SPECIFICATION FOR THE DESIGN, FABRICATION & ERECTION OF STRUCTURAL STEEL FOR BUILDINGS

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Preface

Research completed since the last revision of the AISC *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings* in 1963, together with the publication of new ASTM specifications covering grades of structural steel often affording improved economy, account for the additions and most of the changes in this revision of the AISC Specification.

Among the new provisions attributable to recent research are those covering the use of hybrid flexural members, that is, beams and girders having higher strength steel in the flanges than in the web. Also included are: a more rational set of working stresses for fillet welds that consider the mechanical properties of both weld and base metal and cover a much wider strength range than heretofore; the extension of existing working stress provisions and geometric limitations, expressed in terms of specified minimum yield stress, to steels having a yield stress of 100 ksi; and the extension of plastic design rules to cover braced multi-story structures and steels having a yield stress up to 65 ksi.

As in the past, in order to avoid reference to proprietary steels which may be available from but one source, only steels which can be identified by ASTM specifications are listed. However, 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.

Steels covered by the listed ASTM specifications which have been adopted since the 1963 revision of the AISC Specification have been available for some time as proprietary products and considerable experience has already been acquired in their use. Also listed for the first time are several grades of steel having less frequent applications in building construction, which have been covered by ASTM specifications for many years. Their inclusion is for clarification. They have proven entirely satisfactory when used in accordance with the provisions of the AISC Specification.

With the extension of plastic design to steels having a yield stress in excess of the previous 36 ksi limitation, more restrictive width-thickness ratios are imposed on compression elements in Sect. 2.7 than in Sect. 1.5.1.4.1. This is because the required plastic hinge rotations in structures designed according to the provisions of Part 2 may be considerably greater than in designs executed according to the provisions of Sect. 1.5.1.4.1. The term *compact section* will continue to apply to members meeting the geometric limitations of Sect. 1.5.1.4.1 with respect to profile; shapes conforming to the requirements of Sect. 2.7 may be referred to as *plastic design sections*.

In order to simplify design calculations, the forces, stresses, and formulas related to them are now expressed in kips or kips per square inch, instead of pounds or pounds per square inch as in the past.

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 recommended practice for skylights, fire escapes, or other items not specifically enumerated in that Code. 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.

Many provisions of the Specification, notably in the sections dealing with fabrication and erection practices, have evolved from years of shop and field experience and need no further elaboration. Others are the outgrowth of recent extensive research. A separate *Commentary*, providing the background for such provisions, published by the American Institute of Steel Construction, is available at no cost to users of the Specification.

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Table of Contents

PART 1		*Page*
Sect. 1.1	Plans and Drawings	5-11
1.2	Types of Construction	5-12
1.3	Loads and Forces	5-13
1.4	Material	5-14
1.5	Allowable Stresses	5-16
1.6	Combined Stresses	5-22
1.7	Members and Connections Subject to Repeated Variation of Stress (Fatigue)	5-24
1.8	Stability and Slenderness Ratios	5-24
1.9	Width-Thickness Ratios	5-25
1.10	Plate Girders and Rolled Beams	5-26
1.11	Composite Construction	5-32
1.12	Simple and Continuous Spans	5-35
1.13	Deflections, Vibration, and Ponding	5-35
1.14	Gross and Net Sections	5.36
1.15	Connections	5-39
1.16	Rivets and Bolts	5-42
1.17	Welds	5-44
1.18	Built-Up Members	5-47
1.19	Camber	5-49
1.20	Expansion	5-50
1.21	Column Bases	5-50
1.22	Anchor Bolts	5-50
1.23	Fabrication	5-50
1.24	Shop Painting	5-55
1.25	Erection	5-56
1.26	Quality Control	5-57
PART 2		
Sect. 2.1	Scope	5-58
2.2	Structural Steel	5-58
2.3	Vertical Bracing System	5-59
2.4	Columns	5-59
2.5	Shear	5-60
2.6	Web Crippling	5-60
2.7	Minimum Thickness (Width-Thickness Ratios)	5-60
2.8	Connections	5-61
2.9	Lateral Bracing	5-61
2.10	Fabrication	5-62
APPENDIX A		5-63
APPENDIX B—Fatigue		5-105
APPENDIX C—Slender Compression Elements		5-114

Nomenclature

A_b Nominal body area of a fastener
A_c Actual area of effective concrete flange in composite design
A_{bc} Planar area of web at beam-to-column connection
A_f Area of compression flange
A_s Area of steel beam in composite design
A_{sr} Area of reinforcing steel providing composite action at point of negative moment
A_{st} Cross-sectional area of stiffener or pair of stiffeners
A_w Area of girder web
C Ratio of bolt tensile strength to tensile strength of connected part
C_a Coefficient used in Table 1-A
C_b Bending coefficient dependent upon moment gradient; equal to

$$1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2$$

C_c Column slenderness ratio dividing elastic and inelastic buckling; equal to

$$\sqrt{\frac{2\pi^2 E}{F_y}}$$

C_m Coefficient applied to bending term in interaction formula and dependent upon column curvature caused by applied moments
C_p Stiffness factor for primary member in a flat roof
C_s Stiffness factor of secondary member in a flat roof
C_v Ratio of "critical" web stress, according to the linear buckling theory, to the shear yield stress of web material; equal to

$$\frac{\pi^2 E k\sqrt{3}}{12(1-\nu^2)(h/t)^2 F_y} \quad \text{or} \quad \frac{190}{h/t}\sqrt{\frac{k}{F_y}} \quad \text{(See Sect. 1.10.5.2)}$$

C_1 Ratio of beam yield stress to column yield stress
C_2 Ratio of column yield stress to stiffener yield stress
D Factor depending upon type of transverse stiffeners
E Modulus of elasticity of steel (29,000 kips per square inch)
E_c Modulus of elasticity of concrete
F Load factor in plastic design
F_a Axial stress permitted in the absence of bending moment
F_{as} Axial compressive stress, permitted in the absence of bending moment, for bracing and other secondary members
F_b Bending stress permitted in the absence of axial force
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
F'_e Euler stress divided by factor of safety; equal to

$$\frac{12\pi^2 E}{23(Kl_b/r_b)^2}$$

F_p Allowable bearing stress

F_{sr} Stress range
F_t Allowable tensile stress
F_v Allowable shear stress
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_{yr} Yield stress of reinforcing steel providing composite action at point of negative moment
I_d Moment of inertia of steel deck on a flat roof
I_p Moment of inertia of primary member in flat roof framing
I_s Moment of inertia of secondary member in flat roof framing
I_{tr} Moment of inertia of transformed composite section
K Effective length factor
L Span length (feet)
L_p Length of primary member in a flat roof (feet)
L_s Length of secondary member in a flat roof (feet)
M Moment (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 absence of axial load
M_o Reduced plastic moment
M_p Plastic moment
N Length of bearing of applied load (inches)
N_1 Number of shear connectors equal to V_h/q or V'_h/q, as applicable
N_2 Number of shear connectors required where closer spacing is needed adjacent to point of zero moment
P Applied load (kips)
$P_{cr} = 1.70\ AF_a$
$P_e = 1.92\ AF'_e$
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
Q_s Axial stress reduction factor where width-thickness ratio of unstiffened elements exceeds limiting value given in Sect. 1.9.1.2
R Reaction or concentrated transverse load applied to beam or girder (kips)
S Spacing of secondary members in a flat roof (feet)
S_{eff} Effective section modulus corresponding to partial composite action
S_s Section modulus of steel beam used in composite design, referred to the bottom flange
S_{tr} Section modulus of transformed composite cross-section, referred to the bottom flange
T_b Specified pretension of a high strength bolt (kips)
V Statical shear on beam (kips)
V_h Total horizontal shear to be resisted by connectors under full composite action (kips)

V'_h Total horizontal shear to be resisted by 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
a Clear distance between transverse stiffeners
a' Distance required at ends of welded partial length cover plate to develop stress
b Effective width of concrete slab; actual width of stiffened and unstiffened compression elements
b_e Effective width of stiffened compression element
b_f Flange width of rolled beam or plate girder
c Distance from neutral axis to extreme fiber of beam
d Depth of beam or girder. Also diameter of roller or rocker bearing
d_c Column web depth clear of fillets
e Horizontal displacement, in the direction of the span, between top and botton of simply supported beam at its ends
f Axial compression load on member divided by effective area (kips per square inch)
f_a Computed axial stress
f_b Computed bending stress
f'_c Specified compression strength of concrete
f_t Computed tensile stress
f_v Computed shear stress
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
h Clear distance between flanges of a beam or girder
k Coefficient relating linear buckling strength of a plate to its dimensions and condition of edge support. Also distance from outer face of flange to web toe of fillet of rolled shape or equivalent distance on welded section
l Actual unbraced length (inches)
l_b Actual unbraced length in plane of bending (inches)
l_{cr} Critical unbraced length adjacent to plastic hinge (inches)
n Modular ratio; equal to E/E_c
q Allowable horizontal shear to be resisted by a shear connector
r Governing radius of gyration
r_b Radius of gyration about axis of concurrent bending
r_y Lesser radius of gyration
s Spacing (pitch) between successive holes in line of stress
t Girder, beam, or column web thickness
t_b Beam flange thickness at rigid beam-to-column connection
t_f Flange thickness
t_t Thickness of thinner part joined by partial penetration groove weld
w Length of channel shear connectors
x Subscript relating symbol to strong axis bending
y Subscript relating symbol to weak axis bending
α Ratio of hybrid girder web yield stress to flange yield stress
β Ratio S_{tr}/S_s or S_{eff}/S_s
ν Poisson's ratio, may be taken as 0.3 for steel

SPECIFICATION FOR THE

Design, Fabrication and Erection of Structural Steel for Buildings

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 and Long-span 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, in so far 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 tier buildings designed as Type 2 construction (that is, 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 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 non-elastic 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 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 member 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 traveling crane support girders and their connections	25 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 rail, one-half on each side of the runway, and shall be considered as acting in either direction normal to the runway rail.

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.

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 Sect. 1.3.1, 1.3.2, 1.3.5 and 1.3.6 above shall be not less than those recommended in the *USA Standard Building Code Requirements for Minimum Design Loads in Buildings and Other Structures*, USASI A58.1, latest edition.

SECTION 1.4 MATERIAL

1.4.1 Structural Steel

1.4.1.1 Material conforming to one of the following listing (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 Hot-Rolled Steel Sheet and Strip, ASTM A375
High-Strength Structural Steel, ASTM A440
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
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
High-Yield Strength Quenched and Tempered Alloy Steel Plate, Suitable for Welding, ASTM A514. (Quenched and tempered alloy steel structural shapes and seamless mechanical tubing meeting all of the mechanical and chemical requirements of A514 steel, except that the specified maximum tensile strength may be 140,000 psi for structural shapes and 145,000 psi for seamless mechanical tubing, shall be considered as A514 steel.)

Certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6 and the governing specification shall constitute sufficient evidence of conformity with one of the above ASTM specifications. 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 specifications, latest edition:

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

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

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

Carbon Steel Forgings for General Industrial Use, ASTM A235, Class C1, F and G. (Class C1 Forgings that are to be welded shall be ordered in accordance with Supplemental Requirements S5 of A235.)
Alloy Steel Forgings for General Industrial Use, ASTM A237, Class A

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

1.4.3 Rivets

Rivets shall conform to the provisions of the *Specification for Structural Rivets,* ASTM A502, Grade 1 or Grade 2, latest edition:

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

1.4.4 Bolts

High strength steel bolts shall conform to one of the following specifications, latest edition:

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

Other bolts shall conform to the *Specification for Low-Carbon Steel Externally and Internally Threaded Standard Fasteners,* ASTM A307, latest edition, hereinafter designated as A307 bolts.

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

1.4.5 Filler Metal for Welding

Welding electrodes for manual shielded metal-arc welding shall conform to the *Specification for Mild Steel Covered Arc-Welding Electrodes,* AWS A5.1, latest edition, or the *Specification for Low-Alloy Steel Covered Arc-Welding Electrodes,* AWS A5.5, latest edition.

Bare electrodes and granular flux used in the submerged-arc process shall conform to F60 or F70 AWS-flux classifications of the *Specification for Bare Mild Steel Electrodes and Fluxes for Submerged Arc Welding,* AWS A5.17, latest edition, or the provisions of Sect. 1.17.3.

E60S or E70S electrodes used in the gas metal-arc process shall conform to the *Specification for Mild Steel Electrodes for Gas Metal-Arc Welding,* AWS A5.18, latest edition, or the provisions of Sect. 1.17.3; E60T or E70T electrodes used in the flux cored-arc process shall conform to the *Specification for Mild Steel Electrodes for Flux-Cored-Arc Welding,* AWS A5.20, latest edition, or the provisions of Sect. 1.17.3.

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

SECTION 1.5 ALLOWABLE STRESSES*

Except as provided in Sects. 1.6, 1.7, 1.10, 1.11 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 they are rounded off in Appendix A.

1.5.1 Structural Steel

1.5.1.1 Tension

On the net section, except at pin holes:

$$F_t = 0.60F_y$$

but not more than 0.5 times the minimum tensile strength of the steel.

On the net section at pin holes in eyebars, pin-connected plates or built-up members:

$$F_t = 0.45F_y$$

For tension on threaded parts see Table 1.5.2.1.

1.5.1.2 Shear

On the gross section: $F_v = 0.40F_y$

(The gross section of rolled and fabricated shapes may be taken as the product of the overall depth and the thickness of the web. See Sect. 1.10 for reduction required for thin webs. For discussion of high shear stress within boundaries of rigid connections of members whose webs lie in a common plane, see Commentary Sect. 1.5.1.2.)

1.5.1.3 Compression

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

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

where

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

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

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):

$$F_a = 0.75F_y$$

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:

a. The flanges shall be continuously connected to the web or webs.
b. The width-thickness ratio of unstiffened projecting elements of the compression flange, as defined in Sect. 1.9.1.1, shall not exceed $52.2/\sqrt{F_y}$.
c. The width-thickness ratio of stiffened elements of the compression flange, as defined in Sect. 1.9.2.1, shall not exceed $190/\sqrt{F_y}$.
d. The depth-thickness ratio of the web or webs shall not exceed the value

$$d/t = 412\left(1 - 2.33\frac{f_a}{F_y}\right)\Big/\sqrt{F_y} \tag{1.5-4}$$

except that it need not be less than $257/\sqrt{F_y}$.
e. The compression flange shall be supported laterally at intervals not to exceed $76.0b_f/\sqrt{F_y}$ nor $\dfrac{20{,}000}{(d/A_f)F_y}$

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 sub-paragraphs a, b, c, d and e above and are continuous over supports or are rigidly framed to columns by means of rivets,

* For this case, K is taken as unity.

high strength bolts or welds, may be proportioned for $\frac{9}{10}$ 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 $\frac{1}{10}$ 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 $\frac{1}{10}$ 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 $52.2/\sqrt{F_y}$ but is less than $95.0/\sqrt{F_y}$, may be designed on the basis of an allowable bending stress

$$F_b = F_y\left[0.733 - 0.0014\left(\frac{b_f}{2t_f}\right)\sqrt{F_y}\right] \qquad (1.5\text{-}5)$$

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 a and b, and bent about their minor axis (except members of A514 steel); solid round and square bars; and solid rectangular sections bent about their weaker axis:

$$F_b = 0.75F_y$$

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 and whose compression flange is braced laterally at intervals not exceeding $2{,}500/F_y$ times the transverse distance out-to-out of the webs:

$$F_b = 0.60F_y$$

1.5.1.4.5 Tension 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:

$$F_b = 0.60F_y$$

1.5.1.4.6a Compression on extreme fibers of flexural members included under Sect. 1.5.1.4.5, 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 $\sqrt{\dfrac{102 \times 10^3 C_b}{F_y}} \leqslant \dfrac{l}{r_T} \leqslant \sqrt{\dfrac{510 \times 10^3 C_b}{F_y}}$

$$F_b = \left[\frac{2}{3} - \frac{F_y(l/r_T)^2}{1{,}530 \times 10^3 C_b}\right]F_y \qquad (1.5\text{-}6a)$$

---

* Only Formula (1.5-7) applicable to channels.

** See Commentarv Sects. 1.5.1.4.5 and 1.5.1.4.6, last two paragraphs.

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

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

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

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

In the foregoing,

l = distance between cross-sections braced against twist or lateral displacement of the compression flange

r_T = radius of gyration of a section comprising the compression flange plus one-third of the compression web area, taken about an axis in the plane of the web

A_f = area of the compression flange

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. C_b shall also be taken as unity in computing the value of F_{bx} and F_{by} to be used in Formula (1.6-1a). See Sect. 1.10 for further limitation in plate girder flange stress.

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.

1.5.1.4.6b Compression on extreme fibers of flexural members included under Sect. 1.5.1.4.5, but not included in Sect. 1.5.1.4.6a:

$$F_b = 0.60 F_y$$

provided that sections bent about their major axis are braced laterally in the region of compression stress at intervals not exceeding $76.0 b_f/\sqrt{F_y}$.

1.5.1.5 Bearing (on contact area)

1.5.1.5.1 Milled surfaces, including bearing stiffeners and pins in reamed, drilled, or bored holes:

$$F_p = 0.90 F_y{**}$$

---

* C_b can be conservatively taken as unity. For smaller values see Appendix A, Fig. A1, p. 5-104.

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

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 in inches.

1.5.2 Rivets, Bolts, and Threaded Parts

1.5.2.1 Allowable tension and shear stresses on rivets, bolts and threaded parts (kips per square inch of area of rivets before driving or unthreaded-body area of bolts and threaded parts except as noted) shall be as given in 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.

TABLE 1.5.2.1

Description of Fastener	Tension (F_t)	Shear (F_v)	
		Friction-Type Connections	Bearing-Type Connections
A502, Grade 1, hot-driven rivets	20.0		15.0
A502, Grade 2, hot-driven rivets	27.0		20.0
A307 bolts	20.0[1]		10.0
Threaded parts[3] of steel meeting the requirements of Sect. 1.4.1	$0.60F_y$ [1]		$0.30F_y$
A325 and A449 bolts, when threading is *not* excluded from shear planes	40.0[2]	15.0	15.0
A325 and A449 bolts, when threading is excluded from shear planes	40.0[2]	15.0	22.0
A490 bolts, when threading is *not* excluded from shear planes	54.0[2,4]	20.0	22.5
A490 bolts, when threading is excluded from shear planes	54.0[2,4]	20.0	32.0

[1] Applied to tensile stress area equal to $0.7854\left(D - \frac{0.9743}{n}\right)^2$ where D is the major thread diameter and n is the number of threads per inch.

[2] Applied to the nominal bolt area.

[3] Since the nominal area of an upset rod is less than the stress area, the former area will govern.

[4] Static loading only.

TABLE 1.5.3

Kind of Stress	Permissible Stress	Required Electrode[4]	"Matching" Base Metal[4]
Tension and Compression parallel to axis of any complete penetration groove weld	Same as for base metal[1]		
Tension normal to effective throat of complete-penetration groove weld	Same as allowable tensile stress for base metal[1]		
Compression normal to effective throat of complete or partial-penetration groove weld	Same as allowable compressive stress for base metal[1]		
Shear on effective throat of complete-penetration groove weld and partial-penetration groove weld	Same as allowable shear stress for base metal[1]		
Shear stress on effective[2] throat of fillet weld regardless of direction of application of load; tension normal[3] to the axis on the effective throat of a partial-penetration groove weld; and shear stress on effective area of a plug or slot weld. The given stresses shall also apply to such welds made with the specified electrode on steel having a yield stress greater than that of the "matching" base metal. The permissible stress, regardless of electrode classification used, shall not exceed that given in the table for the weaker "matching" base metal being joined.	18.0 ksi	AWS A5.1, E60XX electrodes AWS A5.17, F6X-EXXX flux-electrode combination AWS A5.20, E60T-X electrodes	A500 Grade A A570 Grade D
	21.0 ksi	AWS A5.1 or A5.5, E70XX electrodes AWS A5.17, F7X-EXXX flux-electrode combination AWS A5.18, E70S-X or E70U-1 electrodes AWS A5.20, E70T-X electrodes	A36 A53 Grade B A242 A375 A441 A500 Grade B A501 A529 A570 Grade E A572 Grades 42 to 60 A588
	24.0 ksi	AWS A5.5, E80XX electrodes Grade 80 Submerged Arc, Gas Metal-Arc or Flux Cored Arc Weld Metal	A572 Grade 65
	27.0 ksi	AWS A5.5, E90XX electrodes Grade 90 Submerged Arc, Gas Metal-Arc or Flux Cored Arc Weld Metal	A514 over 2½ in. thick
	30.0 ksi	AWS A5.5, E100XX electrodes Grade 100 Submerged Arc, Gas Metal-Arc or Flux Cored Arc Weld Metal	A514 over 2½ in. thick
	33.0 ksi	AWS A5.5, E110XX electrodes Grade 110 Submerged Arc, Gas Metal-Arc or Flux Cored Arc Weld Metal	A514 2½ in. and less in thickness

[1] The electrode or flux specified in Table 1.17.2 shall be used.

[2] For definition of effective throat of fillet welds and partial penetration groove welds see Sect. 1.14.7.

[3] 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 tension or compression stress in these elements parallel to the axis of the welds.

[4] Only low-hydrogen electrodes shall be used on A242, A441, A514, A572 and A588.

1.5.2.2 Allowable bearing stress on projected area of bolts in bearing-type connections and on rivets:

$$F_p = 1.35F_y$$

where F_y is the yield stress of the connected part. (Bearing stress is not restricted in friction-type connections assembled with A325, A449 or A490 bolts.)

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.

1.5.4 Cast Steel and Steel Forgings

Allowable stresses 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	$F_p = 0.25f'_c$
On one-third of this area	$F_p = 0.375f'_c$

where f'_c is the specified compression strength of the concrete.

1.5.6 Wind and Seismic Stresses

Allowable stresses may be increased one-third above the values provided in Sect. 1.5.1 1.5.2, 1.5.3, 1.5.4 and 1.5.5 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 one-third stress increase.

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:

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

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

When $\dfrac{f_a}{F_a} \leqslant 0.15$, Formula (1.6-2) may be used in lieu of Formulas (1.6-1a) and (1.6-1b)

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

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

F_a = axial stress that would be permitted if axial force alone existed

F_b = compressive bending stress that would be permitted if bending moment alone existed

$F'_e = \dfrac{12\pi^2 E}{23(Kl_b/r_b)^2}$ (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 0.6 F_y, F'_e may be increased one-third in accordance with Sect. 1.5.6.)

f_a = computed axial stress

f_b = computed compressive bending stress at the point under consideration

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

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

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

   where M_1/M_2 is the ratio of the smaller to larger moments at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1/M_2 is positive when the member is bent in reverse curvature and negative when it is bent in single curvature.
3. For compression members in frames braced against joint translation in the plane of loading and 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, in kips per square inch, produced by forces applied to the connected parts, shall not exceed the following:

For A502 Grade 1 rivets	$F_t = 28.0 - 1.6f_v \leqslant 20.0$
For A502 Grade 2 rivets.	$F_t = 38.0 - 1.6f_v \leqslant 27.0$
For A307 bolts (applied to stress area)	$F_t = 28.0 - 1.6f_v \leqslant 20.0$
For A325 and A449 bolts in bearing-type joints	$F_t = 50.0 - 1.6f_v \leqslant 40.0$
For A490 bolts in bearing-type joints .	$F_t = 70.0 - 1.6f_v \leqslant 54.0$

where f_v, the shear stress produced by the same forces, shall not exceed the value for shear given in Sect. 1.5.2.

For bolts used in friction-type joints, the shear stress allowed in Sect. 1.5.2 shall be reduced so that:

For A325 and A449 bolts	$F_v \leqslant 15.0(1 - f_t A_b/T_b)$
For A490 bolts.	$F_v \leqslant 20.0(1 - f_t A_b/T_b)$

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.

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.

1.7.2 Design for Fatigue

Members and their connections, subject to fatigue loading as defined in Appendix B, shall be proportioned to satisfy the stress range limitations as provided therein.

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.

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 Sidesway Prevented

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

1.8.3 Sidesway Not Prevented

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, Kl/r, of tension members, other than rods, preferably should not exceed:

For main members	240
For bracing 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-shape members and tees shall be taken as one-half the full nominal width. The thickness of a sloping flange shall be measured halfway 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	$76.0/\sqrt{F_y}$
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	$95.0/\sqrt{F_y}$
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 which 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 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.

SECTION 1.10 PLATE GIRDERS AND ROLLED BEAMS

1.10.1 Proportions

Riveted and welded plate girders, cover-plated 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. 1.14.3, exceeds 15 percent of the gross flange area, the excess shall be deducted.

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 clear distance between flanges, in inches, shall not exceed

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

times the web thickness, where F_y is the yield stress of the compression flange, except that it need not be less than $2{,}000/\sqrt{F_y}$ when transverse stiffeners are provided, spaced not more than $1\frac{1}{2}$ times the girder depth.

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* 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.

† 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.

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. But the longitudinal spacing shall not exceed the maximum permitted, respectively, for compression or tension members in Sect. 1.18.2.3 or 1.18.3.1. 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 cut-off point and the extended portion shall be attached to the beam or girder by rivets, high strength bolts (friction-type joint), or fillet welds adequate, at the applicable stresses allowed in Sect. 1.5.2 or 1.5.3 or Sect. 1.7, to develop the cover plate's portion of the flexural stresses in the beam or girder at the theoretical cut-off 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:

1. A distance equal to the width of the cover plate when there is a continuous weld equal to or larger than $\frac{3}{4}$ of the plate thickness across the end of the plate and continued 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 $\frac{3}{4}$ of the plate thickness across the end of the plate and continued 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

* 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 cut-off point.

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

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 a width equal to 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 ¾ 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).

$$F_v = \frac{F_y}{2.89}(C_v) \leqslant 0.4F_y \qquad (1.10\text{-}1)$$

where

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

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

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

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

t = thickness of web, in inches
a = clear distance between transverse stiffeners, in inches
h = clear distance between flanges, in inches

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 $C_v \leqslant 1$, the allowable shear given by Formula (1.10-2) may be used in lieu of the value given by Formula (1.10-1).

$$F_v = \frac{F_y}{2.89}\left[C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}}\right] \leqslant 0.4F_y \qquad (1.10\text{-}2)^*$$

1.10.5.3 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 Formulas (1.10-1) or (1.10-2), as applicable, and the ratio a/h shall not exceed $\left(\frac{260}{h/t}\right)^2$, nor 3.0.

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* Formula (1.10-2) recognizes the contribution of tension field action. For values of F_v provided by this formula, see Tables 3-36 through 3-100 in Appendix A.

In girders designed on the basis of tension field action, the spacing between stiffeners at end panels and panels containing large holes shall be such that the smaller panel dimension, a or h, shall not exceed $348t/\sqrt{f_v}$.

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 defined in Sect. 1.10.5.2

$Y = \dfrac{\text{yield stress of web steel}}{\text{yield stress of stiffener steel}}$

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 the formula

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

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.

Intermediate stiffeners may be stopped short of the tension flange a distance not to exceed 4 times the web thickness, provided bearing is not needed to transmit a concentrated load or reaction. 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 stress in the compression flange shall not exceed

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

where

F_b = applicable bending stress given in Sect. 1.5.1
A_w = area of the web
A_f = area of compression flange

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

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

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 $0.6F_y$ nor

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

where

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

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 be complete penetration groove welds and shall develop the full strength of the smaller spliced section. Other types of splices in cross-sections of plate girders and in 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 the value of $0.75F_y$; otherwise, bearing stiffeners shall be provided. The governing formulas shall be:

For interior loads,

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

For end-reactions,

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

where

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

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

when the flange is restrained against rotation, nor

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

when the flange is not so restrained.

These stresses shall be computed as follows:

Concentrated loads and loads distributed over partial length of a panel shall be divided by the product of the web thickness and the girder depth or the length of panel in which the load is placed, whichever is the lesser panel dimension.

Any other distributed loading, in kips per linear inch of length, shall be divided by the web thickness.

1.10.11 Rotational Restraint at Points of Support

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

SECTION 1.11 COMPOSITE CONSTRUCTION

1.11.1 Definition

Composite construction shall consist of steel beams or girders supporting a reinforced concrete slab, so inter-connected 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 one-fourth of the span of the beam, and its effective projection beyond the edge of the beam shall not be taken as more than one-half the clear distance to the adjacent beam, nor more than eight times the slab thickness. When the slab is present on only one side of the beam, the effective width of the concrete flange (projection beyond the beam) shall be taken as not more than one-twelfth of the beam span, nor six times its thickness, nor one-half 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 inter-connected to the concrete by natural bond, without additional anchorage, provided the top of the beam is at least 1½ inches below the top and 2 inches above the bottom of the slab, and 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.

Reinforcement parallel to the beam within the effective width of the slab, when anchored in accordance with the provisions of the applicable code, may be included in computing the properties of composite sections subject to negative bending moment, 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. The compression area of the concrete on the compression side of the neutral axis shall be treated as an equivalent area of steel by dividing it by the modular ratio n.

