SPECIFICATION

FOR THE

DESIGN,

FABRICATION

& ERECTION

OF

STRUCTURAL

STEEL FOR

BUILDINGS

(ADOPTED NOVEMBER 30, 1961)

AMERICAN INSTITUTE
OF STEEL CONSTRUCTION
101 PARK AVENUE, NEW YORK 17, N.Y.

Preface

This Specification formulates rules for use in the design, fabrication and erection of carbon and high strength constructional steels for structural purposes in buildings and structures other than bridges. Allowable stress design provisions are included in Part 1; plastic design rules are given in Part 2. In the Appendix, which constitutes an integral part of the Specification, are tabulated the numerical values for algebraic expressions given in Parts 1 and 2, applicable to steels of different strength levels.

In the preparation of the Specification, the Committee has studied available results of recent research and earlier editions of the AISC Specification. Based upon these studies, a considerable number of sections have been revised and new provisions added.

As used throughout the Specification, the term "structural steel" refers exclusively to those items enumerated in Section 2 of the *Code of Standard Practice for Steel Buildings and Bridges* of the American Institute of Steel Construction, and nothing herein contained is intended as a recommended practice for members formed of flat rolled sheet or strip, light-gage steel construction, skylights, fire escapes, or other items not specifically enumerated in that Code.

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Nomenclature

A_b Nominal body area of a bolt

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_{st} Cross-sectional area of stiffener or pair of stiffeners

A_w Area of girder web

B Coefficient used in column formula for plastic design

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_v Ratio of "critical" web stress, according to the linear buckling theory, to the shear yield point of web material; equal to

$$\frac{\pi^2 E k \sqrt{3}}{12(1 - \nu^2)(h/t)^2 F_y}$$

D Factor depending upon type of transverse stiffeners

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

E_c Modulus of elasticity of concrete

F_a Axial compressive stress permitted in the absence of bending stress

F_{as} Axial compressive stress, permitted in the absence of bending stress, for bracing and other secondary members

F_b Bending stress permitted in the absence of axial stress

F'_b Allowable bending stress in compression flange of plate girders as reduced because of large web depth-to-thickness ratio

F'_e Euler stress divided by factor of safety; equal to

$$\frac{149,000,000}{(l/r)^2}$$

F_p Allowable bearing stress

F_t Allowable tensile stress

F_v Allowable shear stress

F_y Specified minimum yield point of the type of steel being used

G Coefficient used in column formula in plastic design

I_{tr} Moment of inertia of transformed composite section

J Coefficient used in column formula in plastic design

K Coefficient used in column formula in plastic design

L Span length, in feet

M Moment

M_1 Smaller end moment on unbraced length of beam-column

M_2 Larger end moment on unbraced length of beam-column

M_D Moment produced by dead load

M_L Moment produced by live load

M_o Reduced plastic moment

M_p Plastic moment

N Length of bearing of applied load

P Applied load

P_y Plastic axial load; equal to profile area times specified minimum yield point

R Reaction or concentrated transverse load applied to beam or girder

S_s Section modulus of steel beam used in composite design, referred to the tension flange

S_{tr} Section modulus of transformed composite cross-section, referred to the tension flange

T_b Proof load of a high strength bolt

V Statical shear on beam

V_h Total horizontal shear to be resisted by connectors

V_u Statical shear produced by "ultimate" load in plastic design

Y Ratio of yield point of web steel to yield point of stiffener steel

a Center-to-center distance between transverse stiffeners

a' Distance required at ends of welded partial length cover plate to develop stress

b Effective width of concrete slab

c Distance from neutral axis to top of concrete slab

d Depth of beam or girder. Also diameter of roller or rocker bearing

e Horizontal displacement, in the direction of the span, between top and bottom of simply supported beam at its ends

f_a Computed axial stress

f_b Computed bending stress

f'_c Specified compression strength of concrete at 28 days

f_t Computed tensile stress

f_v Computed shear stress, in pounds per square inch

f_{vs} Shear between girder web and transverse stiffeners, in pounds 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

l Actual unbraced length, in inches. Also effective unbraced length

l_{cr} Critical unbraced length adjacent to plastic hinge, in inches

n Modular ratio; equal to E/E_c

q Allowable horizontal shear to be resisted by a connector

r Governing radius of gyration

r_b Radius of gyration about axis of concurrent bending

r_y Lesser radius of gyration

5

s Spacing (pitch) between successive holes in line of stress

t Girder or beam web thickness

t_f Flange thickness

t_t Thickness of thinner part joined by partial penetration groove weld

w Web thickness of plastically designed rolled beams.   Also length of channel shear connectors

ν Poisson's ratio

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 locked-up stresses and distortion.

