
18. *Ozer, E.; Okten, O. S.; Morino, S.; Daniels, J. H.; and Lu, L. W.* Frame Stability and Design of Columns in Unbraced Multistory Steel Frames *Fritz Engineering Laboratory Report No. 375.2, Lehigh University, November, 1974.*
19. *Cheong-Siat Moy, F.; Ozer, e.; and Lu, L. W.* Strength of Steel Frames Under Gravity Loads *Journal of the Structural Division, ASCE, Vol. 103, No. ST6, June, 1977.*
20. *Springfield, J. and Adams, P. F.* Aspects of Column Design in Tall Steel Buildings *Journal of the Structural Division, ASCE, Vol. 98, No. ST5, May, 1972.*
21. *Liapunov, S.* Ultimate Load Studies of Plane Multi-Story Steel Rigid Frames *Journal of the Structural Division, ASCE, Vol. 100, No. ST8, Proc. Paper 10750, August, 1974.*
22. *Daniels, J. H. and Lu, L. W.* Plastic Subassemblage Analysis for Unbraced Frames *Journal of the Structural Division, ASCE, Vol. 98, No. ST8, August, 1972.*
23. *LeMessurier, W. J.* A Practical Method of Second Order Analysis/Part 1—Pin Jointed Frames *AISC Engineering Journal, Vol. 13, No. 4, 4th Quarter, 1976.*
24. *LeMessurier, W. J.* A Practical Method of Second Order Analysis/Part 2—Rigid Frames *AISC Engineering Journal, Vol. 14, No. 2, 2nd Quarter, 1977.*
25. *Winter, G.* Lateral Bracing of Columns and Beams *Journal of the Structural Division, ASCE, Vol. 84, No. ST2, March, 1958.*
26. *Chajes, A. and Winter, G.* Torsional Flexural Buckling of Thin-Walled Members *Journal of the Structural Division, ASCE, Vol. 91, No. ST4, August, 1965.*
27. *v. Karman, T.; Sechler, E. E.; and Donnell, L. H.* The Strength of Thin Plates in Compression *1932 ASME Transactions, Vol. 54, APM-54-5.*
28. *Winter, G.* Strength of Steel Compression Flanges. *1947 ASCE Transactions.*
29. *Lilly, S. B. and Carpenter, S. T.* Effective Moment of Inertia of a Riveted Plate Girder *1940 ASCE Transactions.*

30. Design of Hybrid Steel Beams *Report of Subcommittee 1 of the Joint ASCE-AASHO-Committee on Flexural Members, Journal of the Structural Division, ASCE, Vol. 94, No. ST6, June, 1968.*
31. *Basler, K. and Thürlimann, B.* Strength of Plate Girders in Bending *Journal of the Structural Division, ASCE, Vol. 89, No. ST4, August, 1963.*
32. *Basler, K.* Strength of Plate Girders in Shear *Journal of the Structural Division, ASCE, Vol. 87, No. ST7, October, 1961.*
33. *Basler, K.; Yen, B. T.; Mueller, J. A.; and Thürlimann, B.* Web Buckling Tests on Welded Plate Girders *Welding Research Council Bulletin No. 64, September, 1960.*
34. *Basler, K.* Strength of Plate Girders Under Combined Bending and Shear *Journal of the Structural Division, ASCE, Vol. 87, No. ST7, October, 1961.*
35. *Basler, K.* New Provisions for Plate Girder Design *Appendix C, 1961 Proceedings AISC National Engineering Conference.*
36. *Grant, J. A.; Fisher, J. W.; and Slutter, R. O.* Composite Beams with Formed Steel Deck *AISC Engineering Journal, Vol. 14, No. 1, 1st Quarter, 1977.*
37. *Fisher, J. W.* Design of Composite Beams with Formed Metal Deck *AISC Engineering Journal, Vol. 7, No. 3, July, 1970.*
38. Progress Report of the Joint ASCE-ACI Committee on Composite Construction *ASCE Journal of the Structural Division, December, 1960.*
39. *Slutter, R. G. and Driscoll, G. C.* Flexural Strength of Steel-Concrete Composite Beams *Journal of the Structural Division, ASCE, Vol. 91, No. ST2, April, 1965.*
40. *McGarraugh, J. B. and Baldwin, J. W.* Lightweight Concrete-on-Steel Composite Beams *AISC Engineering Journal, Vol. 8, No. 3, July, 1971.*
41. *Ollgaard, J. G.; Slutter, R. G; and Fisher, J. W.* Shear Strength of Stud Shear Connections in Lightweight and Normal Weight Concrete *AISC Engineering Journal, Vol. 8, No. 2, April, 1971.*
42. *Murray, T. M.* Design to Prevent Floor Vibration *AISC Engineering Journal, Vol. 12, No. 3, 3rd Quarter, 1975.*