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

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

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
S_{tr} = section modulus of the transformed composite section referred to its bottom flange

For construction without temporary shoring, the value of the section modulus of the transformed composite section used in stress calculations (referred to the bottom flange of the steel beam) shall not exceed

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

where 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 its bottom flange). The steel beam alone, supporting the loads before the concrete has hardened, shall not be stressed to more than the applicable bending stress given in Sect. 1.5.1.

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 dead and live load.

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 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).

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

and

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

where

f'_c = specified compression strength of concrete
A_c = actual area of effective concrete flange defined in Sect. 1.11.1
A_s = area of steel beam

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

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

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
F_{yr} = specified minimum yield stress of the longitudinal reinforcing steel

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. Working values for use with concrete having aggregate not conforming to ASTM C33 and for connector types other than those shown in Table 1.11.4 must be established by a suitable test program.

TABLE 1.11.4

Connector	Allowable Horizontal Shear Load (q) (kips) (Applicable only to concrete made with ASTM C33 aggregates)		
	f'_c (kips per square inch)		
	3.0	3.5	4.0
½″ diam. × 2″ hooked or headed stud	5.1	5.5	5.9
⅝″ diam. × 2½″ hooked or headed stud	8.0	8.6	9.2
¾″ diam. × 3″ hooked or headed stud	11.5	12.5	13.3
⅞″ diam. × 3½″ hooked or headed stud	15.6	16.8	18.0
3″ channel, 4.1 lb.	$4.3w$	$4.7w$	$5.0w$
4″ channel, 5.4 lb.	$4.6w$	$5.0w$	$5.3w$
5″ channel, 6.7 lb.	$4.9w$	$5.3w$	$5.6w$

w = length of channel in inches.

For incomplete 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 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 (1.11-6).

$$N_2 = \frac{N_1\left[\dfrac{M\beta}{M_{max}} - 1\right]}{\beta - 1} \qquad (1.11\text{-}6)$$

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

Connectors required in the region of negative bending on a continuous beam 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 concrete cover in all directions. Unless located directly over the web, the diameter of studs shall not be greater than 2.5 times the thickness of the flange to which they are welded.

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 $1/360$ 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 rain water, 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

$$C_p + 0.9C_s \leqslant 0.25 \text{ and } I_d \geqslant 25S^4/10^6$$

where

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

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 member)

S = Spacing of secondary members, feet

I_p = Moment of inertia for primary members, inches^4

I_s = Moment of inertia for secondary member, 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 SECTIONS

1.14.1 Definitions

The gross section 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 section shall be determined by substituting for the gross width the net width computed in accordance with Sects. 1.14.3 to 1.14.6, inclusive.

1.14.2 Application

Unless otherwise specified, tension members shall be designed on the basis of net section. Compression members shall be designed on the basis of gross section. Beams and girders shall be designed in accordance with Sect. 1.10.1.

1.14.3 Net Section

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

$$\frac{s^2}{4g}$$

where

s = longitudinal spacing (pitch, in inches) of any two consecutive holes
g = transverse spacing (gage, in inches) of the same two holes

The critical net section of the part is obtained from that chain which gives the least net width; however, the net section taken through a hole shall in no case be considered as more than 85 percent of the corresponding gross section.

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

1.14.4 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.

1.14.5 Size of Holes

In computing net area the diameter of a rivet or bolt hole shall be taken as $\frac{1}{8}$-inch greater than the nominal diameter of the rivet or bolt.

1.14.6 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 section 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.

The minimum net section across the pin hole, transverse to the axis of the member, in pin-connected plates and built-up members shall be determined at the stress allowed for such sections in Sect. 1.5.1.1. The net section beyond the pin hole, parallel to the axis of the member, shall not be less than $\frac{2}{3}$ of the net section across the pin hole. The corners beyond the pin hole may be cut

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* Members having a different thickness at the pin hole location are termed "built-up."

at 45° to the axis of the member provided the net section 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. The parts of members built up at the pin hole shall be attached to each other by sufficient fasteners to support the stress delivered to them by the pin.

The distance transverse to the axis of a pin-connected plate or any separated 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. The diameter of the pin hole shall not be more than $\frac{1}{32}$-inch greater than the diameter of the pin. In the case of pin-connected plates of uniform thickness, for steels having a yield stress greater than 70 ksi, the diameter of the pin hole shall not exceed 5 times the plate thickness.

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.

1.14.7 Effective Areas of Weld Metal

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

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.

The effective area of fillet welds in holes and slots shall be computed as above specified for fillet welds, using for effective length, the length of centerline 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.

The effective length of a fillet weld shall be the overall length of full-size fillet including returns.

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

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.

The effective throat thickness of a complete penetration groove weld (i.e., a groove weld conforming to the requirements of Sect. 1.23.6) shall be the thickness of the thinner part joined.

The effective throat thickness of single and double partial penetration groove welds shall be the depth of the groove, except that the effective throat thickness of a bevel joint made by manual shielded metal-arc welding shall be $\frac{1}{8}$-inch less than the depth of the groove, and the effective throat thickness of each weld shall be not less than $\sqrt{t_t/6}$, where t_t is the thickness of the thinner part connected by the weld.

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.

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 permit the ends of the beam to rotate sufficiently to accommodate its deflection by providing for a horizontal displacement of the top flange determined as follows:

$e = 0.007d$, when the beam is designed for full uniform load and for live load deflection not exceeding $1/360$ of the span

$= \dfrac{f_b L}{3{,}600}$, when the beam is designed for full uniform load producing the stress f_b at mid-span

where

e = the horizontal displacement of the end of the top flange, in the direction of the span, in inches
f_b = the flexural stress in the beam at mid-span, in kips per square inch
d = the depth of the beam, in inches
L = the span of the beam, in feet

1.15.5 Restrained Members

Fasteners or welds for end connections of beams, girders and trusses not conforming to the requirements of Sect. 1.15.4 shall be designed for the combined effect of end reaction shear and tensile or compressive stresses resulting from moment induced by the rigidity of the connection when the member is fully loaded.*

* For a discussion of high column web shear stress opposite rigid beam connections, see Commentary Sect. 1.5.1.2.

When fully restrained beams are framed to the flange of an I- or H-shape column, stiffeners shall be provided on the column web as follows:

Opposite the compression flange when $t < \dfrac{C_1 A_f}{t_b + 5k}$ (1.15-1)

or when $t \leqslant \dfrac{d_c \sqrt{F_y}}{180}$ (1.15-2)

Opposite the tension flange when $t_f < 0.4\sqrt{C_1 A_f}$ (1.15-3)

where

t = thickness of web to be stiffened
k = distance from outer face of flange to web toe of fillet of member to be stiffened, if a member is a rolled shape
= flange thickness plus the distance to the farthest toe of the connecting weld, if a member is a welded section
t_b = thickness of flange delivering concentrated load
t_f = thickness of flange of member to be stiffened
A_f = area of flange delivering concentrated load
d_c = column web depth clear of fillets
C_1 = ratio of beam flange yield stress to column yield stress
C_2 = ratio of column yield stress to stiffener yield stress

The area of such stiffeners, A_{st}, shall be such that

$$A_{st} \geqslant [C_1 A_f - t(t_b + 5k)]C_2 \qquad (1.15\text{-}4)$$

Their ends shall be welded to the inside face of the flange opposite the concentrated tensile load, so as to transfer the load from the beam flange to the column web. The stiffeners may be fitted against the inside face of the flange opposite the concentrated compression load. When the concentrated load delivered by a beam occurs on one side only, the web stiffener need not exceed one-half the depth of the member, but the welding connecting it to the web shall be sufficient to develop $F_y A_{st}$.

1.15.6 Fillers

When rivets or bolts carrying computed stress pass through fillers thicker than ¼-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 ¼-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 ¼-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.

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 accordance with the provisions of Sect. 1.16.1 as a friction-type connection prior to welding 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 dead loads, and the welding need be adequate only to carry all additional stress.

1.15.11 High Strength Bolts (in Friction-Type Joints) 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.

Roof-truss splices and connections of trusses to columns, column splices, column bracing, knee braces and crane supports, in all structures carrying cranes of over 5-ton capacity.

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

Use of high strength bolts shall conform to the provisions of the *Specifications for Structural Joints Using ASTM A325 or A490 Bolts* as approved by the Research Council on Riveted and Bolted Structural Joints. ASTM A449 bolts no greater than 1½ inches in diameter may be used in lieu of ASTM A325 bolts, provided that a hardened washer is installed under the bolt head. However, nuts used with A449 bolts 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 half 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 Pitch

The minimum distance between centers of rivet and bolt holes shall be not less than 2⅔ times the nominal diameter of the rivet or bolt but preferably not less than 3 diameters.

1.16.5 Minimum Edge Distance

The minimum distance from the center of a rivet or bolt hole to any edge, used in design or in preparation of shop drawings, shall be that given in Table 1.16.5.

TABLE 1.16.5

Rivet or Bolt Diameter (Inches)	Minimum Edge Distance for Punched, Reamed or Drilled Holes (Inches)	
	At Sheared Edges	At Rolled Edges of Plates, Shapes or Bars or Gas Cut Edges**
½	⅞	¾
⅝	1⅛	⅞
¾	1¼	1
⅞	1½*	1⅛
1	1¾*	1¼
1⅛	2	1½
1¼	2¼	1⅝
Over 1¼	1¾ × Diameter	1¼ × Diameter

* These may be 1¼-in. at the ends of beam connection angles.

** All edge distances in this column may be reduced ⅛-in. when the hole is at a point where stress does not exceed 25% of the maximum allowed stress in the element.

1.16.6 Minimum Edge Distance in Line of Stress

1.16.6.1 In connections of tension members, where there are not more than two rivets in a line parallel to the direction of stress, the distance from the center of the end rivet to that end of the connected part toward which the stress is directed shall be not less than the area of the rivet divided by the thickness of the connected part for rivets in single shear or twice this distance for rivets in double shear.

1.16.6.2 In bearing-type connections of tension members, where there are not more than two high strength bolts in a line parallel to the direction of stress, the distance from the center of the end bolt to that end of the connected part toward which the stress is directed shall be not less than A_bC/t for single shear or $2A_bC/t$ for double shear, where A_b is the nominal cross-sectional area of the bolt, t is the thickness of the connected part, and C is the ratio of specified minimum tensile strength of the bolt to the specified minimum tensile strength of the connected part.

1.16.6.3 However, the end distance prescribed in Sects. 1.16.6.1 and 1.16.6.2 may be decreased in such proportion as the fastener stress is less than that permitted in Sect. 1.5.2, but it shall not be less than the distance specified in Sect. 1.16.5 and need not exceed 1½ times the transverse spacing of fasteners.

1.16.6.4 When more than two fasteners are provided in the line of stress, the provisions of Sect. 1.16.5 shall govern.

1.16.7 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 plate, but shall not exceed 6 inches.

SECTION 1.17 WELDS

1.17.1 Welder, Tacker, and Welding Operator Qualifications

Welds shall be made only by welders, tackers, and welding operators who have been previously qualified by tests as prescribed in the *Code for Welding in Building Construction*, AWS D1.0-69, of the American Welding Society to perform the type of work required.

1.17.2 Qualification of Weld and Joint Details

Weld grooves for complete and partial penetration welds which are accepted without welding procedure qualification under the provisions of AWS D1.0-69, may be used under this specification without welding procedure qualification.

Joint forms, details, welding processes, or welding procedures other than those included in the foregoing may be employed provided they shall have been qualified in accordance with the requirements of AWS D1.0-69.

The electrodes or flux specified in Table 1.17.2 shall be used in making complete penetration groove welds designed on the basis of the allowable stresses for the base metal, as provided in Table 1.5.3. The electrodes and fluxes as listed in Table 1.5.3 may be used in making fillet welds and partial penetration groove welds.

Welding of A440 steel is not recommended.

1.17.3 Submerged-Arc, Gas Metal-Arc, and Flux Cored-Arc Welding of High Strength Steel

Electrodes for use in submerged-arc, gas metal-arc, and flux cored-arc welding listed in Tables 1.5.3 and 1.17.2 by grade designation and not covered in AWS A5.17, A5.18 or A5.20, shall meet the provisions of Sections 412, 417 or 418 of AWS D1.0-69, as applicable.

1.17.4 Electroslag and Electrogas Welding

Weld metal deposited by the electroslag or electrogas welding process shall conform to the requirements of Article 422 of AWS D1.0-69. Weldments of A514 steel, made by either process, shall be quenched and tempered after welding.

1.17.5 Minimum Size of Fillet Welds

In joints connected only by fillet welds, the minimum size of fillet weld to be used shall be as shown in Table 1.17.5. 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:

TABLE 1.17.5

Material Thickness of Thicker Part Joined (Inches)	Minimum Size of Fillet Weld (Inches)	Material Thickness of Thicker Part Joined (Inches)	Minimum Size of Fillet Weld (Inches)
To ¼ inclusive	⅛	Over 1½ to 2¼	⅜
Over ¼ to ½	3/16	Over 2¼ to 6	½
Over ½ to ¾	¼	Over 6	⅝
Over ¾ to 1½	5/16		

TABLE 1.17.2

Base Metal[3]	Welding Process[1,2]			
	Shielded Metal-Arc	Submerged-Arc	Gas Metal-Arc	Flux Cored-Arc
ASTM A36, A53 Gr. B, A375, A500, A501, A529, and A570 Gr. D and E	AWS A5.1 or A5.5, E60XX or E70XX[3]	AWS A5.17 F6X or F7X-EXXX	AWS A5.18 E70S-X or E70U-1	AWS 5.20 E60T-X or E70T-X (except EXXT-2 and EXX-3)
ASTM A242, A441, A572 Grades 42 thru 60 and A588[4]	AWS A5.1 or A5.5, E70XX[5]	AWS A5.17 F7X-EXXX	AWS A5.18 E70S-X or E70U-1	AWS 5.20 E70T-X (except E70T-2 and E70T-3)
ASTM A572 Grade 65	AWS A5.5 E80XX[5]	Grade F80	Grade E80S	Grade E80T
ASTM A514 over 2½″ thick	AWS A5.5 E100XX[5]	Grade F100	Grade E100S	Grade E100T
ASTM A514 2½″ thick and under	AWS A5.5 E110XX[5]	Grade F110	Grade E110S	Grade E110T

Use of the same type filler metal having next higher mechanical properties is permitted.

[1] When welds are to be stress relieved the deposited weld metal shall not exceed 0.05 percent vanadium.

[2] See Article 422 of AWS D1.0-69 for electroslag and electrogas weld metal requirements.

[3] On joints involving base metals of different yield strengths, filler metals applicable to the lower yield strength may be used.

[4] For architectural exposed bare unpainted applications, the deposited weld metal shall have similar atmospheric corrosion resistance and coloring characteristics as the base metal used. The steel manufacturer's recommendation shall be followed.

[5] Low hydrogen classifications.

1.17.6 Maximum Effective Size of Fillet Welds

The maximum size of a fillet weld that may be assumed in the design of a connection shall be such that the stresses in the adjacent base material do not exceed the values allowed in Sect. 1.5.1. The maximum size that may be used along edges of connected parts shall be:

1. Along edges of material less than 1/4-inch thick, the maximum size may be equal to the thickness of the material.
2. Along edges of material 1/4-inch or more in thickness, the maximum size shall be 1/16-inch less than the thickness of the material, unless the weld is especially designated on the drawings to be built out to obtain full throat thickness.

1.17.7 Length of Fillet Welds

The minimum effective length of a strength fillet weld shall be not less than 4 times the nominal size, or else the size of the weld shall be considered not to exceed one-fourth 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.

1.17.8 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 1/2 inches.

1.17.9 Lap Joints

The minimum amount of lap on lap joints shall be 5 times the thickness of the thinner part joined and 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.

1.17.10 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 twice 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.

1.17.11 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. 1.14.7. Fillet welds in holes or slots are not to be considered plug or slot welds.

1.17.12 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 5⁄16-inch, rounded to the next greater odd 1⁄16-inch, nor greater than 2¼ 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 5⁄16-inch, rounded to the next greater odd 1⁄16-inch, nor shall it be greater than 2¼ times the thickness of the weld. The ends of the slot shall be semicircular or shall have the corners rounded to a radius not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

The minimum spacing of lines of slot welds in a direction transverse to their length shall be 4 times the width of the slot. The minimum 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 5⁄8-inch or less in thickness shall be equal to the thickness of the material. In material over 5⁄8-inch in thickness, it shall be at least one-half the thickness of the material but not less than 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 1½ 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 $127/\sqrt{F_y}$ when rivets are provided on all gage lines at each section, or when intermittent welds are provided along the edges of the components, but this spacing shall not exceed 12 inches. When rivets or bolts 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 rivets, bolts or welds connecting them to the components of the member. Intermediate tie plates shall have a length not less than one-half of this distance. The thickness of tie plates shall be not less than $1/50$ of the distance between the lines of rivets, bolts or welds connecting them to the segments of the members. In riveted and bolted construction the pitch in tie plates shall be not more than 6 diameters and the tie plates shall be connected to each segment by at least three fasteners. In welded construction, the welding on each line connecting a tie plate shall aggregate not less than one-third 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. In determining the required section for lacing bars, Formula (1.5-1) or (1.5-2) shall be used, l being taken as the unsupported length of the lacing bar between rivets 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 rivets 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 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 direction of stress shall be not less than the transverse distance between nearest lines of connecting rivets, bolts or welds; and the periphery of the holes at all points shall have a minimum radius of 1½ inches.

1.18.3 Tension Members

1.18.3.1 The longitudinal spacing of rivets, bolts 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 rivets, bolts 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 two-thirds the distance between the lines of rivets, bolts or welds connecting them to the components of the member. The thickness of such tie plates shall not be less than $1/50$ of the distance between these lines. The longitudinal spacing of rivets, bolts 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 and half 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 if any, 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 shall be finished in accordance with the following requirements:

1. Rolled steel bearing plates, 2 inches or less in thickness, may be used without planing, 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 planing for all bearing surfaces (except as noted under requirement 3 of this Section), to obtain a satisfactory contact bearing; rolled steel bearing plates over 4 inches in thickness shall be planed for all bearing surfaces (except as noted under requirement 3 of this Section).
2. Column bases other than rolled steel bearing plates shall be planed for all bearing surfaces (except as noted under requirement 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 planed.

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 Straightening Material

Rolled material, before being laid off or worked, must be straight within the tolerances allowed by ASTM Specification A6. If straightening is necessary, it may be done by mechanical means or by the application of a limited amount of localized heat. 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 Oxygen Cutting

Oxygen cutting shall preferably be done by machine. Oxygen cut edges which will be subjected to substantial stress or which are to have weld metal deposited on them shall be reasonably free from gouges; occasional notches or gouges not more than $\frac{3}{16}$-inch deep will be permitted. Gouges greater than $\frac{3}{16}$-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 $\frac{1}{2}$-inch.

1.23.3 Planing of Edges

Planing or finishing of sheared or gas 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

Holes for rivets or bolts shall be $\frac{1}{16}$-inch larger than the nominal diameter of the rivet or bolt. If the thickness of the material is not greater than the nominal diameter of the rivet or bolt plus $\frac{1}{8}$-inch, the holes may be punched. If the thickness of the material is greater than the nominal diameter of the rivet or bolt plus $\frac{1}{8}$-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 $\frac{1}{16}$-inch smaller than the nominal diameter of the rivet or bolt. Holes in A514 steel plates over $\frac{1}{2}$-inch thick shall be drilled.

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. Drifting done 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.

Rivets may be driven cold if approved measures are taken to prevent distortion of the riveted material. The requirements for hot-driven rivets shall apply except as modified in the *Tentative Specifications for Cold-Driven Rivets* of the Industrial Fasteners Institute.

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 joints shall be free of oil, paint, lacquer or galvanizing.

All A325, A449, 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

TABLE 1.23.5

Bolt Size, Inches	Minimum Bolt Tension,[1] Kips	
	A325 and A449 Bolts	A490 Bolts
½	12	15
⅝	19	24
¾	28	35
⅞	39	49
1	51	64
1⅛	56	80
1¼	71	102
1⅜	85	121
1½	103	148
Over 1½		0.7 × T.S.

[1] Equal to 70 percent of specified minimum tensile strengths of bolts, rounded off to the nearest kip.

turn-of-nut method* or with 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 ksi and a hardened washer is required under the head of A449 bolts used in lieu of A325 bolts.

1.23.6 Welded Construction

Surfaces to be welded shall be free from loose scale, slag, rust, grease, paint and any other foreign material except that mill scale which withstands vigorous wire brushing may remain. Joint surfaces shall be free from fins and tears. Preparation of edges by gas cutting shall, wherever practicable, be done by a mechanically guided torch.

Parts to be fillet welded shall be brought in as close contact as practicable and in no event shall be separated by more than $3/16$-inch. If the separation is $1/16$-inch or greater, the size of the fillet welds shall be increased by the amount of the separation. The separation between faying surfaces of lap joints and butt joints on a backing structure shall not exceed $1/16$-inch. The fit of joints at contact surfaces which are not completely sealed by welds, shall be close enough to exclude water after painting.

* See Commentary, Sect. 1.23.5.

Abutting parts to be butt welded shall be carefully aligned. Misalignments greater than 1/8-inch shall be corrected and, in making the correction, the parts shall not be drawn into a sharper slope than 2 degrees (7/16-inch in 12 inches).

The work shall be positioned for flat welding whenever practicable.

In assembling and joining parts of a structure or of built-up members, the procedure and sequence of welding shall be such as will avoid needless distortion and minimize shrinkage stresses. Where it is impossible to avoid high residual stresses in the closing welds of a rigid assembly, such closing welds shall be made in compression elements.

In the fabrication of cover-plated beams and built-up members, all shop splices in each component part shall be made before such component part is welded to other parts of the member. Long girders or girder sections may be made by shop splicing not more than three subsections, each made in accordance with this paragraph.

All complete penetration groove welds made by manual welding, except when produced with the aid of backing material or welded in the flat position from both sides in square-edge material not more than 5/16-inch thick with root opening not less than one-half the thickness of the thinner part joined, shall have the root of the initial layer gouged out on the back side before welding is started from that side, and shall be so welded as to secure sound metal and complete fusion throughout the entire cross-section. Oxygen gouging shall not be permitted on ASTM A514 steel; all carbon deposits shall be removed by grinding after arc gouging A514 steel. Groove welds made with use of a backing of the same material as the base metal shall have the weld metal thoroughly fused with the backing material. Backing strips need not be removed. If required, they may be removed by gouging or gas cutting after welding is completed, provided no injury is done to the base metal and weld metal and the weld metal surface is left flush or slightly convex with full throat thickness.

Groove welds shall be terminated at the ends of a joint in a manner that will ensure their soundness. Where possible, this should be done by use of extension bars or run-off plates. Extension bars or run-off plates, if used, shall be removed upon completion of the weld and the ends of the weld made smooth and flush with the abutting parts.

Base metal shall be preheated as required to the temperature called for in Table 1.23.6 prior to welding, except tack welding which is to be remelted and incorporated into continuous submerged-arc welds. When base metal not otherwise required to be preheated is at a temperature below 32° F, it shall be preheated to at least 70° F prior to tack welding or welding. Preheating shall bring the surface of the base metal within 3 inches of the point of welding to the specified preheat temperature, and this temperature shall be maintained as a minimum interpass temperature while welding is in progress. Minimum preheat and interpass temperatures shall be as specified in Table 1.23.6. Heat input for the welding of ASTM A514 steel should not exceed the steel producer's recommendations or suggestions.

Where required, intermediate layers of multiple-layer welds may be peened with light blows from a power hammer, using a round-nose tool. Peening shall be done after the weld has cooled to a temperature warm to the hand. Care shall be exercised to prevent scaling, or flaking of weld and base metal from over-peening.

TABLE 1.23.6
Minimum Preheat and Interpass Temperature, °F[1]

Thickness of Thickest Part at Point of Welding (inches)	Welding Process				
	Shielded Metal-Arc Welding with other than Low Hydrogen Electrodes	Shielded Metal-Arc Welding with Low Hydrogen Electrodes; Submerged Arc Welding; Gas Metal-Arc Welding; or Flux Cored Arc Welding		Shielded Metal-Arc Welding with Low Hydrogen Electrodes; Submerged Arc Welding with Carbon or Alloy Steel Wire, Neutral Flux; Gas Metal-Arc Welding; or Flux Cored Arc Welding	Submerged Arc Welding with Carbon Steel Wire, Alloy Flux
	ASTM A36; A53 Grade B; A375; A500; A501; A529; A570 Grades D and E	ASTM A36; A242 Weldable Grade; A375; A441; A529; A570 Grades D & E; A572 Grades 42, 45, and 50; A588	ASTM A572 Grades 55, 60, and 65	ASTM A514	ASTM A514
To ¾, incl.	None[2,3]	None[2]	70	50	50
Over ¾ to 1½, incl.	150	70[4]	150	125	200
Over 1½ to 2½, incl.	225	150[4]	225	175	300
Over 2½	300	225	300	225	400

[1] Welding shall not be done when the ambient temperature is lower than 0° F. When the base metal is below the temperature listed for the welding process being used and the thickness of material being welded, it shall be preheated (except as otherwise provided) in such manner that the surface of the parts on which weld metal is being deposited are at or above the specified minimum temperature for a distance equal to the thickness of the part being welded, but not less than 3 in., both laterally and in advance of the welding. Preheat and interpass temperatures must be sufficient to prevent crack formation. Temperature above the minimum shown may be required for highly restrained weld. For A514 steel the maximum preheat and interpass temperature shall not exceed 400° F for thicknesses up to 1½ in., inclusive, and 450° F for greater thicknesses.

[2] When base metal temperature is below 32° F, preheat base metal to at least 70° F and maintain this minimum temperature during welding.

[3] This provision also applies to A36 steel in thicknesses up to 1 in.

[4] Minimum preheat for A36 steel in thicknesses up to 2 in. shall be 50° F.

When required by the plans or specifications, welded assemblies shall be stress relieved by heat treating in accordance with the the provisions of Article 310 of AWS D1.0-69.

The technique of welding employed, the appearance and quality of welds made, and the methods used in correcting defective work shall conform to Section 3—Workmanship and Section 4—Technique of the *Code for Welding in Building Construction,* D1.0-69, of the American Welding Society, except that the tolerance for flatness of girder webs given in Article 305 need not apply for statically loaded girders.

1.23.7 Finishing

Compression joints depending upon contact bearing shall have the bearing surfaces prepared to a common plane by milling, sawing or other suitable means.

1.23.8 Tolerances

1.23.8.1 Straightness

Structural members consisting primarily of a single rolled shape shall, unless otherwise specified, be straight within the appropriate tolerances allowed by ASTM Specification A6 or as prescribed in the following paragraph. Built-up structural members fabricated by riveting or welding, unless otherwise specified, shall be straight within the tolerances allowed for wide flange shapes by ASTM Specification A6 or by the requirements of the following paragraph.

Compression members shall not deviate from straightness by more than $1/1000$ of the axial length between points which are to be laterally supported.

Completed members shall be free from twists, bends, and open joints. Sharp kinks or bends shall be cause for rejection of material.

1.23.8.2 Length

A variation of $1/32$-inch is permissible in the overall length of members with both ends finished for contact bearing as in Sect. 1.23.7.

Members without ends finished for contact bearing, which are to be framed to other steel parts of the structure, may have a variation from the detailed length not greater than $1/16$-inch for members 30 feet or less in length, and not greater than $1/8$-inch for members over 30 feet in length.

SECTION 1.24 SHOP PAINTING

1.24.1 General Requirements

Unless otherwise specified, steelwork which will be concealed by interior building finish need not be painted; steelwork to be encased in concrete shall not be painted. Unless specifically exempted, all other steelwork shall be given one coat of shop paint, applied thoroughly and evenly to dry surfaces which have been cleaned, in accordance with the following paragraph, by brush, spray, roller coating, flow coating, or dipping, at the election of the fabricator.

After inspection and approval and before leaving the shop, all steelwork specified to be painted shall be cleaned by hand-wire brushing, or by other methods elected by the fabricator, of loose mill scale, loose rust, weld slag or flux deposit, dirt and other foreign matter. Oil and grease deposits shall be removed by solvent. Steelwork specified to have no shop paint shall, after fabrication, be cleaned of oil or grease by solvent cleaners and be cleaned of dirt and other foreign material by thorough sweeping with a fiber brush.

The shop coat of paint is intended to protect the steel for only a short period of exposure, even if it is a primer for subsequent painting to be performed in the field by others.

1.24.2 Inaccessible Surfaces

Surfaces inaccessible after assembly shall be treated in accordance with Sect. 1.24.1 before assembly.

1.24.3 Contact Surfaces

Contact surfaces shall be cleaned in accordance with Sect. 1.24.1 before assembly but shall not be painted.

1.24.4 Finished Surfaces

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

1.24.5 Surfaces Adjacent to Field Welds

Unless otherwise provided, surfaces within two inches of any field weld location shall be free of materials that would prevent proper welding or produce objectionable 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 Section 7(h) of the AISC *Code of Standard Practice*, and temporary bracing shall be introduced 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 carried 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 done until as much of the structure as will be stiffened thereby has been properly aligned.