43. *Marino, F. J.* Ponding of Two-Way Roof Systems *AISC Engineering Journal, Vol. 3, No. 3, July, 1966.*
44. *Munse, W. H. and Chesson, E. Jr.* Riveted and Bolted Joints: Net Section Design *Journal of the Structural Division, ASCE, Vol. 89, No. ST1, February, 1963.*
45. *Gaylord, E. H. and Gaylord, C. N.* Design of Steel Structures *Second Edition, McGraw-Hill Book Co., New York, 1972.*
46. *Gibson, G. T. and Wake, B. T.* An Investigation of Welded Connections for Angle Tension Members *The Welding Journal, American Welding Society, January, 1942.*
47. *Koppel, K. and Seeger, T.* Dauerversuche Mit Einschnittigen Hv-Verbindunger Aus ST37 *Der Stahlbau, Vol. 33, No. 8, August, 1964, pp. 225–245, and Vol. 33, No. 11, November, 1964. pp. 335–346.*
48. *Graham, J. D.; Sherbourne, A. N.; Knabbaz, R. N; and Jensen, C. D.* Welded Interior Beam-to-Column Connections *American Institute of Steel Construction, 1959.*
49. *Bendigo, R. A.; Hansen, R. M.; and Rumpf, J. L.* Long Bolted Joints *Journal of the Structural Division, ASCE, Vol. 89, No. ST6, December, 1963.*
50. *Jones, J.* Static Tests on Riveted Joints *Civil Engineering, May, 1940.*
51. *Stang, A. H. and Jaffe, B. S.* Perforated Cover Plates for Steel Columns *Research Paper RP1861, National Bureau of Standards, 1948.*
52. Specification for Structural Joints Using ASTM A325 or A490 Bolts *Research Council on Riveted and Bolted Structural Joints, 1978.*
53. *Bigos, J., Smith, G. W., Ball, E. F. and Foehl, P. J.* Shop Paint and Painting Practice *1954 Proceedings, AISC National Engineering Conference.*
54. Steel Structures Painting Manual, Volume 2, Systems and Specifications *Steel Structures Painting Council, Pittsburgh, Pa.*
55. *Popov, E. P. and Stephen, R. M.* Capacity of Columns with Splice Imperfections *AISC Engineering Journal, Vol. 14, No. 1, 1st Quarter, 1977.*
56. *Van Kuren, R. C. and Galambos, T. V.* Beam-Column Experiments *Journal of the Structural Division, ASCE, Vol. 90, No. ST2, April, 1964.*
57. *Driscoll, G. C. et al.* Plastic Design of Multi-Story Frames—Lecture Notes *Fritz Engineering Laboratory Report No. 273.20, Lehigh University, August, 1965.*
58. *Driscoll, G. C.* Lehigh Conference on Plastic Design of Multi-Story Frames—A Summary *AISC Engineering Journal, April, 1966.*
59. Plastic Design of Braced Multi-Story Steel Frames *American Iron and Steel Institute, 1968.*

60. *Lu, Le-Wu*   Design of Braced Multi-Story Frames by the Plastic Method   *AISC Engineering Journal, January, 1967.*
61. *Adams, P. F.; Lay, M. G.; and Galambos, T. V.*   Experiments on High Strength Steel Members   *Welding Research Council Bulletin No. 110, November, 1965.*
62. Plastic Design in Steel   *ASCE Manual of Engineering Practice No. 41, Second Edition, 1971.*
63. *Douty, R. T. and McGuire, W.*   High Strength Bolted Moment Connections   *Journal of the Structural Division, ASCE, Vol. 91, No. ST2, April, 1965.*
64. *Popov, E. P. and Pinkney, R. B.*   Behavior of Steel Building Connections Subjected to Inelastic Strain Reversals   *Bulletin Nos. 13 and 14, American Iron and Steel Institute, November, 1968.*
65. *Lay, M. G. and Galambos, T. V.*   Inelastic Beams Under Moment Gradient   *Journal of the Structural Division, ASCE, Vol. 93, No. ST1, February, 1967.*
66. *Lee, G. C.; Morrell, M. L.; and Ketter, R. L.*   Design of Tapered Members   *WRC Bulletin No. 173, June 1972.*
67. *Morrell, M. L. and Lee, G. C.*   Allowable Stress for Web-Tapered Beams with Lateral Restraints   *WRC Bulletin No. 192, Feb. 1974.*
68. *Birkemoe, P. C. and Gilmor, M. I.*   Behavior of Bearing-Critical Double Angle Beam Connections   *AISC Engineering Journal, Fourth Quarter, 1978.*