1.25.4 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.

1.25.5 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 *Code for Welding in Building Construction,* D1.0-69, 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 High Strength Steel

Steel which is used for main components and which is required to have a yield stress greater than 36 kips per square inch shall, at all times in the fabricator's plant, be marked to identify its ASTM Specification. Identification of such steel in completed members or assemblies shall be marked by painting the ASTM Specification designation on the piece, over any shop coat of paint, prior to shipment from the fabricator's plant.

PART 2

SECTION 2.1 SCOPE

Subject to the limitations contained herein, simple or continuous beams, one and two-story rigid frames, braced multi-story 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 not be 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. 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-cap angle or standard 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

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* 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 VERTICAL BRACING SYSTEM

The vertical bracing system for a plastically designed braced multi-story 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 area of the member.

Girders and beams included in the vertical bracing system of a braced multi-story 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.

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

$$P_{cr} = 1.7AF_a \tag{2.4-1}$$

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.*

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

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

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

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* See Commentary p. 5-162.

in which

M = maximum applied moment
P = applied axial load
P_e = (23/12) AF'_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

For columns braced in the weak direction:

$$M_m = M_p$$

For columns unbraced in the weak direction:

$$M_m = \left[1.07 - \frac{(l/r_y)\sqrt{F_y}}{3{,}160}\right] M_p \leqslant M_p \qquad (2.4\text{-}4)$$

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

$$V_u \leqslant 0.55F_y td \qquad (2.5\text{-}1)$$

where V_u is the shear, in kips, that would be produced by the required factored loading, d is the depth of the member, and t is its web thickness.

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 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.

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

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

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), 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.

$$\frac{l_{cr}}{r_y} = \frac{1{,}375}{F_y} + 25 \text{ when } + 1.0 > \frac{M}{M_p} > -0.5 \qquad (2.9\text{-}1a)$$

$$\frac{l_{cr}}{r_y} = \frac{1{,}375}{F_y} \text{ when } - 0.5 \geq \frac{M}{M_p} > -1.0 \qquad (2.9\text{-}1b)$$

where

r_y = the radius of gyration of the member about its weak axis

M = the lesser of the moments at the ends of the unbraced segment

M/M_p = the end moment ratio, is 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 value 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:

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.

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

	Yield Stress — F_y (ksi)		
	36.0	42.0	45.0
SECTION 1.5 ALLOWABLE STRESSES			
1.5.1.1 Tension			
Tension on the net section, except at pin holes: $F_t = 0.60F_y \leq 0.50F_{TS}$ where F_{TS} = minimum tensile strength	22.0	25.2	27.0
Tension on the net section at pin holes in eyebars, pin-connected plates or built-up members: $F_t = 0.45F_y$	16.2	19.0	20.3
1.5.1.2 Shear			
Shear on the gross section (see Table 3 for reduced values for girder webs): $F_v = 0.40F_y$	14.5	17.0	18.0
1.5.1.3 Compression			
1.5.1.3.1 Compression on the gross section of axially loaded compression members when Kl/r is less than C_c: Formula (1.5-1) $F_a = \dfrac{\left[1 - \dfrac{(Kl/r)^2}{2C_c^2}\right]F_y}{\dfrac{5}{3} + \dfrac{3(Kl/r)}{8C_c} - \dfrac{(Kl/r)^3}{8C_c^3}}$	Table 1-36	Table 1-42	Table 1-45
1.5.1.3.2 Compression on the gross section of axially loaded compression members when Kl/r exceeds C_c: Formula (1.5-2) $F_a = \dfrac{12\pi^2 E}{23(Kl/r)^2}$	Table 1-36	Table 1-42	Table 1-45
1.5.1.3.3 Compression on the gross section of axially loaded bracing and secondary members when l/r exceeds 120: Formula (1.5-3) $F_{as} = \dfrac{F_a \text{ [by Formula (1.5-1) or (1.5-2)]}}{1.6 - \dfrac{l}{200r}}$	Table 1-36	Table 1-42	Table 1-45

* Value equal to 0.50 times minimum tensile strength ($= 0.50F_{TS}$)

Yield Stress — F_y (ksi)					
50.0	55.0	60.0	65.0	90.0	100.0
30.0	33.0	36.0	39.0	52.5*	57.5*
22.5	24.8	27.0	29.3	40.5	45.0
20.0	22.0	24.0	26.0	36.0	40.0
Table 1-50	Table 1-55	Table 1-60	Table 1-65	Table 1-90	Table 1-100
Table 1-50	Table 1-55	Table 1-60	Table 1-65	Table 1-90	Table 1-100
Table 1-50	Table 1-55	Table 1-60	Table 1-65	Table 1-90	Table 1-100

	Yield Stress — F_y (ksi)		
	36.0	42.0	45.0
1.5.1.3 Compression (cont'd)			
1.5.1.3.4 Compression on the gross area of plate girder stiffeners: $F_a = 0.60F_y$	22.0	25.2	27.0
1.5.1.3.5 Compression on the web of rolled shapes at the toe of fillet: $F_a = 0.75F_y$	27.0	31.5	33.8
1.5.1.4 Bending			
1.5.1.4.1 Tension and compression for compact, adequately braced members symmetrical about, and loaded in, the plane of their minor axis: $F_b = 0.66F_y$	24.0	28.0	29.7
when			
a. Flanges are continuously connected to web			
b. $b_f/2t_f \leq 52.2/\sqrt{F_y}$	8.7	8.1	7.8
c. $b/t_f \leq 190/\sqrt{F_y}$	31.7	29.3	28.3
d. Use Formula (1.5-4): $d/t \leq 412\left(1 - 2.33\dfrac{f_a}{F_y}\right)\Big/\sqrt{F_y}$	$68.7 - 4.4f_a$	$63.6 - 3.5f_a$	$61.4 - 3.2f_a$
except that d/t need not be less than $257/\sqrt{F_y}$	42.8	39.7	38.3
e. $l_b \leq 76.0b_f/\sqrt{F_y}$	$12.7b_f$	$11.7b_f$	$11.3b_f$
and			
$l_b \leq \dfrac{20{,}000}{(d/A_f)F_y}$	$\dfrac{556}{d/A_f}$	$\dfrac{476}{d/A_f}$	$\dfrac{444}{d/A_f}$

Yield Stress — F_y (ksi)					
50.0	55.0	60.0	65.0	90.0	100.0
30.0	33.0	36.0	39.0	54.0	60.0
37.5	41.3	45.0	48.8	67.5	75.0
33.0	36.3	39.6	42.9	—	—
7.4	7.0	6.7	6.5	—	—
26.9	25.6	24.5	23.6	—	—
$58.3 - 2.7f_a$	$55.6 - 2.4f_a$	$53.2 - 2.1f_a$	$51.1 - 1.8f_a$	—	—
36.3	34.7	33.2	31.9	—	—
$10.7b_f$	$10.2b_f$	$9.8b_f$	$9.4b_f$	—	—
$\frac{400}{d/A_f}$	$\frac{364}{d/A_f}$	$\frac{333}{d/A_f}$	$\frac{308}{d/A_f}$	—	—

	Yield Stress — F_y (ksi)		
	36.0	42.0	45.0
1.5.1.4 Bending (cont'd)			
1.5.1.4.2 Tension and compression for members which meet the requirements of Sect. 1.5.1.4.1 except subparagraph b: when $\frac{52.2}{\sqrt{F_y}} < \frac{b_f}{2t_f}$	8.7	8.1	7.8
and $\frac{b_f}{2t_f} < \frac{95.0}{\sqrt{F_y}}$	15.8	14.7	14.2
use Formula (1.5-5): $F_b = F_y \left[0.733 - 0.0014 \left(\frac{b_f}{2t_f}\right) \sqrt{F_y}\right]$			
Values of F_b — $\frac{b_f}{2t_f}$ = 7.0	—	—	—
8.0	—	—	29.6
9.0	23.7	27.3	29.2
10.0	23.4	27.0	28.8
11.0	23.1	26.6	28.4
12.0	22.8	26.2	27.9
13.0	22.5	25.8	27.5
14.0	22.1	25.5	27.1
15.0	22.0	—	—
1.5.1.4.3 Tension and compression for: doubly-symmetrical I and H shape members meeting the requirements of Sect. 1.5.1.4.1, except subparagraphs c, d and e, and bent about their minor axis (except members of A514 steel); solid round and square bars; and solid rectangular bars bent about their weaker axis: $F_b = 0.75F_y$	27.0	31.5	33.8
1.5.1.4.4 Tension and compression for box-type flexural members not included in Sect. 1.5.1.4.1, but which meet the requirements of Sect. 1.9: $F_b = 0.60F_y$	22.0	25.2	27.0
when $l_b \leq 2500b/F_y$	69.4b	59.5b	55.6b

Yield Stress — F_y (ksi)					
50.0	55.0	60.0	65.0	90.0	100.0
7.4	7.0	6.7	6.5	—	—
13.4	12.8	12.3	11.8	—	—
—	—	39.4	42.5	—	—
32.7	35.7	38.8	41.8	—	—
32.2	35.2	38.1	41.0	—	—
31.7	34.6	37.5	40.3	—	—
31.2	34.0	36.8	39.6	—	—
30.7	33.5	36.2	—	—	—
30.2	—	—	—	—	—
—	—	—	—	—	—
—	—	—	—	—	—
37.5	41.3	45.0	48.8	—	—
30.0	33.0	36.0	39.0	54.0	60.0
50.0*b*	45.5*b*	41.7*b*	38.5*b*	27.8*b*	25.0*b*

	Yield Stress — F_y (ksi)		
	36.0	42.0	45.0
1.5.1.4 Bending (cont'd)			
1.5.1.4.5 Tension for 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: $F_b = 0.60F_y$	22.0	25.2	27.0
1.5.1.4.6a Compression for flexural members included under Sect. 1.5.1.4.5, having an axis of symmetry in, and loaded in, the plane of their web; compression for channels bent about their major axis: The larger value computed by Formula (1.5-6a) or (1.5-6b) and Formula (1.5-7), but not more than $F_b = 0.60F_y$	22.0	25.2	27.0
when $l/r_T \leq \sqrt{\dfrac{102 \times 10^3 \times C_b}{F_y}}$ *	$53\sqrt{C_b}$	$49\sqrt{C_b}$	$48\sqrt{C_b}$
When this limit is exceeded, use Formula (1.5-6a): $F_b = \left[\dfrac{2}{3} - \dfrac{F_y(l/r_T)^2}{1{,}530 \times 10^3 \times C_b}\right] F_y$*	$24.0 - \dfrac{(l/r_T)^2}{1181C_b}$	$28.0 - \dfrac{(l/r_T)^2}{867C_b}$	$30.0 - \dfrac{(l/r_T)^2}{756C_b}$
unless $l/r_T \geq \sqrt{\dfrac{510 \times 10^3 \times C_b}{F_y}}$ *	$119\sqrt{C_b}$	$110\sqrt{C_b}$	$106\sqrt{C_b}$
in which case, use Formula (1.5-6b): $F_b = \dfrac{170 \times 10^3 \times C_b}{(l/r_T)^2}$ *			
When the compression flange is solid and approximately rectangular in cross-section and its area is not less than that of the tension flange, use Formula (1.5-7): $F_b = \dfrac{12 \times 10^3 \times C_b}{ld/A_f}$ *			

*For values of C_b see Fig. A1, p. 5-104.

Yield Stress — F_y (ksi)					
50.0	55.0	60.0	65.0	90.0	100.0
30.0	33.0	36.0	39.0	54.0	60.0
30.0	33.0	36.0	39.0	54.0	60.0
$45\sqrt{C_b}$	$43\sqrt{C_b}$	$41\sqrt{C_b}$	$40\sqrt{C_b}$	$34\sqrt{C_b}$	$32\sqrt{C_b}$
$33.3 - \frac{(l/r_T)^2}{612C_b}$	$36.7 - \frac{(l/r_T)^2}{506C_b}$	$40.0 - \frac{(l/r_T)^2}{425C_b}$	$43.3 - \frac{(l/r_T)^2}{362C_b}$	$60.0 - \frac{(l/r_T)^2}{189C_b}$	$66.7 - \frac{(l/r_T)^2}{153C_b}$
$101\sqrt{C_b}$	$96\sqrt{C_b}$	$92\sqrt{C_b}$	$89\sqrt{C_b}$	$75\sqrt{C_b}$	$71\sqrt{C_b}$

	Yield Stress — F_y (ksi)		
	36.0	42.0	45.0
1.5.1.4. Bending (cont'd)			
1.5.1.4.6b Compression for flexural members included under Sect. 1.5.1.4.5, which do not satisfy the requirements of Sect. 1.5.1.4.6a, and which if bent about their major axis are braced so that			
$l_b \leq (76.0 b_f / \sqrt{F_y})$	$12.7 b_f$	$11.7 b_f$	$11.3 b_f$
$F_b = 0.60 F_y$	22.0	25.2	27.0
1.5.1.5 Bearing (on contact area)			
1.5.1.5.1 Bearing on milled surfaces, including bearing stiffeners and pins in reamed, drilled, or bored holes:			
$F_p = 0.90 F_y$	33.0	38.0	40.5
1.5.1.5.2 Bearing on expansion rollers and rockers:			
$F_p = \left(\frac{F_y - 13}{20}\right) 0.66d$	$0.76d$	$0.96d$	$1.06d$
1.5.2 Rivets, Bolts, and Threaded Parts			
1.5.2.2 Bearing on projected area of bolts in bearing-type connections and on rivets:			
$F_p = 1.35 F_y$	48.6	56.7	60.8
SECTION 1.9 WIDTH-THICKNESS RATIOS			
1.9.1 Unstiffened Elements Under Compression			
1.9.1.2 Maximum width-to-thickness ratios for unstiffened elements of:			
Single-angle struts; double-angle struts with separators:			
$76.0/\sqrt{F_y}$	12.7	11.7	11.3
Double-angle struts in contact; angles or plates projecting from girders, columns or other compression members, compression flanges of beams; stiffeners on plate girders:			
$95.0/\sqrt{F_y}$	15.8	14.7	14.2
Stems of tees: $127/\sqrt{F_y}$	21.2	19.6	18.9

Yield Stress — F_y (ksi)					
50.0	55.0	60.0	65.0	90.0	100.0
$10.7b_f$	$10.2b_f$	$9.8b_f$	$9.4b_f$	$8.0b_f$	$7.6b_f$
30.0	33.0	36.0	39.0	54.0	60.0
45.0	49.5	54.0	58.5	81.0	90.0
$1.22d$	$1.39d$	$1.55d$	$1.72d$	$2.54d$	$2.87d$
67.5	74.3	81.0	87.8	121.5	135.0
10.7	10.2	9.8	9.4	8.0	7.6
13.4	12.8	12.3	11.8	10.0	9.5
18.0	17.1	16.4	15.8	13.4	12.7

	Yield Stress — F_y (ksi)		
	36.0	42.0	45.0
1.9.2 Stiffened Elements Under Compression			
1.9.2.2 Maximum width-to-thickness ratios for stiffened elements of:			
Flanges of square and rectangular sections of uniform thickness: $\dfrac{238}{\sqrt{F_y}}$	39.7	36.7	35.5
Unsupported width of perforated cover plates: $\dfrac{317}{\sqrt{F_y}}$	52.8	48.9	47.3
All other uniformly compressed elements: $\dfrac{253}{\sqrt{F_y}}$	42.2	39.0	37.7
SECTION 1.10 PLATE GIRDERS AND ROLLED BEAMS			
1.10.1 Proportions			
Maximum axial force resisted by hybrid girders: $P = 0.15F_yA$ where A = gross sectional area	$5.4A$	$6.3A$	$6.8A$
1.10.2 Web			
Maximum clear distance between flanges: $\dfrac{14{,}000}{\sqrt{F_y(F_y + 16.5)}}\,t$	$322t$	$282t$	$266t$
When transverse stiffeners are spaced $1.5 \times d$ or less, the clear distance between flanges need not be less than $\dfrac{2{,}000}{\sqrt{F_y}}\,t$ where t = thickness of web F_y = yield stress of compression flange	$333t$	$309t$	$298t$

Yield Stress — F_y (ksi)					
50.0	55.0	60.0	65.0	90.0	100.0
33.7	32.1	30.7	29.5	25.1	23.8
44.8	42.7	40.9	39.3	33.4	31.7
35.8	34.1	32.7	31.4	26.7	25.3
7.5*A*	8.3*A*	9.0*A*	9.8*A*	13.5*A*	15.0*A*
243*t*	223*t*	207*t*	192*t*	143*t*	130*t*
283*t*	270*t*	258*t*	248*t*	211*t*	200*t*

		Yield Stress — F_y (ksi)		
		36.0	42.0	45.0
1.10.5 Stiffeners				
1.10.5.2 Largest average web shear, F_v, by Formula (1.10-1) or (1.10-2), as applicable		Table 3-36	Table 3-42	Table 3-45
1.10.5.4 Required gross area of intermediate stiffeners, by Formula (1.10-3)		Table 3-36	Table 3-42	Table 3-45
Intermediate stiffeners required by Formula (1.10-2) shall be connected for a total shear transfer not less than $$f_{vs} = h\sqrt{\left(\frac{F_y}{340}\right)^3}$$ where F_y = yield stress of web steel		0.034h	0.043h	0.048h
1.10.7 Combined Shear and Tension Stress				
Bending tensile stress due to moment in the plane of the web shall not exceed $0.6F_y$ nor Formula (1.10-7): $$F_b = \left(0.825 - 0.375\frac{f_v}{F_v}\right)F_y$$	f_v/F_v			
Values of F_b	0.1	22.0	25.2	27.0
	0.2	22.0	25.2	27.0
	0.3	22.0	25.2	27.0
	0.4	22.0	25.2	27.0
	0.5	22.0	25.2	27.0
	0.6	22.0	25.2	27.0
	0.7	20.3	23.6	25.3
	0.8	18.9	22.1	23.6
	0.9	17.6	20.5	21.9
	1.0	16.2	18.9	20.3
1.10.10 Web Crippling				
1.10.10.1 Bearing stiffeners are not required under interior concentrated loads when, by Formula (1.10-8) $$\frac{R}{t(N + 2k)} \leq 0.75F_y$$		27.0	31.5	33.8
or under end reactions when, by Formula (1.10-9) $$\frac{R}{t(N + k)} \leq 0.75F_y$$		27.0	31.5	33.8

Yield Stress — F_y (ksi)					
50.0	55.0	60.0	65.0	90.0	100.0
Table 3-50	Table 3-55	Table 3-60	Table 3-65	Table 3-90	Table 3-100
Table 3-50	Table 3-55	Table 3-60	Table 3-65	Table 3-90	Table 3-100
$0.056h$	$0.065h$	$0.074h$	$0.084h$	$0.136h$	$0.160h$
30.0	33.0	36.0	39.0	54.0	60.0
30.0	33.0	36.0	39.0	54.0	60.0
30.0	33.0	36.0	39.0	54 0	60.0
30.0	33.0	36.0	39.0	54.0	60.0
30.0	33.0	36.0	39.0	54.0	60.0
30.0	33.0	36.0	39.0	54.0	60.0
28.1	30.9	33.8	36.6	50.6	56.3
26.3	28.9	31.5	34.1	47.3	52.5
24.4	26.8	29.3	31.7	43.9	48.8
22.5	24.8	27.0	29.3	40.5	45.0
37.5	41.3	45.0	48.8	67.5	75.0
37.5	41.3	45.0	48.8	67.5	75.0

	Yield Stress — F_y (ksi)		
	36.0	42.0	45.0
SECTION 1.11 COMPOSITE CONSTRUCTION			
1.11.2 Design Assumptions			
1.11.2.1 Tension and compression for encased composite beams based upon the section properties of the composite section: $F_b = 0.66F_y$	24.0	28.0	29.7
Tension and compression for encased composite beams based upon the section properties of the steel beam alone: $F_b = 0.76F_y$	27.4	31.9	34.2
SECTION 1.13 DEFLECTIONS, VIBRATION AND PONDING			
1.13.3 Ponding			
Total bending stress due to dead loads, gravity live loads (if any) and ponding, for primary and secondary members: $F_b = 0.80F_y$	28.8	33.6	36.0
SECTION 1.18 BUILT-UP MEMBERS			
1.18.2 Compression Members			
1.18.2.3 Maximum longitudinal spacing for intermediate rivets, bolts or intermittent welds in built-up members having a component consisting of an outside plate shall not exceed 12 in. nor $\frac{127}{\sqrt{F_y}}t$ where t = thickness of thinnest outside plate	21.2t	19.6t	18.9t
Maximum longitudinal spacing when rivets or bolts are staggered shall not exceed 18 in. nor $\frac{190}{\sqrt{F_y}}t$ where t = thickness of thinnest outside plate	31.7t	29.3t	28.3t

Yield Stress — F_y (ksi)					
50. 0	55.0	60.0	65.0	90.0	100.0
33.0	36.3	39.6	42.9	59.4	66.0
38.0	41.8	45.6	49.4	68.4	76.0
40.0	44.0	48.0	52.0	72.0	80.0
18.0t	17.1t	16.4t	15.8t	13.4t	12.7t
26.9t	25.6t	24.5t	23.6t	20.0t	19.0t

	Yield Stress — F_y (ksi)		
	36.0	42.0	45.0
SECTION 2.3 VERTICAL BRACING SYSTEM			
Maximum axial force due to factored gravity plus factored horizontal loads in members comprising the vertical bracing system: $P = 0.85F_yA$ where A = gross area of the member	$30.6A$	$35.7A$	$38.3A$
SECTION 2.4 COLUMNS			
The ratio of critical moment of a column without axial load and unbraced in the weak direction to the plastic moment of the column section shall not exceed 1 nor Formula (2.4-4): $\frac{M_m}{M_p} \leq \left[1.07 - \frac{(l/r_y)\sqrt{F_y}}{3,160}\right]$	$1.07 - \frac{(l/r_y)}{527}$	$1.07 - \frac{(l/r_y)}{488}$	$1.07 - \frac{(l/r_y)}{471}$
SECTION 2.5 SHEAR			
Shear in unreinforced webs of columns, beams and girders due to factored loading: Formula (2.5-1) $V_u = 0.55F_y td$ where t = thickness of web, d = depth of member	$19.8td$	$23.1td$	$24.8td$
SECTION 2.7 MINIMUM THICKNESS (WIDTH-THICKNESS RATIOS)			
Maximum width-thickness ratios of similarly compressed flange plates in box sections and cover plates: $\frac{b}{t} \leq \frac{190}{\sqrt{F_y}}$	31.7	29.3	28.3
Maximum depth-thickness ratios of webs of members subjected to plastic bending without axial load: When $P/P_y \leq 0.27$, use Formula (2.7-1a): $\frac{d}{t} \leq \frac{412}{\sqrt{F_y}}\left(1 - 1.4\frac{P}{P_y}\right)$	$68.7 - 96.1\frac{P}{P_y}$	$63.6 - 89.0\frac{P}{P_y}$	$61.4 - 86.0\frac{P}{P_y}$
When $P/P_y > 0.27$, use Formula (2.7-1b): $\frac{d}{t} \leq \frac{257}{\sqrt{F_y}}$	42.8	39.6	38.3

Yield Stress — F_y (ksi)					
50.0	55.0	60.0	65.0	90.0	100.0
$42.5A$	$46.8A$	$51.0A$	$55.3A$	—	—
$1.07 - \frac{(l/r_y)}{447}$	$1.07 - \frac{(l/r_y)}{426}$	$1.07 - \frac{(l/r_y)}{408}$	$1.07 - \frac{(l/r_y)}{392}$	—	—
$27.5td$	$30.3td$	$33.0td$	$35.8td$	—	—
26.9	25.6	24.5	23.6	—	—
$58.3 - 81.6\frac{P}{P_y}$	$55.6 - 77.8\frac{P}{P_y}$	$53.2 - 74.5\frac{P}{P_y}$	$51.1 - 71.5\frac{P}{P_y}$	—	—
36.3	34.7	33.2	31.9	—	—

	Yield Stress — F_y (ksi)		
	36.0	42.0	45.0
SECTION 2.9 LATERAL BRACING			
Maximum critical slenderness ratio, l_{cr}/r_y, from braced hinge locations to similarly braced adjacent points on a beam or frame:			
When $1.0 > \frac{M}{M_p} > -0.5$, use Formula (2.9-1a): $\frac{l_{cr}}{r_y} \leq \frac{1{,}375}{F_y} + 25$	63.2	57.7	55.6
When $-0.5 > \frac{M}{M_p} > -1.0$, use Formula (2.9-1b): $\frac{l_{cr}}{r_y} \leq \frac{1{,}375}{F_y}$	38.2	32.7	30.6

Yield Stress — F_y (ksi)					
50.0	55.0	60.0	65.0	90.0	100.0
52.5	50.0	47.9	46.2	—	—
27.5	25.0	22.9	21.2	—	—

$F_y = 36$ ksi

TABLE 1-36

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

Main and Secondary Members Kl/r not over 120						Main Members Kl/r 121 to 200				Secondary Members* 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

* K taken as 1.0 for secondary members.

Note: $C_c = 126.1$

TABLE 1-42

ALLOWABLE STRESS (KSI) FOR COMPRESSION MEMBERS OF 42 KSI SPECIFIED YIELD STRESS STEEL

$F_y = 42$ ksi

Main and Secondary Members Kl/r not over 120						Main Members Kl/r 121 to 200				Secondary Members* 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	25.15	41	21.98	81	16.92	121	10.20	161	5.76	121	10.25	161	7.25
2	25.10	42	21.87	82	16.77	122	10.03	162	5.69	122	10.13	162	7.20
3	25.05	43	21.77	83	16.62	123	9.87	163	5.62	123	10.02	163	7.16
4	24.99	44	21.66	84	16.47	124	9.71	164	5.55	124	9.91	164	7.12
5	24.94	45	21.55	85	16.32	125	9.56	165	5.49	125	9.80	165	7.08
6	24.88	46	21.44	86	16.17	126	9.41	166	5.42	126	9.70	166	7.04
7	24.82	47	21.33	87	16.01	127	9.26	167	5.35	127	9.59	167	7.00
8	24.76	48	21.22	88	15.86	128	9.11	168	5.29	128	9.49	168	6.96
9	24.70	49	21.10	89	15.71	129	8.97	169	5.23	129	9.40	169	6.93
10	24.63	50	20.99	90	15.55	130	8.84	170	5.17	130	9.30	170	6.89
11	24.57	51	20.87	91	15.39	131	8.70	171	5.11	131	9.21	171	6.85
12	24.50	52	20.76	92	15.23	132	8.57	172	5.05	132	9.12	172	6.82
13	24.43	53	20.64	93	15.07	133	8.44	173	4.99	133	9.03	173	6.79
14	24.36	54	20.52	94	14.91	134	8.32	174	4.93	134	8.94	174	6.76
15	24.29	55	20.40	95	14.75	135	8.19	175	4.88	135	8.86	175	6.73
16	24.2	56	20.28	96	14.59	136	8.07	176	4.82	136	8.78	176	6.70
17	24.15	57	20.16	97	14.43	137	7.96	177	4.77	137	8.70	177	6.67
18	24.07	58	20.03	98	14.26	138	7.84	178	4.71	138	8.62	178	6.64
19	24.00	59	19.91	99	14.09	139	7.73	179	4.66	139	8.54	179	6.61
20	23.92	60	19.79	100	13.93	140	7.62	180	4.61	140	8.47	180	6.58
21	23.84	61	19.66	101	13.76	141	7.51	181	4.56	141	8.39	181	6.56
22	23.76	62	19.53	102	13.59	142	7.41	182	4.51	142	8.32	182	6.53
23	23.68	63	19.40	103	13.42	143	7.30	183	4.46	143	8.25	183	6.51
24	23.59	64	19.27	104	13.25	144	7.20	184	4.41	144	8.18	184	6.49
25	23.51	65	19.14	105	13.08	145	7.10	185	4.36	145	8.12	185	6.46
26	23.42	66	19.01	106	12.90	146	7.01	186	4.32	146	8.05	186	6.44
27	23.33	67	18.88	107	12.73	147	6.91	187	4.27	147	7.99	187	6.42
28	23.24	68	18.75	108	12.55	148	6.82	188	4.23	148	7.93	188	6.40
29	23.15	69	18.61	109	12.37	149	6.73	189	4.18	149	7.87	189	6.38
30	23.06	70	18.48	110	12.19	150	6.64	190	4.14	150	7.81	190	6.36
31	22.97	71	18.34	111	12.01	151	6.55	191	4.09	151	7.75	191	6.35
32	22.88	72	18.20	112	11.83	152	6.46	192	4.05	152	7.69	192	6.33
33	22.78	73	18.06	113	11.65	153	6.38	193	4.01	153	7.64	193	6.31
34	22.69	74	17.92	114	11.47	154	6.30	194	3.97	154	7.59	194	6.30
35	22.59	75	17.78	115	11.28	155	6.22	195	3.93	155	7.53	195	6.28
36	22.49	76	17.64	116	11.10	156	6.14	196	3.89	156	7.48	196	6.27
37	22.39	77	17.50	117	10.91	157	6.06	197	3.85	157	7.43	197	6.26
38	22.29	78	17.35	118	10.72	158	5.98	198	3.81	158	7.39	198	6.24
39	22.19	79	17.21	119	10.55	159	5.91	199	3.77	159	7.34	199	6.23
40	22.08	80	17.06	120	10.37	160	5.83	200	3.73	160	7.29	200	6.22

* K taken as 1.0 for secondary members.

Note: $C_c = 116.7$

$F_y = 45$ ksi

TABLE 1-45

ALLOWABLE STRESS (KSI) FOR COMPRESSION MEMBERS OF 45 KSI SPECIFIED YIELD STRESS STEEL

Main and Secondary Members Kl/r not over 120						Main Members Kl/r 121 to 200				Secondary Members* 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	26.95	41	23.39	81	17.67	121	10.20	161	5.76	121	10.25	161	7.25
2	26.89	42	23.27	82	17.51	122	10.03	162	5.69	122	10.13	162	7.20
3	26.83	43	23.15	83	17.34	123	9.87	163	5.62	123	10.02	163	7.16
4	26.77	44	23.03	84	17.17	124	9.71	164	5.55	124	9.91	164	7.12
5	26.71	45	22.90	85	17.00	125	9.56	165	5.49	125	9.80	165	7.08
6	26.64	46	22.78	86	16.82	126	9.41	166	5.42	126	9.70	166	7.04
7	26.58	47	22.65	87	16.65	127	9.26	167	5.35	127	9.59	167	7.00
8	26.51	48	22.53	88	16.48	128	9.11	168	5.29	128	9.49	168	6.96
9	26.44	49	22.40	89	16.30	129	8.97	169	5.23	129	9.40	169	6.93
10	26.37	50	22.27	90	16.12	130	8.84	170	5.17	130	9.30	170	6.89
11	26.30	51	22.14	91	15.95	131	8.70	171	5.11	131	9.21	171	6.85
12	26.22	52	22.01	92	15.77	132	8.57	172	5.05	132	9.12	172	6.82
13	26.15	53	21.88	93	15.59	133	8.44	173	4.99	133	9.03	173	6.79
14	26.07	54	21.74	94	15.40	134	8.32	174	4.93	134	8.94	174	6.76
15	25.99	55	21.61	95	15.22	135	8.19	175	4.88	135	8.86	175	6.73
16	25.91	56	21.47	96	15.04	136	8.07	176	4.82	136	8.78	176	6.70
17	25.82	57	21.33	97	14.85	137	7.96	177	4.77	137	8.70	177	6.67
18	25.74	58	21.19	98	14.66	138	7.84	178	4.71	138	8.62	178	6.64
19	25.65	59	21.05	99	14.47	139	7.73	179	4.66	139	8.54	179	6.61
20	25.57	60	20.91	100	14.28	140	7.62	180	4.61	140	8.47	180	6.58
21	25.48	61	20.77	101	14.09	141	7.51	181	4.56	141	8.39	181	6.56
22	25.39	62	20.63	102	13.90	142	7.41	182	4.51	142	8.32	182	6.53
23	25.29	63	20.48	103	13.71	143	7.30	183	4.46	143	8.25	183	6.51
24	25.20	64	20.34	104	13.51	144	7.20	184	4.41	144	8.18	184	6.49
25	25.11	65	20.19	105	13.32	145	7.10	185	4.36	145	8.12	185	6.46
26	25.01	66	20.04	106	13.12	146	7.01	186	4.32	146	8.05	186	6.44
27	24.91	67	19.89	107	12.92	147	6.91	187	4.27	147	7.99	187	6.42
28	24.81	68	19.74	108	12.72	148	6.82	188	4.23	148	7.93	188	6.40
29	24.71	69	19.59	109	12.52	149	6.73	189	4.18	149	7.87	189	6.38
30	24.61	70	19.43	110	12.31	150	6.64	190	4.14	150	7.81	190	6.36
31	24.50	71	19.28	111	12.11	151	6.55	191	4.09	151	7.75	191	6.35
32	24.40	72	19.12	112	11.90	152	6.46	192	4.05	152	7.69	192	6.33
33	24.29	73	18.97	113	11.69	153	6.38	193	4.01	153	7.64	193	6.31
34	24.18	74	18.81	114	11.49	154	6.30	194	3.97	154	7.59	194	6.30
35	24.07	75	18.65	115	11.29	155	6.22	195	3.93	155	7.53	195	6.28
36	23.96	76	18.49	116	11.10	156	6.14	196	3.89	156	7.48	196	6.27
37	23.85	77	18.33	117	10.91	157	6.06	197	3.85	157	7.43	197	6.26
38	23.74	78	18.17	118	10.72	158	5.98	198	3.81	158	7.39	198	6.24
39	23.62	79	18.00	119	10.55	159	5.91	199	3.77	159	7.34	199	6.23
40	23.51	80	17.84	120	10.37	160	5.83	200	3.73	160	7.29	200	6.22

* K taken as 1.0 for secondary members.

Note: $C_c = 112.8$

TABLE 1-50

ALLOWABLE STRESS (KSI) FOR COMPRESSION MEMBERS OF 50 KSI SPECIFIED YIELD STRESS STEEL

Main and Secondary Members Kl/r not over 120						Main Members Kl/r 121 to 200				Secondary Members* 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

* K taken as 1.0 for secondary members.

Note: C_c = 107.0

F_y = 50 ksi

F_y = 55 ksi

TABLE 1-55

ALLOWABLE STRESS (KSI) FOR COMPRESSION MEMBERS OF 55 KSI SPECIFIED YIELD STRESS STEEL

Main and Secondary Members Kl/r not over 120						Main Members Kl/r 121 to 200				Secondary Members* 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	32.93	41	27.94	81	19.80	121	10.20	161	5.76	121	10.25	161	7.25
2	32.85	42	27.78	82	19.56	122	10.03	162	5.69	122	10.13	162	7.20
3	32.77	43	27.61	83	19.32	123	9.87	163	5.62	123	10.02	163	7.16
4	32.69	44	27.43	84	19.08	124	9.71	164	5.55	124	9.91	164	7.12
5	32.60	45	27.26	85	18.83	125	9.56	165	5.49	125	9.80	165	7.08
6	32.51	46	27.08	86	18.58	126	9.41	166	5.42	126	9.70	166	7.04
7	32.42	47	26.91	87	18.34	127	9.26	167	5.35	127	9.59	167	7.00
8	32.33	48	26.73	88	18.08	128	9.11	168	5.29	128	9.49	168	6.96
9	32.23	49	26.55	89	17.83	129	8.97	169	5.23	129	9.40	169	6.93
10	32.14	50	26.36	90	17.58	130	8.84	170	5.17	130	9.30	170	6.89
11	32.03	51	26.18	91	17.32	131	8.70	171	5.11	131	9.21	171	6.85
12	31.93	52	25.99	92	17.06	132	8.57	172	5.05	132	9.12	172	6.82
13	31.82	53	25.80	93	16.80	133	8.44	173	4.99	133	9.03	173	6.79
14	31.72	54	25.61	94	16.53	134	8.32	174	4.93	134	8.94	174	6.76
15	31.61	55	25.42	95	16.27	135	8.19	175	4.88	135	8.86	175	6.73
16	31.49	56	25.23	96	16.00	136	8.07	176	4.82	136	8.78	176	6.70
17	31.38	57	25.03	97	15.73	137	7.96	177	4.77	137	8.70	177	6.67
18	31.26	58	24.83	98	15.46	138	7.84	178	4.71	138	8.62	178	6.64
19	31.14	59	24.63	99	15.19	139	7.73	179	4.66	139	8.54	179	6.61
20	31.02	60	24.43	100	14.91	140	7.62	180	4.61	140	8.47	180	6.58
21	30.89	61	24.23	101	14.63	141	7.51	181	4.56	141	8.39	181	6.56
22	30.76	62	24.03	102	14.35	142	7.41	182	4.51	142	8.32	182	6.53
23	30.63	63	23.82	103	14.08	143	7.30	183	4.46	143	8.25	183	6.51
24	30.50	64	23.61	104	13.81	144	7.20	184	4.41	144	8.18	184	6.49
25	30.37	65	23.40	105	13.54	145	7.10	185	4.36	145	8.12	185	6.46
26	30.23	66	23.19	106	13.29	146	7.01	186	4.32	146	8.05	186	6.44
27	30.09	67	22.98	107	13.04	147	6.91	187	4.27	147	7.99	187	6.42
28	29.95	68	22.76	108	12.80	148	6.82	188	4.23	148	7.93	188	6.40
29	29.81	69	22.54	109	12.57	149	6.73	189	4.18	149	7.87	189	6.38
30	29.67	70	22.33	110	12.34	150	6.64	190	4.14	150	7.81	190	6.36
31	29.52	71	22.11	111	12.12	151	6.55	191	4.09	151	7.75	191	6.35
32	29.37	72	21.88	112	11.90	152	6.46	192	4.05	152	7.69	192	6.33
33	29.22	73	21.66	113	11.69	153	6.38	193	4.01	153	7.64	193	6.31
34	29.07	74	21.43	114	11.49	154	6.30	194	3.97	154	7.59	194	6.30
35	28.91	75	21.21	115	11.29	155	6.22	195	3.93	155	7.53	195	6.28
36	28.76	76	20.98	116	11.10	156	6.14	196	3.89	156	7.48	196	6.27
37	28.60	77	20.75	117	10.91	157	6.06	197	3.85	157	7.43	197	6.26
38	28.44	78	20.51	118	10.72	158	5.98	198	3.81	158	7.39	198	6.24
39	28.28	79	20.28	119	10.55	159	5.91	199	3.77	159	7.34	199	6.23
40	28.11	80	20.04	120	10.37	160	5.83	200	3.73	160	7.29	200	6.22

* K taken as 1.0 for secondary members.

Note: C_c = 102.0

TABLE 1-60

ALLOWABLE STRESS (KSI)
FOR COMPRESSION MEMBERS OF 60 KSI SPECIFIED YIELD STRESS STEEL

$F_y = 60$ ksi

Main and Secondary Members Kl/r not over 120						Main Members Kl/r 121 to 200				Secondary Members* 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	35.92	41	30.15	81	20.65	121	10.20	161	5.76	121	10.25	161	7.25
2	35.83	42	29.95	82	20.37	122	10.03	162	5.69	122	10.13	162	7.20
3	35.74	43	29.75	83	20.09	123	9.87	163	5.62	123	10.02	163	7.16
4	35.64	44	29.55	84	19.80	124	9.71	164	5.55	124	9.91	164	7.12
5	35.54	45	29.35	85	19.51	125	9.56	165	5.49	125	9.80	165	7.08
6	35.44	46	29.15	86	19.22	126	9.41	166	5.42	126	9.70	166	7.04
7	35.34	47	28.94	87	18.93	127	9.26	167	5.35	127	9.59	167	7.00
8	35.23	48	28.73	88	18.63	128	9.11	168	5.29	128	9.49	168	6.96
9	35.12	49	28.52	89	18.34	129	8.97	169	5.23	129	9.40	169	6.93
10	35.01	50	28.31	90	18.04	130	8.84	170	5.17	130	9.30	170	6.89
11	34.89	51	28.09	91	17.73	131	8.70	171	5.11	131	9.21	171	6.85
12	34.77	52	27.87	92	17.43	132	8.57	172	5.05	132	9.12	172	6.82
13	34.65	53	27.66	93	17.12	133	8.44	173	4.99	133	9.03	173	6.79
14	34.52	54	27.43	94	16.81	134	8.32	174	4.93	134	8.94	174	6.76
15	34.40	55	27.21	95	16.50	135	8.19	175	4.88	135	8.86	175	6.73
16	34.27	56	26.98	96	16.19	136	8.07	176	4.82	136	8.78	176	6.70
17	34.13	57	26.76	97	15.87	137	7.96	177	4.77	137	8.70	177	6.67
18	34.00	58	26.53	98	15.55	138	7.84	178	4.71	138	8.62	178	6.64
19	33.86	59	26.29	99	15.24	139	7.73	179	4.66	139	8.54	179	6.61
20	33.71	60	26.06	100	14.93	140	7.62	180	4.61	140	8.47	180	6.58
21	33.57	61	25.82	101	14.64	141	7.51	181	4.56	141	8.39	181	6.56
22	33.42	62	25.58	102	14.35	142	7.41	182	4.51	142	8.32	182	6.53
23	33.27	63	25.34	103	14.08	143	7.30	183	4.46	143	8.25	183	6.51
24	33.12	64	25.10	104	13.81	144	7.20	184	4.41	144	8.18	184	6.49
25	32.96	65	24.86	105	13.54	145	7.10	185	4.36	145	8.12	185	6.46
26	32.81	66	24.61	106	13.29	146	7.01	186	4.32	146	8.05	186	6.44
27	32.65	67	24.36	107	13.04	147	6.91	187	4.27	147	7.99	187	6.42
28	32.48	68	24.11	108	12.80	148	6.82	188	4.23	148	7.93	188	6.40
29	32.32	69	23.86	109	12.57	149	6.73	189	4.18	149	7.87	189	6.38
30	32.15	70	23.60	110	12.34	150	6.64	190	4.14	150	7.81	190	6.36
31	31.98	71	23.34	111	12.12	151	6.55	191	4.09	151	7.75	191	6.35
32	31.81	72	23.08	112	11.90	152	6.46	192	4.05	152	7.69	192	6.33
33	31.63	73	22.82	113	11.69	153	6.38	193	4.01	153	7.64	193	6.31
34	31.45	74	22.56	114	11.49	154	6.30	194	3.97	154	7.59	194	6.30
35	31.28	75	22.29	115	11.29	155	6.22	195	3.93	155	7.53	195	6.28
36	31.09	76	22.02	116	11.10	156	6.14	196	3.89	156	7.48	196	6.27
37	30.91	77	21.75	117	10.91	157	6.06	197	3.85	157	7.43	197	6.26
38	30.72	78	21.48	118	10.72	158	5.98	198	3.81	158	7.39	198	6.24
39	30.53	79	21.21	119	10.55	159	5.91	199	3.77	159	7.34	199	6.23
40	30.34	80	20.93	120	10.37	160	5.83	200	3.73	160	7.29	200	6.22

* K taken as 1.0 for secondary members.

Note: $C_c = 97.7$

$F_y = 65$ ksi

TABLE 1-65

ALLOWABLE STRESS (KSI) FOR COMPRESSION MEMBERS OF 65 KSI SPECIFIED YIELD STRESS STEEL

Main and Secondary Members Kl/r not over 120						Main Members Kl/r 121 to 200				Secondary Members* 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	38.90	41	32.30	81	21.36	121	10.20	161	5.76	121	10.25	161	7.25
2	38.81	42	32.08	82	21.03	122	10.03	162	5.69	122	10.13	162	7.20
3	38.70	43	31.85	83	20.70	123	9.87	163	5.62	123	10.02	163	7.16
4	38.59	44	31.62	84	20.37	124	9.71	164	5.55	124	9.91	164	7.12
5	38.48	45	31.39	85	20.04	125	9.56	165	5.49	125	9.80	165	7.08
6	38.37	46	31.35	86	19.70	126	9.41	166	5.42	126	9.70	166	7.04
7	38.25	47	30.92	87	19.36	127	9.26	167	5.35	127	9.59	167	7.00
8	38.13	48	30.68	88	19.02	128	9.11	168	5.29	128	9.49	168	6.96
9	38.00	49	30.43	89	18.67	129	8.97	169	5.23	129	9.40	169	6.93
10	37.87	50	30.19	90	18.32	130	8.84	170	5.17	130	9.30	170	6.89
11	37.74	51	29.94	91	17.97	131	8.70	171	5.11	131	9.21	171	6.85
12	37.61	52	29.69	92	17.62	132	8.57	172	5.05	132	9.12	172	6.82
13	37.47	53	29.44	93	17.26	133	8.44	173	4.99	133	9.03	173	6.79
14	37.32	54	29.18	94	16.90	134	8.32	174	4.93	134	8.94	174	6.76
15	37.18	55	28.92	95	16.55	135	8.19	175	4.88	135	8.86	175	6.73
16	37.03	56	28.66	96	16.20	136	8.07	176	4.82	136	8.78	176	6.70
17	36.87	57	28.40	97	15.87	137	7.96	177	4.77	137	8.70	177	6.67
18	36.72	58	28.14	98	15.55	138	7.84	178	4.71	138	8.62	178	6.64
19	36.56	59	27.87	99	15.24	139	7.73	179	4.66	139	8.54	179	6.61
20	36.40	60	27.60	100	14.93	140	7.62	180	4.61	140	8.47	180	6.58
21	36.23	61	27.33	101	14.64	141	7.51	181	4.56	141	8.39	181	6.56
22	36.06	62	27.05	102	14.35	142	7.41	182	4.51	142	8.32	182	6.53
23	35.89	63	26.78	103	14.08	143	7.30	183	4.46	143	8.25	183	6.51
24	35.71	64	26.50	104	13.81	144	7.20	184	4.41	144	8.18	184	6.49
25	35.54	65	26.21	105	13.54	145	7.10	185	4.36	145	8.12	185	6.46
26	35.36	66	25.93	106	13.29	146	7.01	186	4.32	146	8.05	186	6.44
27	35.17	67	25.64	107	13.04	147	6.91	187	4.27	147	7.99	187	6.42
28	34.99	68	25.35	108	12.80	148	6.82	188	4.23	148	7.93	188	6.40
29	34.80	69	25.06	109	12.57	149	6.73	189	4.18	149	7.87	189	6.38
30	34.60	70	24.76	110	12.34	150	6.64	190	4.14	150	7.81	190	6.36
31	34.41	71	24.47	111	12.12	151	6.55	191	4.09	151	7.75	191	6.35
32	34.21	72	24.17	112	11.90	152	6.46	192	4.05	152	7.69	192	6.33
33	34.01	73	23.87	113	11.69	153	6.38	193	4.01	153	7.64	193	6.31
34	33.81	74	23.56	114	11.49	154	6.30	194	3.97	154	7.59	194	6.30
35	33.60	75	23.25	115	11.29	155	6.22	195	3.93	155	7.53	195	6.28
36	33.39	76	22.94	116	11.10	156	6.14	196	3.89	156	7.48	196	6.27
37	33.18	77	22.63	117	10.91	157	6.06	197	3.85	157	7.43	197	6.26
38	32.96	78	22.32	118	10.72	158	5.98	198	3.81	158	7.39	198	6.24
39	32.75	79	22.00	119	10.55	159	5.91	199	3.77	159	7.34	199	6.23
40	32.53	80	21.68	120	10.37	160	5.83	200	3.73	160	7.29	200	6.22

* K taken as 1.0 for secondary members.

Note: $C_c = 93.8$.

TABLE 1-90

ALLOWABLE STRESS (KSI)
FOR COMPRESSION MEMBERS OF 90 KSI SPECIFIED YIELD STRESS STEEL

Main and Secondary Members Kl/r not over 120						Main Members Kl/r 121 to 200				Secondary Members* 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	53.84	41	42.39	81	22.76	121	10.20	161	5.76	121	10.25	161	7.25
2	53.68	42	42.00	82	22.21	122	10.03	162	5.69	122	10.13	162	7.20
3	53.51	43	41.59	83	21.68	123	9.87	163	5.62	123	10.02	163	7.16
4	53.33	44	41.19	84	21.16	124	9.71	164	5.55	124	9.91	164	7.12
5	53.15	45	40.78	85	20.67	125	9.56	165	5.49	125	9.80	165	7.08
6	52.95	46	40.36	86	20.19	126	9.41	166	5.42	126	9.70	166	7.04
7	52.75	47	39.94	87	19.73	127	9.26	167	5.35	127	9.59	167	7.00
8	52.55	48	39.51	88	19.28	128	9.11	168	5.29	128	9.49	168	6.96
9	52.33	49	39.08	89	18.85	129	8.97	169	5.23	129	9.40	169	6.93
10	52.11	50	38.65	90	18.44	130	8.84	170	5.17	130	9.30	170	6.89
11	51.89	51	38.21	91	18.03	131	8.70	171	5.11	131	9.21	171	6.85
12	51.65	52	37.77	92	17.64	132	8.57	172	5.05	132	9.12	172	6.82
13	51.41	53	37.32	93	17.27	133	8.44	173	4.99	133	9.03	173	6.79
14	51.17	54	36.86	94	16.90	134	8.32	174	4.93	134	8.94	174	6.76
15	50.92	55	36.41	95	16.55	135	8.19	175	4.88	135	8.86	175	6.73
16	50.66	56	35.94	96	16.20	136	8.07	176	4.82	136	8.78	176	6.70
17	50.39	57	35.47	97	15.87	137	7.96	177	4.77	137	8.70	177	6.67
18	50.12	58	35.00	98	15.55	138	7.84	178	4.71	138	8.62	178	6.64
19	49.85	59	34.52	99	15.24	139	7.73	179	4.66	139	8.54	179	6.61
20	49.56	60	34.04	100	14.93	140	7.62	180	4.61	140	8.47	180	6.58
21	49.28	61	33.56	101	14.64	141	7.51	181	4.56	141	8.39	181	6.56
22	48.98	62	33.06	102	14.35	142	7.41	182	4.51	142	8.32	182	6.53
23	48.68	63	32.57	103	14.08	143	7.30	183	4.46	143	8.25	183	6.51
24	48.38	64	32.07	104	13.81	144	7.20	184	4.41	144	8.18	184	6.49
25	48.07	65	31.56	105	13.54	145	7.10	185	4.36	145	8.12	185	6.46
26	47.75	66	31.05	106	13.29	146	7.01	186	4.32	146	8.05	186	6.44
27	47.43	67	30.53	107	13.04	147	6.91	187	4.27	147	7.99	187	6.42
28	47.10	68	30.01	108	12.80	148	6.82	188	4.23	148	7.93	188	6.40
29	46.77	69	29.48	109	12.57	149	6.73	189	4.18	149	7.87	189	6.38
30	46.43	70	28.95	110	12.34	150	6.64	190	4.14	150	7.81	190	6.36
31	46.09	71	28.41	111	12.12	151	6.55	191	4.09	151	7.75	191	6.35
32	45.74	72	27.87	112	11.90	152	6.46	192	4.05	152	7.69	192	6.33
33	45.39	73	27.32	113	11.69	153	6.38	193	4.01	153	7.64	193	6.31
34	45.03	74	26.77	114	11.49	154	6.30	194	3.97	154	7.59	194	6.30
35	44.67	75	26.21	115	11.29	155	6.22	195	3.93	155	7.53	195	6.28
36	44.30	76	25.65	116	11.10	156	6.14	196	3.89	156	7.48	196	6.27
37	43.93	77	25.08	117	10.91	157	6.06	197	3.85	157	7.43	197	6.26
38	43.55	78	24.50	118	10.72	158	5.98	198	3.81	158	7.39	198	6.24
39	43.17	79	23.92	119	10.55	159	5.91	199	3.77	159	7.34	199	6.23
40	42.78	80	23.33	120	10.37	160	5.83	200	3.73	160	7.29	200	6.22

* K taken as 1.0 for secondary members.

Note: $C_c = 79.8$

$F_y = 100$ ksi

TABLE 1-100

ALLOWABLE STRESS (KSI) FOR COMPRESSION MEMBERS OF 100 KSI SPECIFIED YIELD STRESS STEEL

Main and Secondary Members Kl/r not over 120						Main Members Kl/r 121 to 200				Secondary Members* 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	59.82	41	46.12	81	22.76	121	10.20	161	5.76	121	10.25	161	7.25
2	59.62	42	45.64	82	22.21	122	10.03	162	5.69	122	10.13	162	7.20
3	59.42	43	45.16	83	21.68	123	9.87	163	5.62	123	10.02	163	7.16
4	59.21	44	44.67	84	21.16	124	9.71	164	5.55	124	9.91	164	7.12
5	58.99	45	44.17	85	20.67	125	9.56	165	5.49	125	9.80	165	7.08
6	58.76	46	43.67	86	20.19	126	9.41	166	5.42	126	9.70	166	7.04
7	58.53	47	43.17	87	19.73	127	9.26	167	5.35	127	9.59	167	7.00
8	58.28	48	42.65	88	19.28	128	9.11	168	5.29	128	9.49	168	6.96
9	58.03	49	42.14	89	18.85	129	8.97	169	5.23	129	9.40	169	6.93
10	57.77	50	41.61	90	18.44	130	8.84	170	5.17	130	9.30	170	6.89
11	57.50	51	41.08	91	18.03	131	8.70	171	5.11	131	9.21	171	6.85
12	57.22	52	40.55	92	17.64	132	8.57	172	5.05	132	9.12	172	6.82
13	56.93	53	40.00	93	17.27	133	8.44	173	4.99	133	9.03	173	6.79
14	56.64	54	39.46	94	16.90	134	8.32	174	4.93	134	8.94	174	6.76
15	56.34	55	38.90	95	16.55	135	8.19	175	4.88	135	8.86	175	6.73
16	56.03	56	38.35	96	16.20	136	8.07	176	4.82	136	8.78	176	6.70
17	55.72	57	37.78	97	15.87	137	7.96	177	4.77	137	8.70	177	6.67
18	55.39	58	37.21	98	15.55	138	7.84	178	4.71	138	8.62	178	6.64
19	55.06	59	36.63	99	15.24	139	7.73	179	4.66	139	8.54	179	6.61
20	54.72	60	36.05	100	14.93	140	7.62	180	4.61	140	8.47	180	6.58
21	54.38	61	35.46	101	14.64	141	7.51	181	4.56	141	8.39	181	6.56
22	54.03	62	34.87	102	14.35	142	7.41	182	4.51	142	8.32	182	6.53
23	53.67	63	34.26	103	14.08	143	7.30	183	4.46	143	8.25	183	6.51
24	53.30	64	33.66	104	13.81	144	7.20	184	4.41	144	8.18	184	6.49
25	52.93	65	33.04	105	13.54	145	7.10	185	4.36	145	8.12	185	6.46
26	52.55	66	32.42	106	13.29	146	7.01	186	4.32	146	8.05	186	6.44
27	52.17	67	31.80	107	13.04	147	6.91	187	4.27	147	7.99	187	6.42
28	51.78	68	31.16	108	12.80	148	6.82	188	4.23	148	7.93	188	6.40
29	51.38	69	30.52	109	12.57	149	6.73	189	4.18	149	7.87	189	6.38
30	50.97	70	29.88	110	12.34	150	6.64	190	4.14	150	7.81	190	6.36
31	50.56	71	29.22	111	12.12	151	6.55	191	4.09	151	7.75	191	6.35
32	50.15	72	28.56	112	11.90	152	6.46	192	4.05	152	7.69	192	6.33
33	49.72	73	27.90	113	11.69	153	6.38	193	4.01	153	7.64	193	6.31
34	49.29	74	27.22	114	11.49	154	6.30	194	3.97	154	7.59	194	6.30
35	48.86	75	26.54	115	11.29	155	6.22	195	3.93	155	7.53	195	6.28
36	48.42	76	25.85	116	11.10	156	6.14	196	3.89	156	7.48	196	6.27
37	47.97	77	25.19	117	10.91	157	6.06	197	3.85	157	7.43	197	6.26
38	47.51	78	24.54	118	10.72	158	5.98	198	3.81	158	7.39	198	6.24
39	47.05	79	23.93	119	10.55	159	5.91	199	3.77	159	7.34	199	6.23
40	46.59	80	23.33	120	10.37	160	5.83	200	3.73	160	7.29	200	6.22

* K taken as 1.0 for secondary members.

Note: $C_c = 75.7$

TABLE 1-A

VALUES OF C_a

For determining F_a from equation $F_a = C_a F_y$, for all grades of steel

$\frac{Kl/r}{C_c}$	C_a	$\frac{Kl/r}{C_c}$	C_a	$\frac{Kl/r}{C_c}$	C_a	$\frac{Kl/r}{C_c}$	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

Note: Use $\frac{Kl/r}{C_c'}$ in lieu of $\frac{Kl/r}{C_c}$ values when ratios exceed the limits of Sect. 1.9.

TABLE 1-B

VALUES OF C_c

For use in Formulas (1.5-1), (1.5-2), and (1.5-3), Sect. 1.5.1.3, and in Table 1-A

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

All grades of steel

TABLE 2

VALUES OF F'_e (KSI)

For use in Formula (1.6–1a), Sect. 1.6.1, for all grades of steel

$\frac{Kl_b}{r_b}$	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	5.82	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	94.1	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

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

TABLE 3-36

ALLOWABLE SHEAR STRESSES (F_v) IN PLATE GIRDERS (KSI) FOR 36 KSI SPECIFIED YIELD STRESS STEEL

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

Slenderness ratios h/t: web depth to web thickness	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*												

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

* For single angle stiffeners, multiply by 1.8; for single plate stiffeners, multiply by 2.4.

TABLE 3-42

ALLOWABLE SHEAR STRESSES (F_v) IN PLATE GIRDERS (KSI) FOR 42 KSI SPECIFIED YIELD STRESS STEEL

(*Italic* values indicate gross area, as percent of web area, required for **pairs** of intermediate stiffeners of 42 ksi yield stress steel.) *

Slenderness ratios h/t: web depth to web thickness	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														17.0
60								17.0	17.0	17.0	17.0	17.0	17.0	16.4
70						17.0	17.0	16.5	16.0	15.6	15.3	14.9	14.6	14.1
80				17.0	17.0	16.3	15.2	14.5 *0.1*	14.2 *0.5*	14.0 *0.7*	13.8 *0.8*	13.5 *0.9*	13.3 *0.9*	12.3
90			17.0	16.6	15.4	14.5 *0.1*	14.1 *1.0*	13.7 *1.5*	13.4 *1.8*	13.1 *1.9*	12.9 *1.9*	12.5 *1.9*	12.1 *1.8*	10.3
100		17.0	16.4	15.0	14.3 *0.7*	14.0 *1.5*	13.5 *2.3*	13.0 *2.7*	12.5 *3.2*	12.1 *3.3*	11.7 *3.4*	11.0 *3.2*	10.5 *2.9*	8.3
110		16.8	15.0	14.2 *1.0*	13.9 *2.0*	13.5 *2.7*	12.7 *3.9*	12.0 *4.5*	11.5 *4.7*	11.0 *4.7*	10.6 *4.6*	9.9 *4.2*	9.4 *3.8*	6.9
120	17.0	15.4	14.3 *0.9*	13.9 *2.1*	13.4 *3.2*	12.8 *4.5*	12.0 *5.5*	11.3 *5.9*	10.7 *5.9*	10.2 *5.8*	9.8 *5.6*	9.1 *5.0*	8.5 *4.4*	5.8
130	16.5	14.5 *0.3*	14.0 *1.9*	13.5 *3.3*	12.8 *4.9*	12.3 *6.0*	11.4 *6.7*	10.7 *6.9*	10.1 *6.8*	9.6 *6.6*	9.2 *6.3*	8.4 *5.6*	7.9 *4.9*	4.9
140	15.3	14.2 *1.3*	13.7 *2.9*	13.0 *4.9*	12.4 *6.3*	11.8 *7.2*	10.9 *7.7*	10.2 *7.8*	9.7 *7.6*	9.2 *7.3*	8.7 *6.9*	7.9 *6.0*	7.3 *5.3*	4.2
150	14.5 *0.2*	14.0 *2.2*	13.3 *4.3*	12.6 *6.2*	12.0 *7.4*	11.4 *8.1*	10.6 *8.5*	9.9 *8.5*	9.3 *8.2*	8.8 *7.8*	8.3 *7.4*	7.5 *6.4*	6.9 *5.6*	3.7
160	14.3 *1.1*	13.8 *3.1*	13.0 *5.6*	12.3 *7.3*	11.7 *8.3*	11.1 *8.9*	10.3 *9.2*	9.6 *9.0*	9.0 *8.6*	8.4 *8.2*	8.0 *7.8*	7.2 *6.7*		3.2
170	14.1 *1.8*	13.4 *4.4*	12.7 *6.7*	12.0 *8.1*	11.4 *9.0*	10.9 *9.6*	10.0 *9.7*	9.3 *9.5*	8.7 *9.0*	8.2 *8.6*	7.7 *8.1*			2.9
180	14.0 *2.5*	13.1 *5.5*	12.4 *7.6*	11.8 *8.9*	11.2 *9.7*	10.7 *10.1*	9.8 *10.2*	9.1 *9.9*	8.5 *9.4*	8.0 *8.9*	7.5 *8.3*			2.6
200	13.5 *4.4*	12.7 *7.2*	12.0 *9.0*	11.4 *10.0*	10.9 *10.7*	10.3 *11.0*	9.5 *10.9*	8.8 *10.5*	8.1 *9.9*					2.1
220	13.1 *6.1*	12.4 *8.5*	11.7 *10.0*	11.1 *10.9*	10.6 *11.4*	10.1 *11.6*	9.2 *11.4*	8.5 *10.9*						1.7
240	12.8 *7.3*	12.1 *9.5*	11.5 *10.8*	10.9 *11.6*	10.4 *12.0*	9.9 *12.1*								1.4
260	12.6 *8.3*	11.9 *10.2*	11.3 *11.4*	10.8 *12.1*	10.3 *12.4*	9.8 *12.5*								1.2
280	12.4 *9.0*	11.8 *10.8*	11.2 *11.9*	10.7 *12.5*										
300	12.3 *9.6*	11.7 *11.3*	11.1 *12.3*											

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

* For areas of other intermediate stiffeners, multiply *italic* values by appropriate factor below:

Stiffener Steel Grade	Pairs of Stiffeners	Single Angle Stiffeners	Single Plate Stiffeners
F_y = 42 ksi	1.0	1.8	2.4
F_y = 36 ksi	1.2	2.1	2.8

TABLE 3-45

ALLOWABLE SHEAR STRESSES (F_v) IN PLATE GIRDERS (KSI)
FOR 45 KSI SPECIFIED YIELD STRESS STEEL

(*Italic* values indicate gross area, as percent of web area, required for **pairs** of intermediate stiffeners of 45 ksi yield stress steel.) *

Slenderness ratios h/t: web depth to web thickness	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													18.0	18.0
60								18.0	18.0	18.0	18.0	18.0	17.7	17.0
70					18.0	18.0	18.0	17.1	16.6	16.2	15.9	15.5 *0.1*	15.3 *0.2*	14.6
80				18.0	17.9	16.8	15.7	15.3 *0.5*	15.0 *0.8*	14.7 *1.0*	14.5 *1.1*	14.2 *1.2*	13.9 *1.1*	12.7
90			18.0	17.2	15.9	15.3 *0.6*	14.9 *1.4*	14.5 *1.9*	14.1 *2.1*	13.8 *2.2*	13.5 *2.3*	12.8 *2.3*	12.3 *2.2*	10.3
100		18.0	17.0	15.5 *0.1*	15.1 *1.2*	14.8 *2.0*	14.2 *2.7*	13.6 *3.4*	13.0 *3.8*	12.5 *3.9*	12.1 *3.9*	11.3 *3.6*	10.8 *3.2*	8.3
110		17.4	15.5 *0.1*	15.1 *1.4*	14.7 *2.4*	14.2 *3.3*	13.3 *4.6*	12.6 *5.1*	12.0 *5.2*	11.5 *5.2*	11.0 *5.0*	10.2 *4.5*	9.7 *4.0*	6.9
120	18.0	15.9	15.2 *1.3*	14.7 *2.6*	14.1 *3.9*	13.5 *5.1*	12.6 *6.1*	11.8 *6.4*	11.2 *6.3*	10.7 *6.2*	10.2 *5.9*	9.4 *5.2*	8.8 *4.6*	5.8
130	17.1	15.4 *0.8*	14.9 *2.4*	14.2 *4.0*	13.5 *5.6*	12.9 *6.6*	12.0 *7.2*	11.2 *7.3*	10.6 *7.2*	10.1 *6.9*	9.6 *6.6*	8.8 *5.8*	8.1 *5.1*	4.9
140	15.9	15.1 *1.8*	14.5 *3.6*	13.7 *5.6*	13.0 *6.8*	12.4 *7.7*	11.5 *8.1*	10.8 *8.1*	10.1 *7.9*	9.6 *7.5*	9.1 *7.1*	8.2 *6.2*	7.6 *5.4*	4.2
150	15.4 *0.7*	14.9 *2.6*	14.1 *5.0*	13.3 *6.8*	12.7 *7.9*	12.1 *8.6*	11.2 *8.9*	10.4 *8.8*	9.8 *8.4*	9.2 *8.0*	8.7 *7.6*	7.8 *6.6*	7.2 *5.7*	3.7
160	15.2 *1.5*	14.5 *3.8*	13.7 *6.2*	13.0 *7.8*	12.3 *8.7*	11.8 *9.3*	10.9 *9.5*	10.1 *9.3*	9.4 *8.9*	8.9 *8.4*	8.4 *7.9*	7.5 *6.9*		3.2
170	15.0 *2.2*	14.2 *5.1*	13.4 *7.2*	12.7 *8.6*	12.1 *9.4*	11.5 *9.9*	10.6 *10.0*	9.8 *9.7*	9.2 *9.3*	8.6 *8.7*	8.1 *8.2*			2.9
180	14.8 *3.0*	13.9 *6.1*	13.2 *8.1*	12.5 *9.3*	11.9 *10.0*	11.3 *10.4*	10.4 *10.4*	9.6 *10.1*	9.0 *9.6*	8.4 *9.0*	7.9 *8.5*			2.6
200	14.3 *5.1*	13.5 *7.7*	12.8 *9.4*	12.1 *10.4*	11.5 *11.0*	11.0 *11.2*	10.1 *11.1*	9.3 *10.6*	8.6 *10.1*					2.1
220	13.9 *6.6*	13.2 *8.9*	12.5 *10.3*	11.8 *11.2*	11.3 *11.6*	10.7 *11.8*	9.8 *11.6*	9.0 *11.1*						1.7
240	13.6 *7.7*	12.9 *9.8*	12.3 *11.1*	11.6 *11.8*	11.1 *12.2*	10.5 *12.3*								1.4
260	13.4 *8.6*	12.7 *10.5*	12.1 *11.6*	11.5 *12.3*	10.9 *12.6*	10.4 *12.6*								1.2
280	13.2 *9.3*	12.6 *11.1*	11.9 *12.1*	11.4 *12.6*										
300	13.1 *9.9*	12.4 *11.5*	11.8 *12.5*											

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

* For areas of other intermediate stiffeners, multiply *italic* values by appropriate factor below:

Stiffener Steel Grade	Pairs of Stiffeners	Single Angle Stiffeners	Single Plate Stiffeners
F_y = 45 ksi	1.0	1.8	2.4
F_y = 36 ksi	1.3	2.3	3.0

TABLE 3-50

ALLOWABLE SHEAR STRESSES (F_v) IN PLATE GIRDERS (KSI) FOR 50 KSI SPECIFIED YIELD STRESS STEEL

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

Slenderness ratios h/t: web depth to web thickness	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*								1.2
280	14.6 *9.8*	13.9 *11.4*	13.2 *12.4*	12.5 *12.9*										

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

* For areas of other intermediate stiffeners, multiply *italic* values by appropriate factor below:

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

TABLE 3-55

ALLOWABLE SHEAR STRESSES (F_v) IN PLATE GIRDERS (KSI) FOR 55 KSI SPECIFIED YIELD STRESS STEEL

(*Italic* values indicate gross area, as percent of web area, required for **pairs** of intermediate stiffeners of 55 ksi yield stress steel.) *

Slenderness ratios h/t: web depth to web thickness	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									22.0	22.0	22.0	22.0	22.0	22.0
60						22.0	22.0	22.0	21.3	20.8	20.5	19.9	19.5	18.8
70				22.0	22.0	21.3	19.8	19.0 *0.1*	18.6 *0.5*	18.4 *0.7*	18.1 *0.8*	17.7 *0.9*	17.4 *0.9*	16.1
80			22.0	21.4	19.8	18.9 *0.3*	18.3 *1.2*	17.8 *1.7*	17.4 *1.9*	17.1 *2.0*	16.8 *2.0*	16.0 *2.1*	15.4 *2.0*	13.0
90		22.0	20.9	19.0	18.5 *1.1*	18.1 *1.9*	17.4 *2.6*	16.6 *3.3*	15.9 *3.7*	15.3 *3.8*	14.8 *3.8*	13.9 *3.5*	13.3 *3.2*	10.3
100		21.2	19.0 *0.2*	18.4 *1.5*	17.9 *2.5*	17.3 *3.5*	16.2 *4.7*	15.3 *5.2*	14.6 *5.3*	14.0 *5.2*	13.4 *5.1*	12.5 *4.6*	11.8 *4.1*	8.3
110	22.0	19.2	18.5 *1.5*	17.9 *2.7*	17.1 *4.2*	16.3 *5.4*	15.2 *6.3*	14.3 *6.5*	13.6 *6.5*	12.9 *6.3*	12.4 *6.0*	11.4 *5.3*	10.6 *4.7*	6.9
120	20.5	18.7 *1.1*	18.1 *2.6*	17.2 *4.5*	16.4 *5.9*	15.6 *6.9*	14.5 *7.5*	13.6 *7.6*	12.8 *7.4*	12.1 *7.1*	11.6 *6.8*	10.5 *5.9*	9.8 *5.2*	5.8
130	19.0 *0.1*	18.3 *2.1*	17.5 *4.2*	16.6 *6.0*	15.8 *7.3*	15.0 *8.0*	13.9 *8.4*	13.0 *8.4*	12.2 *8.1*	11.5 *7.7*	11.0 *7.3*	9.9 *6.4*	9.1 *5.5*	4.9
140	18.7 *1.1*	18.0 *3.1*	17.0 *5.6*	16.1 *7.3*	15.3 *8.3*	14.6 *8.9*	13.5 *9.2*	12.5 *9.0*	11.7 *8.7*	11.1 *8.2*	10.5 *7.8*	9.4 *6.7*	8.6 *5.8*	4.2
150	18.4 *1.9*	17.5 *4.6*	16.5 *6.8*	15.7 *8.3*	14.9 *9.2*	14.2 *9.7*	13.1 *9.8*	12.1 *9.5*	11.3 *9.1*	10.7 *8.6*	10.1 *8.1*	8.9 *7.0*	8.1 *6.1*	3.7
160	18.2 *2.7*	17.1 *5.8*	16.2 *7.8*	15.3 *9.1*	14.6 *9.9*	13.9 *10.3*	12.8 *10.3*	11.8 *10.0*	11.0 *9.5*	10.3 *8.9*	9.8 *8.4*	8.6 *7.2*		3.2
170	17.8 *3.9*	16.8 *6.8*	15.9 *8.6*	15.1 *9.8*	14.3 *10.4*	13.6 *10.8*	12.5 *10.7*	11.6 *10.3*	10.8 *9.8*	10.1 *9.2*	9.5 *8.7*			2.9
180	17.5 *5.0*	16.5 *7.6*	15.6 *9.3*	14.8 *10.3*	14.1 *10.9*	13.4 *11.2*	12.3 *11.1*	11.4 *10.6*	10.6 *10.0*	9.9 *9.4*	9.3 *8.9*			2.6
200	17.0 *6.7*	16.1 *9.0*	15.2 *10.4*	14.5 *11.2*	13.8 *11.7*	13.1 *11.8*	12.0 *11.6*	11.0 *11.1*	10.2 *10.4*					2.1
220	16.6 *7.9*	15.7 *9.9*	14.9 *11.2*	14.2 *11.9*	13.5 *12.2*	12.9 *12.3*	11.8 *12.0*	10.8 *11.4*						1.7
240	16.3 *8.8*	15.5 *10.7*	14.7 *11.8*	14.0 *12.4*	13.3 *12.7*	12.7 *12.7*								1.4
260	16.1 *9.6*	15.3 *11.2*	14.5 *12.2*	13.8 *12.8*	13.2 *13.0*	12.5 *13.0*								1.2

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

* For areas of other intermediate stiffeners, multiply *italic* values by appropriate factor below:

Stiffener Steel Grade	Pairs of Stiffeners	Single Angle Stiffeners	Single Plate Stiffeners
F_y = 55 ksi	1.0	1.8	2.4
F_y = 36 ksi	1.5	2.8	3.7

TABLE 3-60

ALLOWABLE SHEAR STRESSES (F_v) IN PLATE GIRDERS (KSI)
FOR 60 KSI SPECIFIED YIELD STRESS STEEL

(*Italic* values indicate gross area, as percent of web area, required for **pairs** of intermediate stiffeners of 60 ksi yield stress steel.) *

Slenderness ratios h/t: web depth to web thickness	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
40														24.0
50								24.0	24.0	24.0	24.0	24.0	24.0	23.5
60					24.0	24.0	24.0	23.1	22.3	21.8	21.4	20.8	20.5 *0.1*	19.6
70				24.0	23.7	22.2	20.7	20.3 *0.6*	19.9 *1.0*	19.5 *1.1*	19.3 *1.2*	18.7 *1.3*	18.4 *1.2*	16.8
80			24.0	22.4	20.7	20.3 *0.9*	19.6 *1.8*	19.0 *2.2*	18.6 *2.4*	18.0 *2.6*	17.5 *2.7*	16.6 *2.7*	15.9 *2.5*	13.0
90		24.0	21.8	20.5 *0.6*	19.9 *1.7*	19.4 *2.4*	18.5 *3.5*	17.5 *4.1*	16.7 *4.4*	16.1 *4.4*	15.5 *4.4*	14.5 *4.0*	13.8 *3.6*	10.3
100	24.0	22.1	20.4 *0.8*	19.8 *2.1*	19.3 *3.1*	18.4 *4.4*	17.2 *5.4*	16.2 *5.8*	15.4 *5.9*	14.7 *5.7*	14.1 *5.5*	13.0 *4.9*	12.2 *4.4*	8.3
110	23.3	20.6 *0.5*	19.9 *2.1*	19.2 *3.5*	18.2 *5.1*	17.4 *6.2*	16.2 *6.9*	15.2 *7.1*	14.4 *7.0*	13.7 *6.7*	13.1 *6.4*	11.9 *5.6*	11.1 *4.9*	6.9
120	21.4	20.2 *1.6*	19.4 *3.3*	18.4 *5.4*	17.5 *6.7*	16.7 *7.5*	15.4 *8.0*	14.4 *8.0*	13.6 *7.8*	12.9 *7.4*	12.3 *7.1*	11.1 *6.2*	10.2 *5.4*	5.8
130	20.5 *0.7*	19.8 *2.6*	18.7 *5.1*	17.7 *6.8*	16.9 *7.9*	16.1 *8.6*	14.9 *8.9*	13.9 *8.8*	13.0 *8.4*	12.3 *8.0*	11.6 *7.6*	10.4 *6.6*	9.6 *5.7*	4.9
140	20.2 *1.6*	19.3 *4.1*	18.2 *6.4*	17.2 *7.9*	16.4 *8.9*	15.6 *9.4*	14.4 *9.6*	13.4 *9.4*	12.5 *8.9*	11.8 *8.5*	11.2 *8.0*	9.9 *6.9*	9.0 *6.0*	4.2
150	19.9 *2.4*	18.8 *5.4*	17.8 *7.5*	16.8 *8.8*	16.0 *9.6*	15.3 *10.1*	14.0 *10.1*	13.0 *9.8*	12.1 *9.4*	11.4 *8.8*	10.8 *8.3*	9.5 *7.2*	8.6 *6.2*	3.7
160	19.6 *3.6*	18.4 *6.5*	17.4 *8.4*	16.5 *9.6*	15.7 *10.3*	15.0 *10.6*	13.7 *10.6*	12.7 *10.2*	11.8 *9.7*	11.1 *9.1*	10.4 *8.6*	9.2 *7.4*		3.2
170	19.2 *4.7*	18.1 *7.4*	17.1 *9.2*	16.2 *10.2*	15.4 *10.8*	14.7 *11.1*	13.5 *11.0*	12.5 *10.5*	11.6 *10.0*	10.8 *9.4*	10.2 *8.8*			2.9
180	18.9 *5.7*	17.8 *8.2*	16.8 *9.8*	16.0 *10.7*	15.2 *11.2*	14.5 *11.5*	13.3 *11.3*	12.2 *10.8*	11.4 *10.2*	10.6 *9.6*	9.9 *9.0*			2.6
200	18.3 *7.2*	17.3 *9.4*	16.5 *10.8*	15.6 *11.5*	14.9 *11.9*	14.2 *12.1*	13.0 *11.8*	11.9 *11.2*	11.0 *10.6*					2.1
220	18.0 *8.4*	17.0 *10.3*	16.2 *11.5*	15.4 *12.1*	14.6 *12.5*	13.9 *12.5*	12.7 *12.2*	11.7 *11.5*						1.7
240	17.7 *9.3*	16.8 *11.0*	15.9 *12.0*	15.2 *12.6*	14.4 *12.8*	13.7 *12.9*								1.4
260	17.4 *9.9*	16.6 *11.5*	15.8 *12.5*	15.0 *13.0*	14.3 *13.1*	13.6 *13.1*								1.2

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

* For areas of other intermediate stiffeners, multiply *italic* values by appropriate factor below:

Stiffener Steel Grade	Pairs of Stiffeners	Single Angle Stiffeners	Single Plate Stiffeners
F_y = 60 ksi	1.0	1.8	2.4
F_y = 36 ksi	1.7	3.0	4.0

TABLE 3-65

ALLOWABLE SHEAR STRESSES (F_v) IN PLATE GIRDERS (KSI) FOR 65 KSI SPECIFIED YIELD STRESS STEEL

(*Italic* values indicate gross area, as percent of web area, required for **pairs** of intermediate stiffeners of 65 ksi yield stress steel.) *

Slenderness ratios h/t: web depth to web thickness	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
40												26.0	26.0	26.0
50							26.0	26.0	26.0	26.0	26.0	25.9	25.5	24.5
60					26.0	26.0	25.2	24.0	23.2	22.7	22.3 *0.1*	21.9 *0.4*	21.6 *0.4*	20.4
70			26.0	26.0	24.6	23.1	22.1 *0.6*	21.5 *1.1*	21.1 *1.4*	20.7 *1.5*	20.4 *1.6*	19.8 *1.6*	19.4 *1.5*	17.0
80		26.0	25.6	23.3	22.2 *0.6*	21.6 *1.5*	20.9 *2.2*	20.3 *2.6*	19.4 *3.1*	18.7 *3.3*	18.2 *3.3*	17.1 *3.2*	16.4 *2.9*	13.0
90		25.6	22.7	21.9 *1.2*	21.3 *2.2*	20.8 *2.9*	19.4 *4.3*	18.4 *4.8*	17.5 *5.0*	16.8 *5.0*	16.2 *4.8*	15.0 *4.4*	14.2 *3.9*	10.3
100	26.0	23.0	21.9 *1.3*	21.2 *2.6*	20.4 *4.0*	19.4 *5.2*	18.1 *6.1*	17.1 *6.4*	16.2 *6.3*	15.4 *6.2*	14.8 *5.9*	13.6 *5.2*	12.7 *4.6*	8.3
110	24.3	22.1 *1.0*	21.4 *2.6*	20.4 *4.4*	19.4 *5.9*	18.5 *6.8*	17.2 *7.4*	16.1 *7.5*	15.2 *7.4*	14.4 *7.1*	13.7 *6.7*	12.5 *5.9*	11.6 *5.1*	6.9
120	22.4 *0.2*	21.6 *2.2*	20.7 *4.2*	19.6 *6.1*	18.6 *7.3*	17.7 *8.1*	16.4 *8.5*	15.3 *8.4*	14.4 *8.1*	13.6 *7.7*	12.9 *7.3*	11.6 *6.4*	10.7 *5.6*	5.8
130	22.1 *1.2*	21.2 *3.3*	20.0 *5.8*	18.9 *7.4*	18.0 *8.4*	17.1 *9.0*	15.8 *9.3*	14.7 *9.1*	13.8 *8.7*	13.0 *8.3*	12.3 *7.8*	11.0 *6.8*	10.0 *5.9*	4.9
140	21.7 *2.1*	20.6 *4.9*	19.4 *7.1*	18.4 *8.5*	17.5 *9.3*	16.7 *9.8*	15.4 *9.9*	14.3 *9.6*	13.3 *9.2*	12.5 *8.7*	11.8 *8.2*	10.5 *7.1*	9.5 *6.1*	4.2
150	21.4 *3.0*	20.1 *6.1*	19.0 *8.1*	18.0 *9.3*	17.1 *10.0*	16.3 *10.4*	15.0 *10.4*	13.9 *10.1*	12.9 *9.6*	12.1 *9.0*	11.4 *8.5*	10.1 *7.3*	9.1 *6.3*	3.7
160	20.9 *4.3*	19.7 *7.1*	18.6 *8.9*	17.7 *10.0*	16.8 *10.6*	16.0 *10.9*	14.7 *10.9*	13.6 *10.4*	12.6 *9.9*	11.8 *9.3*	11.1 *8.7*	9.7 *7.5*		3.2
170	20.5 *5.4*	19.4 *8.0*	18.3 *9.6*	17.4 *10.6*	16.6 *11.1*	15.8 *11.4*	14.5 *11.2*	13.3 *10.7*	12.4 *10.1*	11.5 *9.5*	10.8 *9.0*			2.9
180	20.2 *6.3*	19.1 *8.7*	18.1 *10.2*	17.2 *11.1*	16.3 *11.5*	15.6 *11.7*	14.3 *11.5*	13.1 *11.0*	12.2 *10.4*	11.3 *9.7*	10.6 *9.1*			2.6
200	19.7 *7.8*	18.6 *9.8*	17.7 *11.1*	16.8 *11.8*	16.0 *12.2*	15.2 *12.3*	13.9 *12.0*	12.8 *11.4*						2.1
220	19.3 *8.8*	18.3 *10.6*	17.4 *11.7*	16.5 *12.4*	15.7 *12.6*	15.0 *12.7*	13.7 *12.3*	12.5 *11.7*						1.7
240	19.0 *9.6*	18.1 *11.3*	17.2 *12.3*	16.3 *12.8*	15.5 *13.0*	14.8 *13.0*								1.4

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

* For areas of other intermediate stiffeners, multiply *italic* values by appropriate factor below:

Stiffener Steel Grade	Pairs of Stiffeners	Single Angle Stiffeners	Single Plate Stiffeners
F_y = 65 ksi	1.0	1.8	2.4
F_y = 36 ksi	1.8	3.3	4.3

TABLE 3-90

ALLOWABLE SHEAR STRESSES (F_v) IN PLATE GIRDERS (KSI) FOR 90 KSI SPECIFIED YIELD STRESS STEEL

(*Italic* values indicate gross area, as percent of web area, required for **pairs** of intermediate stiffeners of 90 ksi yield stress steel.) *

Slenderness ratios h/t: web depth to web thickness	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
40							36.0	36.0	36.0	36.0	36.0	36.0	36.0	36.0
50					36.0	36.0	35.5	33.9	32.8	32.0	31.4	30.7 *0.2*	30.3 *0.3*	28.8
60			36.0	36.0	33.8	31.8	30.5 *0.7*	29.7 *1.2*	29.1 *1.5*	28.5 *1.6*	28.1 *1.7*	27.3 *1.6*	26.7 *1.5*	23.1
70		36.0	34.4	31.3	30.4 *1.0*	29.6 *1.8*	28.6 *2.6*	27.3 *3.2*	26.2 *3.6*	25.2 *3.7*	24.4 *3.7*	22.9 *3.5*	21.9 *3.2*	17.0
80	36.0	33.8	30.8 *0.5*	29.9 *1.8*	29.1 *2.8*	27.9 *4.0*	26.1 *5.1*	24.6 *5.5*	23.4 *5.6*	22.4 *5.5*	21.5 *5.3*	19.9 *4.8*	18.8 *4.2*	13.0
90	34.9	30.9 *0.5*	29.9 *2.1*	28.8 *3.6*	27.3 *5.2*	26.1 *6.2*	24.3 *6.9*	22.8 *7.1*	21.5 *7.0*	20.5 *6.7*	19.6 *6.4*	17.8 *5.6*	16.6 *4.9*	10.3
100	31.4	30.1 *1.9*	28.9 *3.8*	27.3 *5.7*	26.0 *7.0*	24.7 *7.8*	22.9 *8.3*	21.4 *8.2*	20.1 *8.0*	19.1 *7.6*	18.1 *7.2*	16.4 *6.3*	15.1 *5.5*	8.3
110	30.6 *1.1*	29.4 *3.2*	27.7 *5.7*	26.2 *7.4*	24.9 *8.4*	23.8 *9.0*	22.0 *9.2*	20.4 *9.1*	19.1 *8.7*	18.0 *8.2*	17.1 *7.8*	15.3 *6.7*	14.0 *5.9*	6.9
120	30.0 *2.2*	28.4 *5.0*	26.8 *7.2*	25.4 *8.6*	24.2 *9.4*	23.0 *9.9*	21.2 *10.0*	19.7 *9.7*	18.4 *9.2*	17.3 *8.7*	16.3 *8.2*	14.4 *7.1*	13.1 *6.2*	5.8
130	29.4 *3.5*	27.6 *6.4*	26.1 *8.3*	24.8 *9.5*	23.6 *10.2*	22.5 *10.6*	20.7 *10.6*	19.1 *10.2*	17.8 *9.7*	16.7 *9.1*	15.7 *8.6*	13.8 *7.4*	12.4 *6.4*	4.9
140	28.7 *4.9*	27.0 *7.6*	25.6 *9.3*	24.3 *10.3*	23.1 *10.9*	22.0 *11.2*	20.2 *11.0*	18.6 *10.6*	17.3 *10.0*	16.2 *9.4*	15.2 *8.8*	13.3 *7.6*	11.9 *6.6*	4.2
150	28.1 *6.0*	26.5 *8.5*	25.2 *10.0*	23.9 *10.9*	22.7 *11.4*	21.6 *11.6*	19.8 *11.4*	18.3 *10.9*						
160	27.6 *7.0*	26.1 *9.2*	24.8 *10.6*	23.6 *11.4*	22.4 *11.8*	21.3 *12.0*	19.5 *11.7*	18.0 *11.2*						
170	27.3 *7.8*	25.8 *9.8*	24.5 *11.1*	23.3 *11.8*	22.1 *12.2*	21.1 *12.3*	19.3 *12.0*	17.7 *11.4*						
180	26.9 *8.4*	25.5 *10.3*	24.2 *11.5*	23.0 *12.2*	21.9 *12.5*	20.9 *12.5*	19.1 *12.2*	17.5 *11.6*						
200	26.4 *9.4*	25.1 *11.1*	23.9 *12.1*	22.7 *12.7*	21.6 *12.9*	20.5 *12.9*	18.7 *12.5*	17.2 *11.8*						

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

* For areas of other intermediate stiffeners, multiply *italic* values by appropriate factor below:

Stiffener Steel Grade	Pairs of Stiffeners	Single Angle Stiffeners	Single Plate Stiffeners
F_y = 90 ksi	1.0	1.8	2.4
F_y = 36 ksi	2.5	4.5	6.0

TABLE 3-100

ALLOWABLE SHEAR STRESSES (F_v) IN PLATE GIRDERS (KSI) FOR 100 KSI SPECIFIED YIELD STRESS STEEL

(*Italic* values indicate gross area, as percent of web area, required for **pairs** of intermediate stiffeners of 100 ksi yield stress steel.) *

Slenderness ratios h/t: web depth to web thickness	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
30													40.0	40.0
40							40.0	40.0	40.0	40.0	40.0	40.0	39.5	38.0
50				40.0	40.0	40.0	37.5	35.7	34.6	34.1 *0.3*	33.7 *0.5*	32.9 *0.6*	32.4 *0.7*	30.4
60			40.0	38.5	35.7	34.2 *0.5*	33.1 *1.4*	32.2 *1.8*	31.5 *2.0*	30.8 *2.1*	30.2 *2.2*	28.7 *2.3*	27.7 *2.1*	23.1
70		40.0	36.3	34.1 *0.7*	33.2 *1.7*	32.3 *2.5*	30.7 *3.5*	29.1 *4.2*	27.8 *4.5*	26.7 *4.5*	25.8 *4.4*	24.0 *4.0*	22.8 *3.6*	17.0
80	40.0	35.7	33.8 *1.2*	32.8 *2.5*	31.5 *3.8*	30.0 *5.0*	28.0 *6.0*	26.4 *6.3*	25.0 *6.3*	23.9 *6.1*	22.9 *5.9*	21.0 *5.2*	19.7 *4.6*	13.0
90	36.8	33.9 *1.2*	32.8 *2.8*	31.1 *4.7*	29.6 *6.1*	28.2 *7.0*	26.2 *7.6*	24.5 *7.7*	23.1 *7.5*	21.9 *7.2*	20.9 *6.8*	19.0 *6.0*	17.6 *5.2*	10.3
100	34.3 *0.6*	33.1 *2.6*	31.3 *4.9*	29.7 *6.7*	28.2 *7.8*	26.9 *8.5*	24.9 *8.8*	23.2 *8.7*	21.7 *8.4*	20.5 *8.0*	19.5 *7.5*	17.5 *6.5*	16.0 *5.7*	8.3
110	33.6 *1.8*	32.0 *4.4*	30.2 *6.7*	28.6 *8.1*	27.2 *9.0*	25.9 *9.6*	23.9 *9.7*	22.2 *9.5*	20.7 *9.0*	19.5 *8.6*	18.4 *8.1*	16.4 *7.0*	14.9 *6.0*	6.9
120	33.0 *2.9*	31.0 *6.0*	29.3 *8.0*	27.8 *9.2*	26.4 *10.0*	25.2 *10.4*	23.2 *10.4*	21.4 *10.0*	20.0 *9.5*	18.7 *9.0*	17.6 *8.5*	15.6 *7.3*	14.0 *6.3*	5.8
130	32.1 *4.5*	30.2 *7.3*	28.6 *9.0*	27.1 *10.1*	25.8 *10.7*	24.6 *11.0*	22.6 *10.9*	20.9 *10.5*	19.4 *9.9*	18.1 *9.3*	17.0 *8.8*	14.9 *7.5*	13.4 *6.5*	4.9
140	31.4 *5.8*	29.6 *8.3*	28.1 *9.8*	26.6 *10.8*	25.3 *11.3*	24.1 *11.5*	22.1 *11.3*	20.4 *10.8*						
150	30.8 *6.8*	29.1 *9.1*	27.6 *10.5*	26.2 *11.3*	25.0 *11.7*	23.8 *11.9*	21.8 *11.6*	20.0 *11.1*						
160	30.3 *7.7*	28.7 *9.7*	27.3 *11.0*	25.9 *11.8*	24.6 *12.1*	23.5 *12.2*	21.5 *11.9*	19.7 *11.3*						
170	29.9 *8.4*	28.4 *10.3*	27.0 *11.5*	25.6 *12.1*	24.4 *12.4*	23.2 *12.5*	21.2 *12.1*	19.5 *11.5*						
180	29.6 *9.0*	28.1 *10.8*	26.7 *11.8*	25.4 *12.4*	24.2 *12.7*	23.0 *12.7*	21.0 *12.3*	19.2 *11.7*						
200	29.1 *9.9*	27.7 *11.5*	26.3 *12.4*	25.0 *12.9*	23.8 *13.1*	22.7 *13.1*	20.7 *12.6*	18.9 *12.0*						

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

* For areas of other intermediate stiffeners, multiply *italic* values by appropriate factor below:

Stiffener Steel Grade	Pairs of Stiffeners	Single Angle Stiffeners	Single Plate Stiffeners
F_y = 100 ksi	1.0	1.8	2.4
F_y = 36 ksi	2.8	5.0	6.7

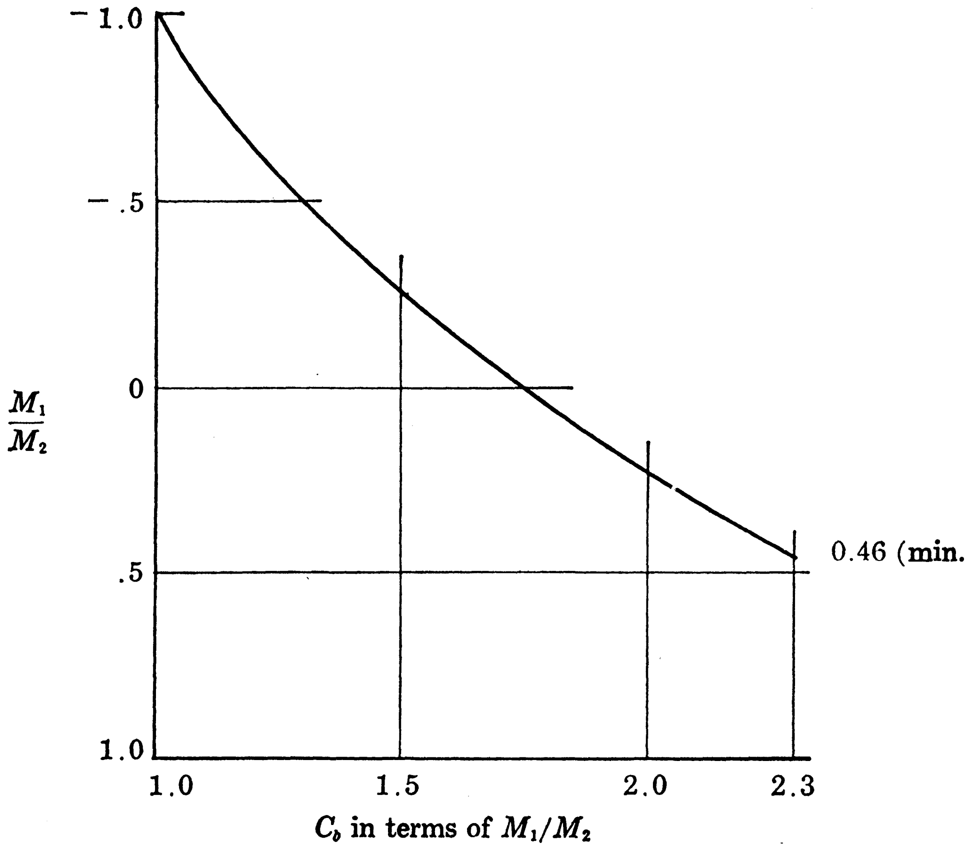

Fig. A1

APPENDIX B
Fatigue

SECTION B1 LOADING CONDITIONS AND 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 type and location of member or detail.

Loading conditions shall be classified as shown in Table B1.

TABLE B1

Loading Condition	Number of Loading Cycles	
	From	To
1	20,000[1]	100,000[2]
2	100,000	500,000[3]
3	500,000	2,000,000[4]
4	Over 2,000,000	

[1] Approximately equivalent to two applications every day for 25 years.
[2] Approximately equivalent to ten applications every day for 25 years.
[3] Approximately equivalent to fifty applications every day for 25 years.
[4] Approximately equivalent to two hundred applications every day for 25 years.

The type and location of material shall be categorized as shown 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 except that, in the case of stress reversal only, the value F'_{sr} given by Formula (B1) may be used as the stress range for those categories marked with an asterisk in Table B2.

$$F'_{sr} = \left(\frac{f_t + f_c}{f_t + 0.6 f_c}\right) F_{sr} \qquad \text{(B1)}$$

where f_t and f_c are, respectively, calculated tensile and compressive stresses considered as positive quantities, and F_{sr} is the allowable stress range given in Table B3.

TABLE B2

General Condition	Situation	Kind of Stress[1]	Stress Category (See Table B3)	Illustrative Example Nos. (See Fig. B1)[2]
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 penetration groove welds parallel to the direction of applied stress.	Rev. Rev. T or C	**B***[3] **B** **B**	3 4 3, 4
	Base metal and weld metal in members, without attachments, built up of plates or shapes connected by continuous fillet welds parallel to the direction of applied stress.	T, C or Rev.	**B**	4, 5, 6
	Calculated flexural stress, f_b, at toe of welds on girder webs or flanges adjacent to welded transverse stiffeners:			
	When $f_v \leqslant F_v/2$	T or Rev.	**C**	7
	When $f_v > F_v/2$	T or Rev.	**D**	7
	where F_v = allowable shear stress.			
	Base metal at end of partial length welded cover plates having square or tapered ends, with or without welds across the ends.	T, C or Rev.	**E**	5

[1] "T" signifies range in tensile stress only: "C" signifies range in compressive stress only; "Rev." signifies a range involving reversal of tensile or compressive stress; "S" signifies range in shear including shear stress reversal.

[2] These examples are provided as guide lines and are not intended to exclude other reasonably similar situations.

[3] Formula (B1) applicable in situations identified by asterisk (*).

[4] Where stress reversal is involved, use of A307 bolts is not recommended.

TABLE B2 (*continued*)

General Condition	Situation	Kind of Stress[1]	Stress Category. (See Table B3)	Illustrative Example Nos. (See Fig. B1)[2]
Mechanically fastened connections	Base metal at net section of high-strength-bolted connections, except bearing-type connections subject to stress reversal and axially loaded joints which induce out-of-plane bending in connected material.	T or Rev.	**A**	8
	Base metal at net section of other mechanically fastened joints.[4]	T or Rev.	**B**	8, 9
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.	T or Rev.	**A**	10
	Base metal and weld metal at full penetration groove welded splices of rolled and welded sections having similar profiles, when welds are ground flush.	T or Rev.	**B**	10, 11
	Base metal and weld metal in or adjacent to 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.	T or Rev.	**B**	12, 13

TABLE B2 (*continued*)

General Condition	Situation	Kind of Stress[1]	Stress Category. (See Table B3)	Illustrative Example Nos. (See Fig. B1)[2]
Groove welds (cont'd)	Base metal and weld metal in or adjacent to 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.	T Rev. T or Rev.	C C* C	10 10 11, 12, 13
	Base metal or weld metal in or adjacent to full penetration groove welds in tee or cruciform joints.	T Rev.	D D*	14 14
	Base metal at details attached by groove welds subject to transverse and/or longitudinal loading.	T, C or Rev.	E	15
	Weld metal of partial penetration transverse groove welds, based on effective throat area of the weld or welds.	T or Rev.	G	16
Fillet welded connections	Base metal at intermittent fillet welds.	T, C 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, C or Rev.	E	17, 18, 19, 20
	Continuous or intermittent longitudinal or transverse fillet welds (except transverse fillet welds in tee joints) and continuous fillet welds	S	F	5, 17, 18, 19, 21

TABLE B2 (*continued*)

General Condition	Situation	Kind of Stress[1]	Stress Category. (See Table B3)	Illustrative Example Nos. (See Fig. B1)[2]
Fillet welded connections (cont'd)	subject to shear parallel to the weld axis in combination with shear due to flexure.			
	Transverse fillet welds in tee joints.	S	**G**	20
Miscellaneous details	Base metal adjacent to short (2 in. maximum length in direction of stress) welded attachments.	C T or Rev.	**C** **D**	22, 23, 24 22, 23, 24, 25
	Base metal adjacent to longer fillet welded attachments.	T, C or Rev.	**E**	26
	Base metal at plug or slot welds.	T, C or Rev.	**E**	27
	Shear stress on nominal area of stud-type shear connectors.	S	**G**	22
	Shear on plug or slot welds.	S	**G**	27

TABLE B3

Category (From Table B2)	Allowable Range of Stress, F_{sr} (ksi)			
	Loading Condition 1 F_{sr1}	Loading Condition 2 F_{sr2}	Loading Condition 3 F_{sr3}	Loading Condition 4 F_{sr4}
A[1]	40	32	24	24
B	33	25	17	15
C	28	21	14	12
D	24	17	10	9
E	17	12	7	6
F	17	14	11	9
G	15	12	9	8

[1] For A514 steels in Category A, substitute the following values: $F_{sr1} = 45$, $F_{sr2} = 35$, $F_{sr3} = 25$ and $F_{sr4} = 25$.

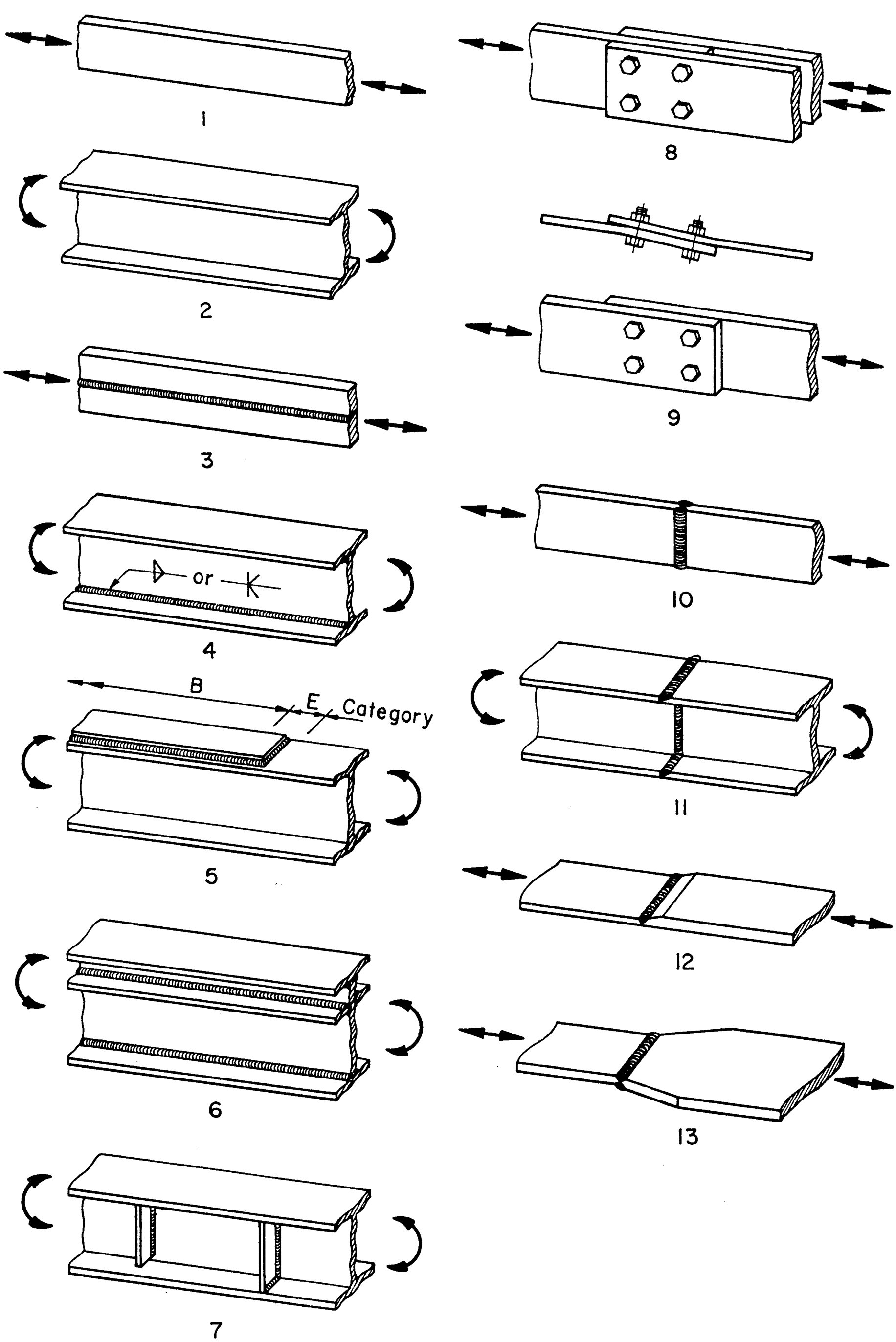

Fig. B1. Illustrative Examples

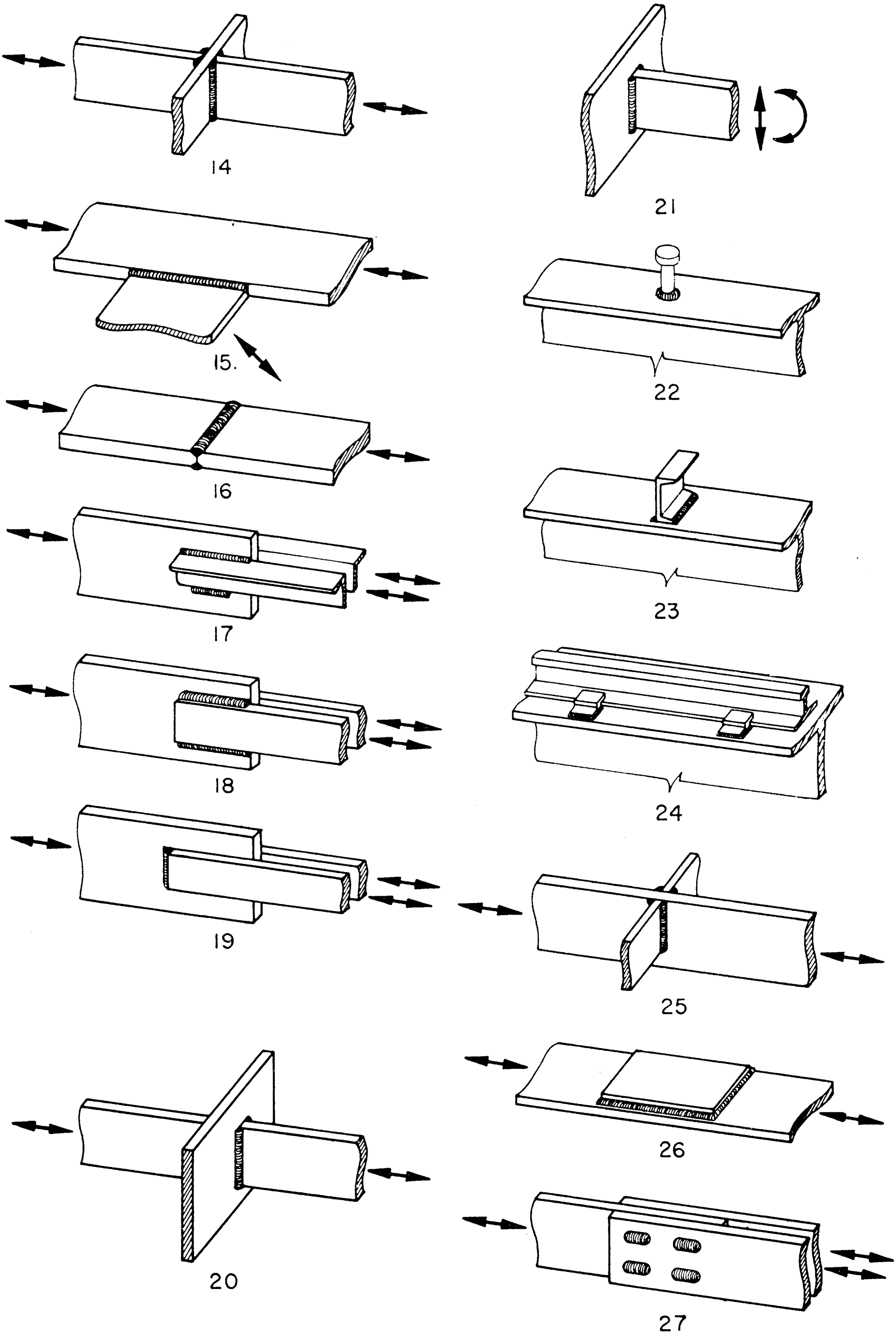

Fig. B1. Illustrative Examples (continued)

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, 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) to (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 $0.6F_yQ_s$ nor the applicable value as provided in Sect. 1.5.1.4.6. 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 $76.0/\sqrt{F_y} < b/t < 155/\sqrt{F_y}$:

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

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

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

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

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

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

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

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

For stems of tees:

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

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

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

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

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 flange width to profile depth	Ratio of flange thickness to web or stem thickness
Built-up or Rolled Channels	$\leqslant 0.25$	$\leqslant 3.0$
	$\leqslant 0.50$	$\leqslant 2.0$
Built-up Tees	$\geqslant 0.50$	$\geqslant 1.25$
Rolled Tees	$\geqslant 0.50$	$\geqslant 1.10$

SECTION C3 EFFECTIVE WIDTH—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:

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

For other uniformly compressed elements:

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

where

b = actual width of a stiffened compression element as defined in Sect. 1.9.2.1
t = its thickness
f = compressive stress in the element computed on the basis of its section properties as provided hereinafter. In the case of axial loading and flexure on extreme fibers, $f = 0.6F_yQ_s$, except as otherwise provided for wind and seismic loading

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.

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, rather than the actual width, shall be used 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:

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

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. 1.5.1.4.6.

COMMENTARY

ON THE

SPECIFICATION FOR THE DESIGN, FABRICATION & ERECTION OF STRUCTURAL STEEL FOR BUILDINGS

FEBRUARY 12, 1969

AMERICAN INSTITUTE
OF STEEL CONSTRUCTION
101 PARK AVENUE, NEW YORK, N.Y. 10017

Commentary

ON THE SPECIFICATION FOR THE DESIGN, FABRICATION AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS

INTRODUCTION

In the belief that the designer can make more efficient use of the Specification if he knows the basis for its various provisions, this Commentary has been prepared.

Many provisions, notably in the sections dealing with fabrication and erection practices, have evolved from years of shop and field experience and need no further elaboration. Attention is directed primarily to less widely understood measures and particularly to modifications appearing for the first time. Many of these are the outgrowth of extensive research which has been carried out in recent years.

Part 1 of the Specification includes all of the provisions necessary for a working-stress design covering all three types of construction. Part 2 covers provisions applicable to plastic design.

SECTION 1.2 TYPES OF CONSTRUCTION

In order that adequate instructions can be issued to the shop and erection forces, the basic assumptions underlying the design must be thoroughly understood by all concerned. As in the earlier AISC Specification, 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 have been reworded in the current Specification, but without change in intent. Justification for these provisions has been discussed by Sourochnikoff,* Disque** 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 USA Standards Institute are recommended as the basis for design.

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* *Sourochnikoff, B.* Wind Stresses in Semi-Rigid Connections of Steel Framework, *1950 ASCE Transactions.*

** *Disque, R. O.* Wind Connections with Simple Framing, *AISC Engineering Journal, Vol. 1, No. 3.*

SECTION 1.4 MATERIAL

The 1961 edition of the Specification provided for the use of structural steel having a specified minimum yield point up to, but not exceeding, 50 kips per square inch. The grades of structural steel now approved for use under the Specification, covered by ASTM standards adopted since that time, extend the yield stress to 100 kips per square inch.

A number of other ASTM specifications are also now listed, covering types of material having infrequent application but suitable for use under the Specification.

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. However, the specified terms "yield point" and "yield stress" are used where they are uniquely applicable.

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

When requested to do so, the fabricator must make affidavit that all steel specified to a yield stress in excess of 36 kips per square inch has been provided in accordance with the plans and Specification.

SECTION 1.5 ALLOWABLE STRESSES

1.5.1. Structural Steel

Because of the introduction of steels having various specified minimum yield stresses, it is convenient to express permissible working stresses in terms of yield stress, F_y.

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

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

1.5.1.1 Tension

The 5/3 factor of safety with respect to yield stress used in determining the basic working stress for the newer and stronger steels is the same as that provided since the Specification was first adopted.

However, a further precaution has been added, applicable only at the net section of axially loaded members. Here a factor of safety of 2 with respect to specified minimum tensile strength must also be provided. This latter provision, of course, would apply only to steel having a yield stress-to-tensile strength ratio 5/6 or greater.

The working stress at the net section at pin holes is based upon research* and experience with eye-bars.

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* *Johnston, B. G.* Pin-Connected Plate Links, *1939 ASCE Transactions.*

1.5.1.2 Shear

While the shear yield stress of structural steel has been variously estimated as between one-half and five-eighths 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 two-thirds 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.

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 based upon the assumption that the moment M is resisted by a couple having an arm equal to $0.95d_b$, where d_b is the depth of the member introducing the moment. Designating as d_c the depth of the member entering the joint more or less at right angles to it, and noting that A_{bc} is approximately equal to $d_b \times d_c$, the minimum thickness of the web not requiring reinforcement can be computed from the equation

$$\text{allowable shear stress} = 0.40F_y = \frac{12M}{0.95A_{bc}t_{\min}}$$

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 Column Research Council.* This estimate assumes that the upper limit of elastic buckling failure is defined by an average column stress equal to one-half 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

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

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, approaching the Euler slenderness range, the factor is increased 15 percent, to approximately the value provided in the AISC Specification since it was first published 46 years ago.

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* Column Research Council Guide to Design Criteria for Metal Compression Members, *Second Edition, Eqs. (2.11) and (2.12).*

In order 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. Substituting $12\pi^2E/23$ for the previous rounded-off value, 149,000,000, in Formula (1.5-2) affords an exact convergence with Formula (1.5-1).

Tables giving the permissible stress for columns and other compression members for a number of the approved 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 columns slender enough to fail by elastic buckling, is based upon a constant factor of safety of 23/12 with respect to the elastic (Euler) column strength.

1.5.1.3.3 By dividing the values obtained from Formulas (1.5-1) and (1.5-2) by the factor $\left(1.6 - \dfrac{l}{200r}\right)$ 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.

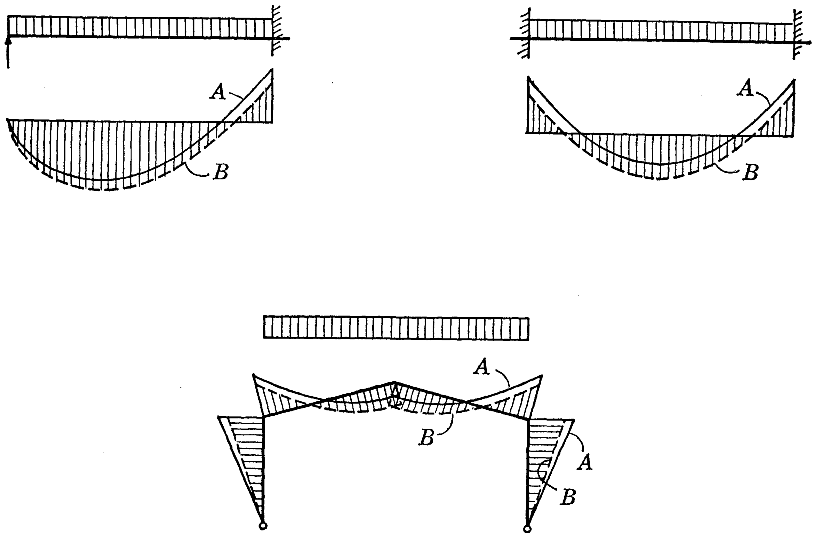
Since Formula (1.5-3) does take advantage of end restraint, the full unbraced length of the member (rather than a reduced effective length, assuming $K < 1.0$) should always be used, and the formula should be restricted to members which are more or less fixed against rotation and translation at braced points.

1.5.1.4 Bending

1.5.1.4.1 When flexural members 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.

Research in plastic design has demonstrated that local buckling will not occur in homogeneous sections meeting the requirements of subparagraphs a to e, inclusive, of Sect. 1.5.1.4.1 before the full plastic moment is reached. Practically all rolled W shapes of A36 steel and a large proportion of these shapes having a yield stress of 50 ksi 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 rolled W shapes 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. Fig. C 1.5.1 illustrates the application of this provision by comparing calculated moment diagrams with the diagrams as altered by this provision.



A = Actual moment diagram
B = Modified diagram corresponding to 10 percent moment reduction allowance at interior supports

Fig. C1.5.1

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 (1.5-5) 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 $52.2/\sqrt{F_y}$, and $0.60F_y$ when this ratio is no more than $95.0/\sqrt{F_y}$. 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 weakest 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.

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 $l/b < 2{,}500/F_y$, 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 and **1.5.1.4.6** The allowable bending stress for all other flexural members is given as $0.60F_y$, provided the compression flange is braced laterally at relatively close intervals ($i/b_f \leqslant 76.0/\sqrt{F_y}$).

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) of (1.5-6b) and (1.5-7) provides a reasonable design criterion in more convenient form.

As in Formula (4) of the 1963 edition of the Specification, 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. The new Formulas (1.5-6a) and (1.5-6b) differ from the earlier Formula (4) in two ways:

1. Whereas the earlier provisions required no stress reduction when l/r was less than 40 (regardless of yield stress value) and then a reduction to the value obtained from the parabolic expression, the new formulas, by increasing F_b at $l = 0$ from $0.60F_y$ to $2F_y/3$, provides a continuous stress relationship with the unbraced length when F_b is reduced from the maximum permissible value of $0.60F_y$.
2. Whereas the earlier single Formula (4) applied even in the range of elastic buckling stress (on the assumption that Formula (5) would govern), the replacement of Formula (4) is liberalized in this range by the addition of an Euler-type expression, since this assumption is not always correct.

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* Column Research Council Guide to Design Criteria for Metal Compression Members, *Second Edition, Sect. 4.2.*

Formula (1.5-7) is a convenient approximation which assumes the presence of both lateral bending resistance and St. Venant torsional resistance. 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. 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.

For some sections having a compression flange area distinctly smaller than the tension flange area, Formula (1.5-7) may be unconservative; hence, 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 by Formula (1.5-6b), affords the better estimate of buckling strength. While these latter formulas underestimate this 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), like the more precise, complex expressions it replaces, 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 it 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 action in conjunction with the vertical loads. Such members usually can be proportioned for the full permissible bending stress when that 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 where, in the case of combined bending and axial compression, this adjustment is provided by the factor C_m in Formula (1.6-1a).

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. This equivalent radius of gyration, r_{equiv}, 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.‡

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* Column Research Council Guide to Design Criteria for Metal Compression Members, *Second Edition, Eq. 4.8.*

** Ibid., *Eq. 4.13.*

† Ibid., *Eqs. (4.9c), (4.30), (4.31) or (4.32).*

‡ Ibid., *Eq. (2.2).*

For the case of a doubly-symmetrical I-shape beam,

$$r^2_{equiv} = \frac{I_y}{2S_x}\sqrt{d^2 + \frac{0.156l^2J}{I_y}}$$

where I_y is the minor axis moment of inertia of the member, S_x is its major axis section modulus, and

$$J = \frac{2b_f t_f^3}{3} + \frac{dt^3}{3}$$

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 rivets. The lower value, nine-tenths of the yield stress of the part containing the pin hole, provides a safeguard against instability of the plate beyond the hole,* which is considerably larger than a rivet hole.

1.5.2 Rivets, Bolts, and Threaded Parts

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 greater convenience in the proportioning of high strength bolted connections, permissible stresses for the bolts are given in terms applicable to their nominal body area, i.e., the area of the unthreaded shank. However, for A307 bolts (which are available in sizes up to 4 in. in diameter) and threaded parts other than high strength bolts, the allowable tensile stress is applicable to a stress area equal to $0.7854\ [D - (0.9743/n)]^2$. This area (intermediate between gross area and area at the root of the thread) when multiplied by the mechanical properties of the unthreaded material, has been found to more closely predict the tensile strength of larger diameter threaded parts, such as might be used for anchor bolts or upset rods.

In recognition of the protection against notch effect in the threading, assured by the required initial tightening of high strength bolts, the Research Council on Riveted and Bolted Structural Joints has recommended a relatively higher working stress in tension for high strength bolts.

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.**

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* *Johnston, B. G.* Pin-Connected Plate Links, *1939 ASCE Transactions.*

** *Munse, W. H.* Research on Bolted Connections, *1956 ASCE Transactions, p. 1265.*

1.5.2.1 Shear

Connections which transmit load by means of shear in their fasteners are categorized as "friction-type" or "bearing-type". The former depend upon sufficiently high clamping force to prevent slip of the connected parts. 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. Hence riveted connections and connections made with A307 bolts for shear are treated as bearing-type. The high clamping force produced by properly tightened high strength bolts is sufficient to prevent slip of the connected parts when an equal number of these bolts are substituted for the rivets of equal size that would be required to transmit a given load — A325 bolts for A502 Grade 1 rivets and A490 bolts for A502 Grade 2 rivets.

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. In the case of high strength bolts, two allowable shear stress values are given: one where threading is excluded from the shear plane and one where it is not. Since it is not customary to control this feature in the case of A307 bolts, it is assumed that threading may extend into the shear plane and the allowable shear value, applicable to the gross area, is reduced accordingly.

1.5.2.2 Bearing

Bearing values are provided, not as a protection to the fastener, because it needs no such protection, but as an index of the efficiency of net sections computed in accordance with Sect. 1.14.3. The same index is valid for joints assembled with rivets or with bolts, regardless of fastener shear strength or the presence or absence of threads in the bearing area. Tests of riveted joints* have shown that the tensile strength of the connected part is not impaired when the bearing pressure on the computed contact area of the fastener is as much as 2¼ times the tensile stress permitted on the net area of the part. In this investigation the contact (bearing) area was computed, according to the usual convention, as the product of nominal fastener diameter and thickness of the connected part. No difference was observed between single-shear bearing and enclosed bearing. Based on these findings, the recommended working stress is the same for single-shear and double-shear bearing, and approximately equal to 2¼ times the tensile working stress recommended for determining required net area.

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.

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* *Jones, Jonathan* Effect of Bearing Ratio on Static Strength of Riveted Joints, *1958 ASCE Transactions.*

In earlier editions of the AISC Specification, working stresses were not given for fillet welds made with electrodes stronger than the E70 classification. The stresses that were given were known to be overly conservative for their recommended use with E60 and E70 classifications. Based upon recent tests,* the allowable stress on fillet welds, deposited on "matching" base metal or steel having mechanical properties higher than those specified for such base metal, is now given in terms of the specified tensile strength of the weld metal.

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 analagous 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 primarily in compression, bearing, or in tension parallel to the longitudinal axis of the groove, they may be proportioned to resist such stress at the same unit value permitted in the base metal.

1.5.4. Cast Steel

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.

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

$$\frac{1}{\left(1 - \frac{f_a}{F'_e}\right)}$$

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.

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* *Higgins, T. R. and Preece, F. R.* Proposed Working Stresses for Fillet Welds in Building Construction, *Welding Journal Research Supplement, Oct., 1968.*

When the computed axial stress is no greater than 15 percent of the permissible axial stress, the influence of

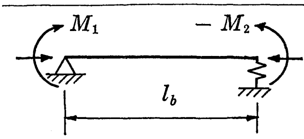
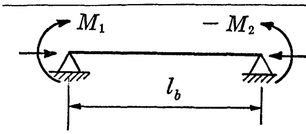
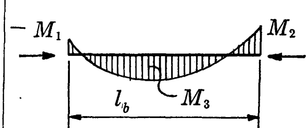
$$\frac{C_m}{\left(1 - \dfrac{f_a}{F'_e}\right)}$$

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. Three categories and the appropriate provisions of Sect. 1.6.1 are listed in Table C 1.6.1.1.

TABLE C 1.6.1.1

Category	Loading conditions ($f_a > 0.15F_a$)	f_b	C_m	Remarks
A	Computed moments maximum at end; joint translation not prevented	$\frac{M_2}{S}$	0.85	 $M_1 < M_2$; $\frac{M_1}{M_2}$ negative as shown. Check both Formulas (1.6-1a) & (1.6-1b)
B	Computed moments maximum at end; no transverse loading; joint translation prevented	$\frac{M_2}{S}$	$\left(0.6 \pm 0.4\frac{M_1}{M_2}\right)$ but not less than 0.4	 Check both Formulas (1.6-1a) & (1.6-1b)
C	Transverse loading; joint translation prevented	$\frac{M_2}{S}$ Using Formula (1.6-1b); $\frac{M_3}{S}$ Using Formula (1.6-1a)	$1 + \psi\frac{f_a}{F'_e}$	 Check both Formulas (1.6-1a) & (1.6-1b)

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). The maximum moment between points of support, however, is used to compute the bending stress for use in Formula (1.6-1a).

Category A 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 B 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.

For this category, the greatest eccentricity, and hence the greatest amplification, occurs when 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 evaluate properly 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† 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

* 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.

** Column Research Council Guide to Design Criteria for Metal Compression Members, *p. 163.* (*Discussion in the Guide uses beam sign convention.*)

† *Austin, W. J.* Strength and Design of Metal Beam-Columns, *ASCE Journal of the Structural Division, April, 1961.*

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 C is exemplified by the compression chord of a truss, subject to transverse loading between panel points. For this case the value for C_m can be computed using the expression

$$C_m = 1 + \psi \frac{f_a}{F'_e}$$

where

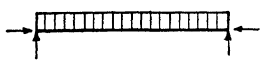
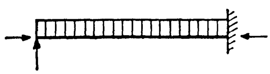
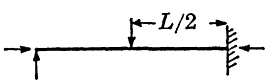
$$\psi = \frac{\pi^2 \delta_0 EI}{M_0 L^2} - 1$$

δ_0 = maximum deflection due to transverse loading

M_0 = maximum moment between supports due to transverse loading

Values for ψ for several conditions of loading and end restraint are given in Table C 1.6.1.2.

TABLE C 1.6.1.2

Case	ψ	C_m
	0	1.0
	−0.3	$1 - 0.3 \frac{f_a}{F'_e}$
	−0.4	$1 - 0.4 \frac{f_a}{F'_e}$
	−0.2	$1 - 0.2 \frac{f_a}{F'_e}$
	−0.4	$1 - 0.4 \frac{f_a}{F'_e}$
	−0.6	$1 - 0.6 \frac{f_a}{F'_e}$

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 required 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 between points of lateral support because the secondary moment, which is the product of the deflection and the axial tension, is opposite in sense to the applied moment, instead of being of the same sense and additive, as in columns.

1.6.3 Shear and Tension

Tests have shown* that the strength of rivets subject to combined tension and shear resulting from externally applied forces (in addition to existing internal shrinkage stresses) can be closely defined by either (1) an ellipse, or (2) three straight lines, as shown in Fig. C 1.6.3.1.

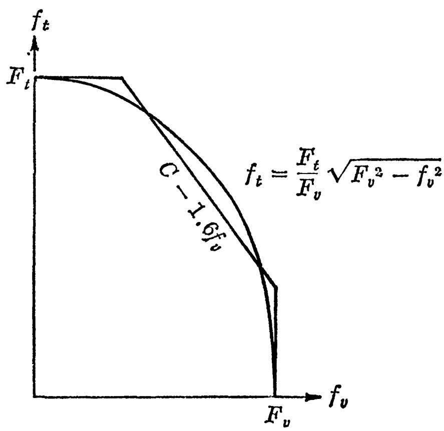


Fig. C1.6.3.1

In most cases the latter representation is the more simple of application, since it requires no modification of the stress recommended for either shear or tension when these stresses act in conjunction, respectively, with relatively large concurrent tension or shear stresses. Therefore, it is the only one given in Sect. 1.6.3, since the inclusion of more than one method is hardly warranted. However, solutions based upon use of the ellipse are equally valid and should be allowed. Any differences in the number of fasteners required by the two prescriptions would be small.

Similar interaction formulas have been derived for the other approved types of fasteners from ellipses constructed with major and minor axis half-lengths equal, respectively, to the tension and shear stress given in Sect. 1.5.2.

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* *Higgins, T. R. and Munse, W. H.* How much Combined Stress Can A Rivet Take? *Engineering News-Record, Dec. 4, 1962.*

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

Because most members in building frames need not be designed for fatigue, the provisions covering such designs have been placed in Appendix B.

Where fatigue is a design consideration, its severity is most significantly affected by the number of load applications and the magnitude of the stress range. It is aggravated by the presence of stress raisers to a varying degree, depending on the particular detail. Consequently, when fatigue is of concern, all the applicable 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 1½ times the applicable value given in Table B3 for Loading Condition 1.

Except where indicated by "C" under "Kind of Stress" in Table B2, fluctuation in stress which does not involve tensile stress is not considered a fatigue situation.

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 loading 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.

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*.

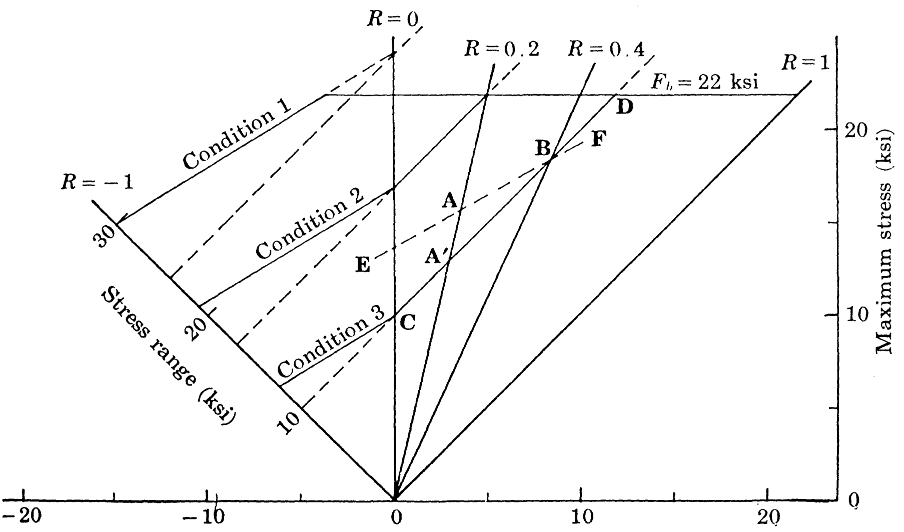


Fig. C1.7.1

The reason for this shift in design criteria is apparent when the provisions of Appendix B are presented in the form of the familiar modified Goodman diagram often used as a design aid in lieu of such formulas. In Fig. C 1.7.1 the provisions of a category "D*" detail of A36 Steel are plotted diagramatically in this form. With maximum stress and stress ratio as the governing parameters, note that points **A** and **B** define substantially different critical maximum stress, with only slightly different stress ratios. However, with the line **CD** drawn parallel to the 45° boundary line representing static loading (min = max; $R = 1$) the permissible range in stress for points **A′** and **B** (or any point between **C** and **D**) is the same. Only minor change in stress range would result had the slope of line **CD** been varied somewhat from 1 on 1, as often indicated in earlier evaluations of fatigue test results and as indicated by the line **EF**.

The allowable range of stress for Loading Condition 3, regardless of maximum stress value, can be read on the maximum stress scale and is represented by the distance **OC**.

This is the value F_{sr3} given in Table B3. It might also be read from a scale plotted on the $R = -1$ boundary line, so laid off that

$$\text{stress range scale} : \text{max stress scale} = \mathbf{1} : \sqrt{2}$$

In developing the stress range values given in Table B3, published fatigue data and data obtained in continuing research were reviewed. In adopting a constant stress range basis for designs involving fatigue (in the interest of a simpler design procedure), it was realized that a number of known characteristics of fatigue strength data would not be taken into consideration. For example, except for A514 steel in category "A", the provisions do not recognize any increase in fatigue strength for the higher strength steels, as compared with that of A36 steel. For a particular category, this increased strength varies for the different steels depending upon the number of cycles of repeated loading.

As a consequence, the provisions may not provide a uniform factor of safety for the different strength steels. However, deviations from a uniform factor of safety are on the conservative side. Comparison of the fatigue provisions of this Specification with available test data indicate that the safety factors inherent in the recommended fatigue provisions are commensurate with static stress provisions.

In a few instances, identified by asterisks in Table B2, the extent of this conservatism warranted the liberalization provided by Formula (B1), which was derived from the expression for maximum permissible fatigue stress:

$$F_r = \frac{f_{ro}}{1 - mR}$$

where

R = Stress ratio, having a negative value with reversal of stress
f_{ro} = Maximum permissible stress when $R = 0$
m = Slope of a fatigue strength line as presented in a modified Goodman diagram ($m \sim 0.6$)

Substituting f_t for F_r, F_{sr} for f_{ro}, 0.6 for m, and $-(f_c/f_t)$ for R, and noting that $F'_{sr} = f_t + f_c$,

$$F'_{sr} = \frac{f_t + f_c}{f_t + 0.6f_c} F_{sr} \qquad \text{(B1)}$$

Since Fig. C 1.7.1 was drawn for category "D*", where Formula (B1) applies when a reversal of stress is involved, the fatigue strength lines (shown solid) represent the liberalization in stress range provided by Formula (B1) as compared with the dashed lines which would govern for category "D".

While greater fatigue strength than indicated by the provisions of Appendix B is attainable using special treatment, and is often provided in the case of manufactured products, the application of such treatment to as-fabricated structural steel is seldom economical. An exception is the grinding flush of full penetration groove welded splices which must be located where the alternate to the higher stress range permitted would be a substantial increase in required member size.

SECTION 1.8 STABILITY AND SLENDERNESS RATIOS

Considerable attention has been given in the technical literature to the subject of "effective" column length (as contrasted with actual unbraced length) as a factor in estimating column strength. The topic is reviewed at some length in Sect. 2.8 of the *Guide to Design Criteria for Metal Compression Members.*

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 a frame dependent entirely upon its own bending stiffness for stability against sidesway, i.e., uninhibited lateral movement, as shown in

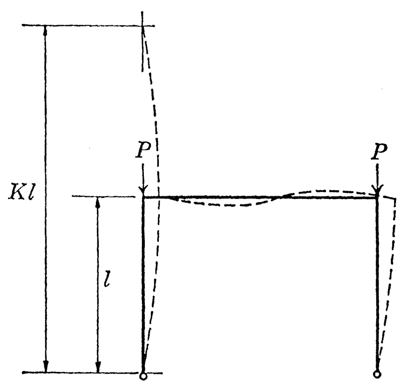
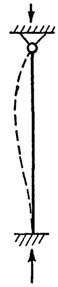
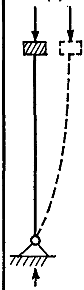
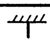
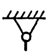


Fig. C 1.8.1

Fig. C 1.8.1, the "effective" length of these columns will exceed their actual length. On the other hand, if the same frame were braced in such a way that lateral movement of the tops of the columns with respect to their bases (translation or sidesway) were prevented, the effective length would be less than the actual length, due to the restraint (resistance to joint rotation) provided by the horizontal member. 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 C 1.8.1. Also shown are suggested design values recommended by the Column Research Council for use when these conditions are approximated in actual design. In general, these suggested values are slightly higher than their theoretical equivalents, since joint fixity is seldom fully realized.

TABLE C 1.8.1

	(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				

If the column base in case (f) of Table C 1.8.1 were truly pinned, K would actually exceed 2.0 for a frame such as that pictured in Fig. C 1.8.1, 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* that the restraining influence of foundations, even where these footings are designed only for vertical load, can be very substantial in the case of flat-ended column base details with ordinary anchorage. For this condition, a design K-value of 1.5 would generally be conservative in case (f).

While ordinarily the existence of masonry walls provides enough lateral support for tier building frames to prevent sidesway, 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

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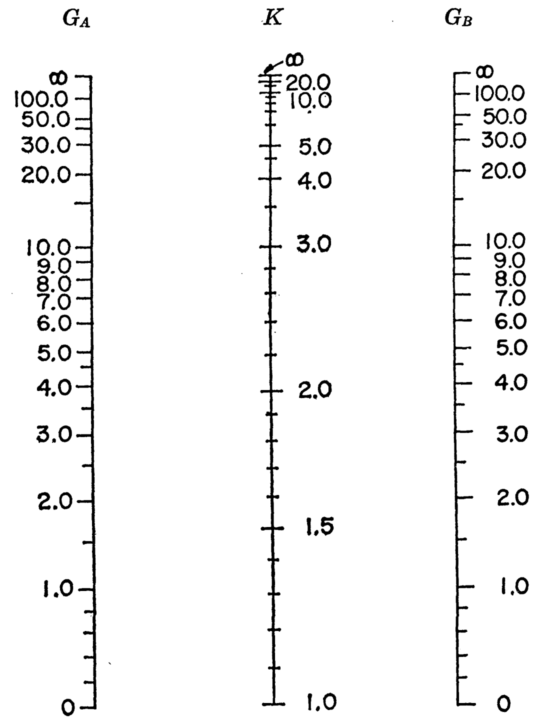
* *Galambos, T. V.* Influence of Partial Base Fixity on Frame Stability, *ASCE Journal of the Structural Division, May, 1960.*

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 C 1.8.1 to very complex analytical procedures.



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

Once a trial selection of framing members has been made, the use of the alignment chart in Fig. C 1.8.2 affords a fairly rapid method for determining suitable K-values.

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.**

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* *Bleich, F.* Buckling Strength of Metal Structures, *pp. 260–265.*

** *Winter, G.* Lateral Bracing of Columns and Beams, *ASCE Journal of the Structural Division, March, 1958.*

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 Sects. 1.9.1.2 or 1.9.2.2, no reduction in allowable stress is necessary in order to prevent local buckling. The design of members containing compression elements having a width-thickness ratio somewhat in excess of these limits is generally conservative if the area provided by the excessive width is ignored, as has been permitted in earlier editions of the Specification.

This expediency, in the case of unstiffened elements, raises a question as to eccentricity between actual and admissible cross-sectional area axes, makes no provision for computing an "effective" section modulus, and may even result in unconservative design. For the infrequent situation where width-thickness ratios substantially in excess of the limits given in Sect. 1.9 are involved, the provisions of Appendix C afford a better design procedure.

Formulas (C2-1) to (C2-6) are based upon the expression* for critical buckling stress for a plate having one or both edges parallel to an in-plane compressive force supported against lateral deflection, with or without torsional restraint along these edges. For this case

$$\sigma_c = k\left[\frac{\pi^2 E\sqrt{\eta}}{12(1 - \nu^2)(b/t)^2}\right] \tag{C1}$$

where η is the ratio of the tangent modulus to the elastic modulus, E_t/E, and ν is Poisson's ratio. The idealized value $k = 0.425$, assumes nothing more than knife-edge lateral support, applied along one edge of the unstiffened element, at the mid-plane of the element providing it. 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.

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. C 1.9.1.

Formula (C2-5) assumes a decrease in the torsional restraint characteristic of tees cut from rolled shapes, which might be expected of tees of quite different proportions formed by welding two plates together.

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* Column Research Council Guide to Design Criteria for Metal Compression Members, *Sect. 3.3.*

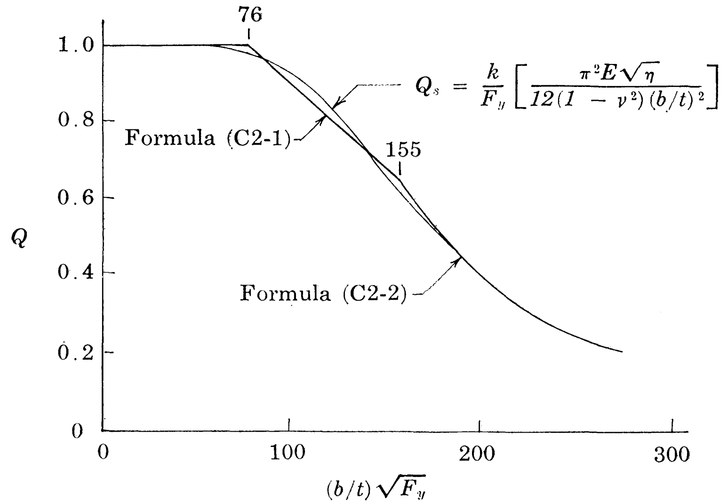

Fig. C1.9.1

It has been shown* 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 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† 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

$$\frac{b_e}{t} = 1.9\sqrt{\frac{E}{f}}\left[1 - \frac{C}{(b/t)}\sqrt{\frac{E}{f}}\right]$$

* *Chajes, A. and Winter, G.* Torsional Flexural Buckling of Thin-Walled Members, *ASCE Journal of the Structural Division, August, 1965.*

** *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, p. 53.*

† *Winter, G.* Strength of Steel Compression Flanges, *1947 ASCE Transactions.*

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 $0.6F_y$, or when 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 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, this allowable stress must be modified by the reduction factor Q_s.

SECTION 1.10 PLATE GIRDERS AND ROLLED BEAMS

1.10.1 Proportions

As in earlier editions, 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. This provision is now extended to include 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

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* *Lilly, S. B. and Carpenter, S. T.* Effective Moment of Inertia of a Riveted Plate Girder, *1940 ASCE Transactions.*

flanges. Numerous tests, summarized in a recent report,* 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 that of a homogeneous one.

1.10.2 Web

The limiting web depth-thickness ratio, included in the 1961 edition of the AISC Specification to prevent vertical buckling of the compression flange into the web before attainment of yield stress in the flange due to flexure, may now be increased when transverse stiffeners are provided, spaced not more than 1½ times the girder depth on centers.

The earlier provision, which was based on an analysis** that placed no limitation on the spacing of transverse stiffeners, correlated reasonably well with tests performed on girders made of A7 steel having a specified yield stress of 33 ksi. The more liberal provision ($h/t \leqslant 2000/\sqrt{F_y}$ is based upon more recent tests† on both homogeneous and hybrid girders with flanges having a specified yield stress of 100 ksi 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 cut-off point beyond which it is not needed, it must be developed in an extension beyond this point by enough rivets, high strength bolts, 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 beginning of extension
Q = Statical moment of cover plate area about neutral axis of cover-plated section
I = Moment of inertia of cover-plated section

When the nature of the loading is such as to produce repeated variations of stress, 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

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* Design of Hybrid Steel Beams, Report of Subcommittee 1 of the Joint ASCE-AASHO Committee on Flexural Members, *ASCE Journal of the Structural Division, June, 1968.*

** *Basler, K. and Thürlimann, B.* Strength of Plate Girders in Bending, *ASCE Journal of the Structural Division, August, 1961.*

† Design of Hybrid Steel Beams, Report of Subcommittee 1 of the Joint ASCE-AASHO Committee on Flexural Members, *p. 1412, ASCE Journal of the Structural Division, June, 1968.*

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 cut-off 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

To provide better clarity, the provisions of Sect. 1.10.5 have been rearranged in the current edition of the Specification, but without substantive change of the provisions in Sect. 1.10.5 of the 1963 adoption.

Provisions governing the design of plate girders prior to the 1961 revision were based upon the assumption that the limit of structural usefulness of a girder web is attained when the level of stress in the web reaches the so-called "buckling" stage. Unlike columns, however, 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 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* and corroborated in an extensive program of tests.** These methods form the basis for Formula (1.10-2). Use of tension field action is not counted upon when

$$\frac{0.6F_y}{\sqrt{3}} \leqslant F_v \leqslant 0.4F_y$$

or 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.

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* *Basler, K.* Strength of Plate Girders in Shear, *ASCE Journal of the Structural Division, October, 1961.*

** *Basler, K., Yen, B. T., Mueller, J. A. and Thürlimann, B.* Web Buckling Tests on Welded Plate Girders, *Welding Research Council Bulletin No. 64.*

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

$$\left(\frac{260}{h/t}\right)^2$$

with a maximum spacing of 3 times the girder depth.

When required, their maximum permissible longitudinal spacing is dependent upon three parameters: a/h, h/t and f_v. For the convenience of the designer, their relationship with one another is presented in Tables 3-36 through 3-100 of Appendix A for several specified yield stresses covered by the Specification. 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 table. Comparison of the web and stiffener material required with two or three trial web thicknesses will quickly indicate the most economical combination.

The corresponding gross area of intermediate stiffeners, given as a percent of the web area, is shown in italics in the column headed by the required 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 when the a/h and h/t ratios are small enough to permit a shear stress larger than $0.35F_y$, which is covered by Formula (1.10-1). For such cases tension field action is not counted upon.

At the ends of the girder, the spacing between adjacent stiffeners is limited to $11{,}000t/\sqrt{f_v}$, to provide an "anchor" for the tension fields developed in interior panels. The stiffeners bounding panels containing large holes likewise are required to be spaced close enough together so that the shear in these panels can be supported without tension field action.

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 new 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 the relative bending strength of this flange being 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) * provides for a reduced allowable flange bending stress applicable to both flanges. The extent of the reduction is dependent upon the ratio of web area to a flange area and the ratio 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** that plate girder webs subject to tension field action can be proportioned on the basis of:

1. Maximum permissible bending stress when the concurrent shear is not greater than 0.6 the full permissible value, or
2. Full permissible shear stress when the bending stress is not more than $\frac{3}{4}$ of the maximum allowable.

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 $0.75F_b$.

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* Design of Hybrid Steel Beams, Report of Subcommittee 1 of the Joint ASCE-AASHO Committee on Flexural Members, *ASCE Journal of the Structural Division, June, 1968.*

** *Basler, K.* Strength of Plate Girders Under Combined Bending and Shear, *ASCE Journal of the Structural Division, October, 1961.*

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 and loading applied longitudinally over partial panel length 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* 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 over-conservative 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.

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* *Basler, K.* New Provisions for Plate Girder Design, *Appendix C, 1961 Proceedings AISC National Engineering Conference.*

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 here. The alternate provision, permitting a stress of $0.76F_y$, to be used in designs where a fully encased beam is proportioned to resist all loads unassisted, reflects a common engineering practice where it is desired to eliminate the calculation of composite section properties.

In keeping with the *Tentative Recommendations for the Design and Construction of Composite Beams and Girders for Buildings**, 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 both live and dead loads, even for unshored construction. This liberalization is based upon an ultimate strength concept, although the proportioning of the member is based upon the elastic section modulus of the transformed cross-section.

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, in tension at the bottom of the beam, for unshored construction, is limited to $(1.35 + 0.35\ M_L/M_D)$ times the section modulus of the bare beam.**

On the other hand, the requirement that flexural stress in the concrete slab, due to actual composite action, be computed on the basis of actual transformed section modulus 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 directly proportional to the shear resistance developed between the steel and concrete, i.e., the number of shear connectors provided between these limits. At times it may not be feasible, nor even necessary, to provide full composite action. Therefore the Specification recognizes two conditions: full and incomplete 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 the steel beam alone.

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* Progress Report of the Joint ASCE-ACI Committee on Composite Construction, *ASCE Journal of the Structural Division, December, 1960.*

** Ibid., Eq. (3).

† *Slutter, R. G. and Driscoll, G. C.* Flexural Strength of Steel-Concrete Composite Beams, *p. 91, ASCE Journal of the Structural Division, April, 1965.*

1.11.4 Shear Connectors

Based upon tests at Lehigh University,* and a re-examination of previously published test data reported by a number of investigators, more liberal working values are recommended for various types and sizes of shear connectors than in use prior to 1961.

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, either side of the point of maximum moment, be sufficient to develop the composite action counted upon at that point. The provisions of the Specification are based upon this concept of composite action.

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 spacing is required over a part of this distance.

Consider, for example, 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 (1.11-6) is provided as a check to determine whether the number of connectors, N_1, required to develop M_{max} would, if uniformly distributed, provide N_2 connectors between one of the concentrated loads and the nearest point of zero moment. It is based upon the requirement that

$$S_{eff}:S_{tr} = M:M_{max}$$

where

$0 < M < M_{max}$

S_{eff} = section modulus corresponding to the minimum amount of incomplete composite action required at the section subject to the moment M

$V'_h:V_h = N_2:N_1$

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, one-half of the ultimate tensile strength of the reinforcement.

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* *Slutter, R. G. and Driscoll, G. C.* Flexural Strength of Steel-Concrete Composite Beams, *p. 91, ASCE Journal of the Structural Division, April 1965.*

The working values for various types of shear connectors are based upon a factor of safety of approximately 2.50 against their demonstrated ultimate strength.

Working values for use with concrete having aggregate not conforming to ASTM C33 and for connector types other than those shown in Table 1.11.4 must be established by a suitable testing program.

The values of q in Table 1.11.4 must not be confused with shear connection values suitable for use when the required number is measured by the parameter VQ/I, where V is the total shear at any given cross-section. Such a misuse could result in providing less than half the number required by Formula (1.11-3), (1.11-4) or (1.11-5).

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.

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.

Obviously, the most satisfactory solution must rest upon the sound judgment of qualified engineers. As a guide, but only a guide, the following rules are suggested:

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.

The depth of fully stressed roof purlins should, if practicable, be not less than $F_y/1{,}000$ 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 the one hand, upon the frequency of the vibration and, on the other, the damping effect provided by components of the construction. 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 make to the total ponding deflection can be expressed* 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.

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 $C_p + 0.9C_s \leqslant 0.25$.

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* *Marino, F. J.* Ponding of Two-Way Roof Systems, *AISC Engineering Journal, July, 1966.*

Given any combination of primary and secondary framing, the stress index is computed as

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_p \text{, for the primary member}$$

$$U_s = \left(\frac{0.8F_y - f_o}{f_o}\right)_s \text{, for the secondary member}$$

where f_o, in each case, is the computed bending stress 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

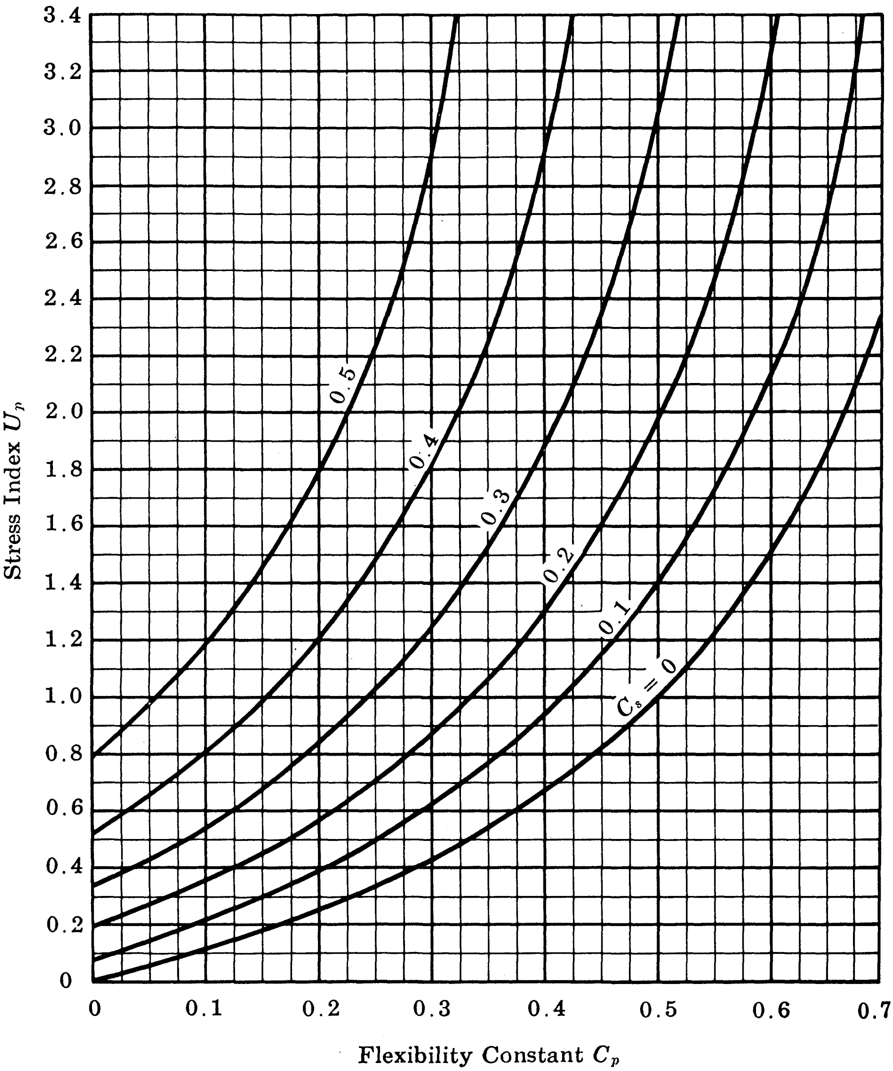

Fig. C1.13.3.1

during torrential summer rains, when the rate of precipitation exceeded the rate of drainage run-off 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.

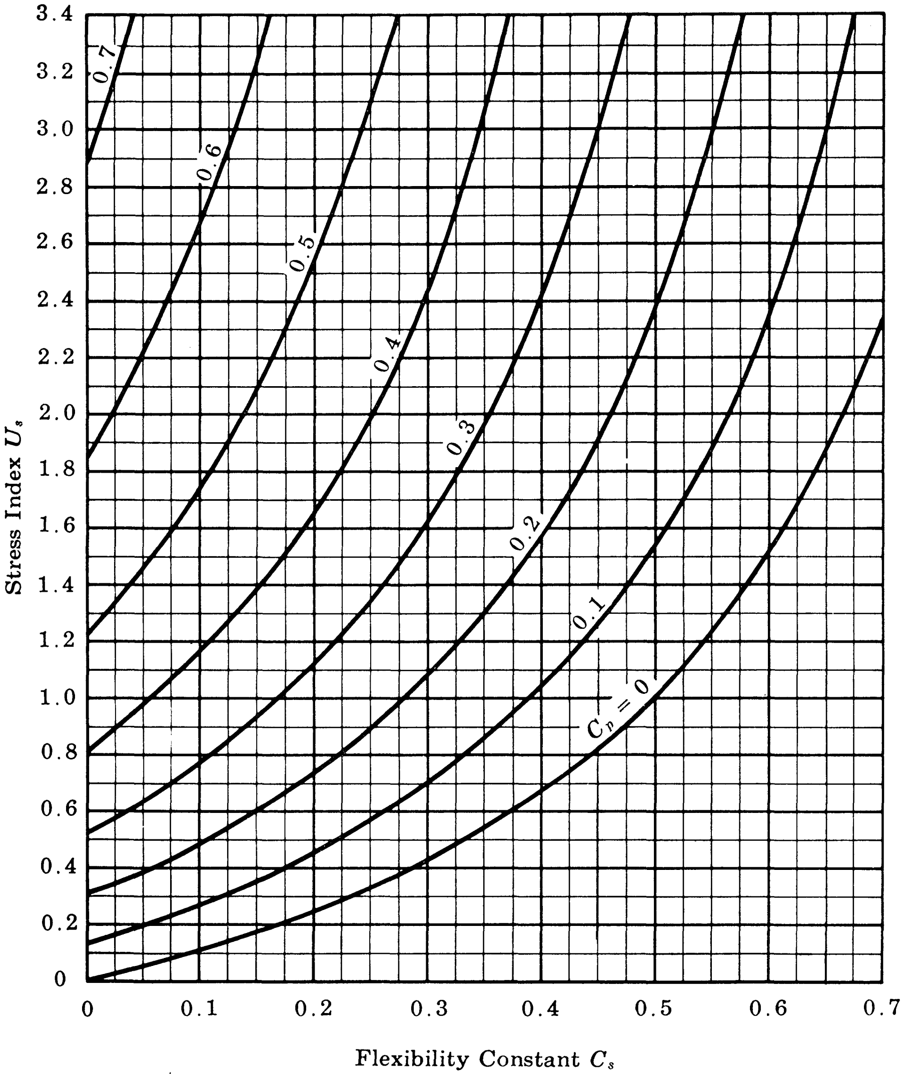

Fig. C1.13.3.2

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 SECTIONS

1.14.3 Net Section

Tests* have indicated that, as the ratio of net to gross section approaches unity, the ultimate tensile strength of a member may be less than the product of the net section multiplied by the tensile strength of the steel determined by standard coupon tests. A precise evaluation of this relationship would depend upon such parameters as hole spacing normal to the applied tension force versus thickness of section, and the ductility of the steel. Pending further investigation, the Specification places the upper limit of the fully effective net section at 85 percent of the gross section.

1.14.6 Pin-Connected Members

Forged eyebars have been replaced by pin-connected plates or eyebars flame-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 flame-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 is likewise based on the results of experimental research.**

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* *Schutz, F. W. and Newmark, N. M.* The Efficiency of Riveted Structural Joints, *Structural Research Series No. 30, University of Illinois.*

Fisher, J. W. Behavior of Fasteners and Plates With Holes, *ASCE Journal of the Structural Division, December, 1965.*

** *Johnston, B. G.* Pin Connected Plate Links, *1939 ASCE Transactions.*

Somewhat stockier proportions are provided for eyebars and pin-connected members fabricated from steel having a yield stress greater than 70 ksi, in order to eliminate any possibility of their "dishing" under the higher working stress for which they may be designed.

1.14.7 Effective Areas of Weld Metal

In recognition of the deeper penetration obtained by the submerged arc process, fillet welds made by this process may be proportioned on the basis of an effective throat thickness somewhat greater than the perpendicular distance from the root to the diagrammatic weld face. For fillet welds of such size as to require more than a single pass, the recognized increase in throat thickness is held constant.

Provision for the use of partial penetration groove welds, which first appeared in the 1961 AISC Specification, has been extended to cover their use on both sides of a joint, in keeping with similar provisions now included in the AWS Building Code.

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 strength of such members. Tests* have shown that similar practice is warranted in the case of welded members in statically loaded structures.

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. C1.15.5.1, depends upon the proportions of these members. Formulas

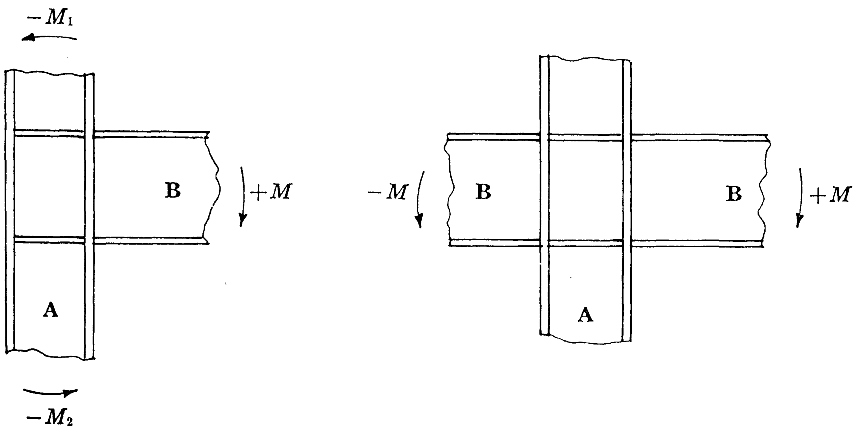

Fig. C 1.15.5.1

* *Gibson, G. T. and Wake, B. T.* An Investigation of Welded Connections for Angle Tension Members, *The Welding Journal, January 1942, American Welding Society.*

(1.15-1) and (1.15-3) are based on tests* 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 and equal to the area of the rigidly connected flange times its yield stress.

Formula (1.15-4), giving the required area of stiffeners when stiffeners are needed, is based upon the same concept.

Formula (1.15-2) limits the slenderness ratio of an unstiffened web of the supporting member, in order to avoid possibility of its buckling.

Since these provisions are based upon the maximum force that can be delivered by the supported member flanges, they obviously would be conservative in the case of less rigidly connected members.

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 joint 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 these 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 in new work. 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, provided the welds are made after the bolts have been tightened.

In making alterations to existing structures it is assumed that whatever slip is likely to occur in riveted joints or high strength bolted, bearing-type joints will have already taken place. Hence, in such cases the use of welding to resist all contemplated stresses in addition to 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

Earlier reference to A354 Grade BC bolts has been deleted since the Specification of the Research Council on Riveted and Bolted Structural Joints has been revised to include A490 bolts, which are better suited and more readily available. At the same time, provision for the use of A449 bolts, in lieu of A325 bolts, has been added. These bolts differ from A325 bolts only as to the size of head and conform to the high strength bolts originally called for in the Council's Specification when the use of hardened washers under head and nut was mandatory.

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* *Graham, J. D., Sherbourne, A. N., Knabbaz, R. N. and Jensen, C. D.* Welded Interior Beam-to-Column Connections, *American Institute of Steel Construction.*

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* 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 Pitch

The recommendations for minimum pitch in the spacing of rivets and bolts is dictated solely by the need for driving or wrenching clearance during the installation of these fasteners.

1.16.6 Minimum Edge Distance in Line of Stress

The requirements of this section have been revised to provide greater flexibility in their application to various combinations of fastener hardness and yield stress in the connected parts. The earlier provisions, covering the use of A502 Grade 1 rivets in mild carbon steel, have been retained as the basic concept.

SECTION 1.17 WELDS

1.17.2 Qualification of Weld and Joint Details

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 for Welding in Building Construction*, D1.0-69. 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 AWS D1.0-69.

SECTION 1.18 BUILT-UP MEMBERS

Requirements dealing with the 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 mem-

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* *Bendigo, R. A., Hansen, R. M. and Rumpf, J. L.* Long Bolted Joints, *ASCE Journal of the Structural Division, December, 1963.*

bers, 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.*

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 1.23 FABRICATION

1.23.1 Straightening Material

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 for such straightening is 1100°F for A514 steel, as contrasted with 1200°F for other steels.

1.23.5 Riveted and High Strength Bolted Construction Assembling

Even when used in bearing-type shear connections, high strength bolts are required to be tightened to their proof load in the case of A325 and A449 bolts, and to 0.7 of their tensile strength in the case of A490 bolts.

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* *Stang, A. H. and Jaffe, B. S.* Perforated Cover Plates for Steel Columns, *Research Paper RP1861, National Bureau of Standards.*

This may be done either by the turn-of-nut method* or by a calibrated wrench. 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.

1.23.6 Welded Construction

Inclusion of a number of grades of steel in the Specification has created the need for a greater control of preheat and interpass temperature in welding. The rules given reflect present practices as indicated by the standards of the American Welding Society.

SECTION 1.24 SHOP PAINTING

The shop painting of structural steel not to be encased in concrete is not mandatory. Steelwork to be covered up by the building finish will be shop painted only if required by the plans and job specification. 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.**

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 1.26 QUALITY CONTROL

Starting at the producing mill, and continuing in the fabricator's plant, steel required to have a yield stress in excess of 36 kips per square inch must at all times be so marked as to identify the ASTM specification and grade to which it conforms.

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* *See* Specification for Structural Joints Using ASTM A325 Or A490 Bolts *Research Council on Riveted and Bolted Structural Joints.*

** *Bigos, J., Smith, G. W., Ball, E. F. and Foehl, P. J.* Shop Paint and Painting Practice, *1954 Proceedings AISC National Engineering Conference.*

† *For a comprehensive treatment of the subject, see* Systems and Specifications, Steel Structures Painting Manual, Volume 2, *published by the Steel Structures Painting Council, Pittsburgh, Pa.*

SECTION 2.1 SCOPE

When provisions for plastic design were first introduced into the AISC Specification in 1961, their use was limited to one- and two-story rigid frames. However, as noted in the *Commentary* at that time, they were not ruled out in the case of beam design for multi-story buildings if resistance to lateral forces applied to the building was provided by means other than the bending stiffness of these beams.

The bending strength of a compact flexural member is greater than its strength at initial yielding, in an amount measured by the shape factor f of its profile; a non-compact member (meeting the provisions of Sect. 1.9, but not those of Sect. 2.7), usually has little reserve strength beyond the elastic limit, because of buckling. Hence, for such members it may be said that the effective shape factor is 1.0.

The superior bending strength of compact sections is recognized in Part 1 of the Specification by increasing the allowable bending stress to $0.66F_y$. By the same token, the logical load factor for plastically designed beams is given by the equation $F = \dfrac{F_y}{0.66F_y} \cdot (f)$. For such shapes listed in the AISC *Steel Construction Manual*, the variation of (f) 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. While a load factor of 1.7, comparable to the basic 5/3 factor of safety inherent in working stress design, was specified for beams, the recommended load factor for frames as a whole was 1.85, pending further investigation of columns and frame stability problems.

Research which has been completed since 1961* has provided a better understanding of the ultimate strength of heavily loaded columns subjected to concurrent bending moments. Based upon this information, the load factor of frames has been made the same as that provided for members subject only to bending. Consistent with this change, the load factor to be used in designing for gravity loading combined with wind or seismic loading has been reduced from 1.4 to 1.3.

Based on continuing research at Lehigh University on multi-story framing,** application of the Specification provisions has been extended to include the complete design of planar frames in high-rise buildings, provided they are braced to take care of any lateral loading. Systematic procedures for application of plastic design in proportioning the members of such frames have been developed† and are available in the current literature.

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* *Van Kuren, R. C. and Galambos, T. V.* Beam Column Experiments, *ASCE Journal of the Structural Division, April, 1964.*

** *Driscoll, G. C. et al.* Plastic Design of Multi-Story Frames—Lecture Notes, *Fritz Engineering Laboratory Report No. 273.20, Lehigh University, August, 1965.*

Driscoll, G. C. Lehigh Conference on Plastic Design of Multi-Story Frames—A Summary, *AISC Engineering Journal, April, 1966.*

† Plastic Design of Braced Multi-Story Steel Frames, *American Iron and Steel Institute, 1968.*

Lu, Le-Wu Design of Braced Multi-Story Frames by the Plastic Method, *AISC Engineering Journal, January, 1967.*

SECTION 2.2 STRUCTURAL STEEL

The 1961 AISC Specification limited the use of plastic design to steels having a specified minimum yield point no higher than 36 ksi. Most of the experimental verification of provisions for plastic design contained in the Specification at that time had used steel of about this strength.

By 1965 the applicability of such provisions, with only minor modifications, to high-strength low-alloy steel furnished to a specified yield point of 50 ksi, had been established.* With the advent of ASTM Specification A572 in 1966, further investigation was undertaken which indicated their applicability for all grades covered by that standard.**

On the basis of these investigations, the list of steels covered by ASTM standard specifications has been increased accordingly.

SECTION 2.3 VERTICAL BRACING SYSTEM

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.

In moment resisting frames, designed in accordance with the provisions of Part 1 of the Specification, the necessary resistance to lateral loading is provided by the bending strength of the beams and columns rigidly connected to one another. Distribution of bending moments is based upon an assumption of completely elastic frame behavior; column strength is based upon an effective unbraced length generally greater than the actual unbraced length.

Neither of these assumptions apply in the analysis of unbraced, plastically designed high-rise frames, although appropriate analytical procedures have been proposed.† Pending further study, design of such framing more than two stories in height, in accordance with the provisions of Part 2 of the Specification, is restricted to fully-braced systems. The role and requirements of such systems‡ are defined by the provisions of Sect. 2.3.

The limitation on axial force of $0.85P_y$ is inserted as a simple means of compensating for three possible effects:¶

a) Loss of stiffness due to residual stress
b) Effect of secondary moments from the vertical bracing system
c) Lateral torsional buckling effect

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* *Adams, P. F., Lay, M. G. and Galambos, T. V.* Experiments on High Strength Steel Members, *Welding Research Council Bulletin No. 110.*

** Plastic Design in Steel, *ASCE Manual of Engineering Practice No. 41, Second Edition, Section 5.1.*

† *Driscoll, G. C. et al.* Plastic Design of Multi-Story Frames—Lecture Notes, *Chapter 14, Fritz Engineering Laboratory Report No. 273.20, Lehigh University, August, 1965.*

‡ *Lu, Le-Wu* Design of Braced Multi-Story Frames by the Plastic Method, *AISC Engineering Journal, January, 1967.*

¶ Plastic Design in Steel, *ASCE Manual of Engineering Practice No 41, Second Edition, Chapter 10.*

SECTION 2.4 COLUMNS

Based on research completed since the previous edition of the Specification, provisions for design of beam-columns have been extensively revised.

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 the larger slenderness ratio for any given unbraced length.**

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. For limiting values of l/r_y applicable to various yield stress steels and end moment ratios, see Sect. 2.9 in Appendix A.

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 $M_1/M_2 = -1.0$. 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 C 2.4.1. In each case l is the distance between points of lateral support corresponding to r_x or r_y, as

TABLE C 2.4.1

	Braced Planar Frames	One- and Two-Story Unbraced Planar Frames
P_{cr}	Use larger ratio, $\frac{l}{r_y}$ or $\frac{l}{r_x}$	[1]Use larger ratio, $\frac{l}{r_y}$ or $\frac{Kl}{r_x}$
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.

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* *Driscoll, G. C. et al.* Plastic Design of Multi-Story Frames—Lecture Notes, *Eq. (4.6) and Eq. (4.7), Fritz Engineering Laboratory Report No. 273.20, Lehigh University, August, 1965.*

** *Ibid.*, p. 4.24.

† *Ibid.*, p. 4.26

applicable. When K is indicated, its value is governed by the provisions of Sect. 1.8.3 of the Specification. Elsewhere, $Kl/r = l/r$.

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 (approx. 0.95 times its actual depth). Thus

$$V_u = \frac{0.95F_y}{\sqrt{3}}\, dt = 0.55F_y\, dt$$

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. C 2.5.1, expressed in kip-feet, to be resisted by a couple of forces 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.

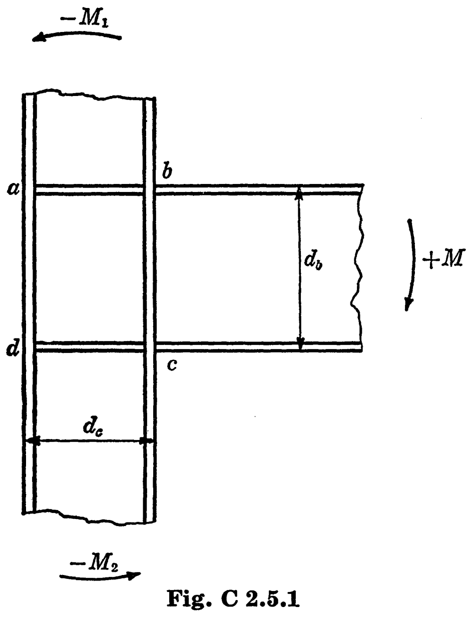

Fig. C 2.5.1

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* Plastic Design in Steel, *ASCE Manual of Engineering Practice No. 41, Second Edition, Section 6.1.*

If the thickness of the web panel is less than that given by this formula, the deficiency may be compensated 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.

SECTION 2.6 WEB CRIPPLING

Usually stiffeners are needed, as at *ab* and *dc* in Fig. C 2.5.1, 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), (1.15-3) and (1.15-4) 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. C 2.6.1 may prove advantageous.

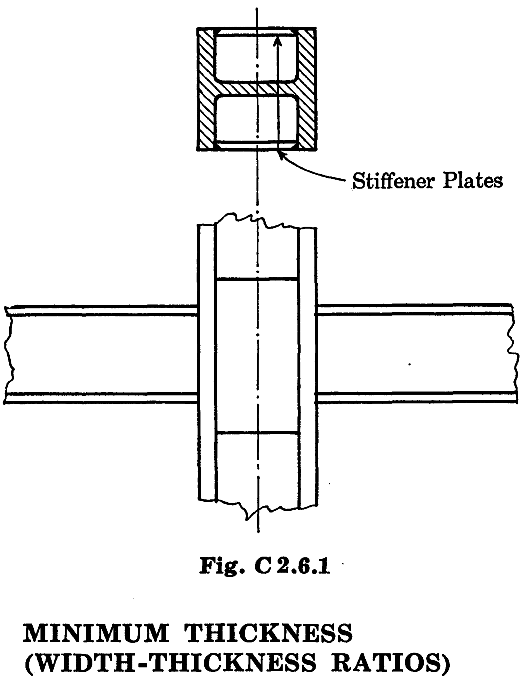

Fig. C 2.6.1

SECTION 2.7 MINIMUM THICKNESS (WIDTH-THICKNESS RATIOS)

In extending the provisions for plastic design to steels having a yield point higher than 36 ksi, considerable research* has been required in order to

* Plastic Design in Steel, *ASCE Manual of Engineering Practice No. 41, Second Edition, Section 6.2.*

define 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.

These studies have shown that the limiting width-thickness ratio is 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 is involved in extending the earlier expression for limiting web depth-thickness ratio to stronger steels. Formulas (2.7-1a) and (2.7-1b) are 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, except that it is written in terms of factored loads instead of allowable stresses at service loading.

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 of the Specification.

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** 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

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* Plastic Design in Steel, *ASCE Manual of Engineering Practice No. 41, Second Edition, Chapter 8.*

** *Douty, R. T. and McGuire, W.* High Strength Bolted Moment Connections, *ASCE Journal of the Structural Division, April, 1965.*

part. It has also been demonstrated* 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 I-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.

For the limited range of steels recognized as suitable for plastic design in earlier editions of the Specification, l_{cr}, the allowable unbraced length of compression flanges subject to plastic bending, was given as

$$\left(60 - 40\frac{M}{M_p}\right) r_y > l_{cr} \geqslant 35r_y$$

where M/M_p, the ratio of end moments, was considered positive only when the unbraced length was bent in *single* curvature.

Based on research seeking to extend the application of plastic design to stronger steels, it was noted** that this expression could be unduly conservative in the region where $-0.5 < M/M_p < 0$.† The new provision reflects this and also includes a more conservative approach in the region where $0 < M/M_p < +1.0$.†

Both Formulas (2.9-1a) and (2.9-1b) are empirical expressions which closely approximate the suggested revisions.‡

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* *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.*

** *Lay, M. G. and Galambos, T. V.* Inelastic Beams Under Moment Gradient, *ASCE Journal of the Structural Division, February, 1967, p. 390.*

† 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) and used here 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.

‡ Plastic Design in Steel, *ASCE Manual of Engineering Practice No. 41, Second Edition, Section 6.3.*