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

1.1.4   Standard Symbols and Nomenclature

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

Unless otherwise noted, the standard nomenclature contained in the joint AISC-SJI *Standard Specifications for Open Web Steel Joists—Longspan or L-Series* shall be used in describing longspan 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 "conventional" or "simple" framing (unrestrained, free-ended), assumes that the ends of beams and girders are connected for shear only, and are free to rotate under 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 complete rigidity of Type 1 and the complete 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. Beam-to-column connections with seats for the reactions and with top clip angles for lateral support only are classed under Type 2.

In tier buildings, designed in general as Type 2 construction (that is, with beam-to-column connections other than wind connections flexible) the distribution of the wind moments between the several joints of the frame may be made by a recognized empirical method provided that either:

1. The wind connections, designed to resist the assumed moments, are adequate to resist the moments induced by the gravity loading and the wind loading at the increased unit stresses permitted therefor, or
2. The wind connections, if welded and if designed to resist the assumed wind moments, are so designed that larger moments induced by the

gravity loading under the actual condition of restraint will be relieved by deformation of the connection material without over-stress in the welds.

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 the 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), applied at the top of 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. The wind pressure is dependent upon the conditions of exposure, geographical location, and shape of the structure.

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 *American Standard Building Code Requirements for Minimum Design Loads in Buildings and Other Structures* ASA A58.1, latest edition.

SECTION 1.4   MATERIAL

1.4.1 Structural Steel

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

> *Steel for Bridges and Buildings*, ASTM A7
> *Structural Steel for Welding*, ASTM A373
> *Structural Steel*, ASTM A36
> *High-Strength Structural Steel*, ASTM A440
> *High-Strength Low-Alloy Structural Manganese Vanadium Steel*, ASTM A441
> *High-Strength Low-Alloy Structural Steel*, ASTM A242

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

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

1.4.3 Rivet Steel

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

> *Structural Rivet Steel*, ASTM A141
> *High-Strength Structural Rivet Steel*, ASTM A195
> *High-Strength Structural Alloy Rivet Steel*, ASTM A406

Certified mill test reports 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 Steel Bolts for Structural Joints*, ASTM A325
> *Quenched and Tempered Alloy Steel Bolts and Studs with Suitable Nuts*, ASTM A354, Grade BC

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 E60 or E70 series of the *Specification for Mild Steel Arc-Welding Electrodes*, ASTM A233, latest edition.

Bare electrodes and granular flux used in the submerged-arc process shall conform to the provisions of Sect. 1.17.3.

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

SECTION 1.5   ALLOWABLE UNIT STRESSES*

Except as provided in Sect. 1.6, 1.7, 1.10, 1.11 and in Part 2, all components of the structure shall be so proportioned that the unit stress, in pounds per square inch, shall not exceed the following values, except as they are rounded off in the Appendix:

1.5.1   Structural Steel

1.5.1.1   Tension

On the net section, except at pin holes

$$F_t = 0.60 F_y$$

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

$$F_t = 0.45 F_y$$

1.5.1.2   Shear

On the gross section of beam and plate girder webs

$$F_v = 0.40 F_y$$

(See Sect. 1.10 for reduction required for thin webs.)

1.5.1.3   Compression

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

$$F_a = \frac{\left[1 - \dfrac{(l/r)^2}{2C_c{}^2}\right] F_y}{\text{F.S.}} \qquad \text{Formula (1)}$$

where

$$\text{F.S.} = \text{factor of safety} = \frac{5}{3} + \frac{3(l/r)}{8C_c} - \frac{(l/r)^3}{8C_c{}^3}$$

and

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

1.5.1.3.2   On the gross section of axially loaded columns when l/r exceeds C_c

$$F_a = \frac{149,000,000}{(l/r)^2} \qquad \text{Formula (2)}$$

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

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

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

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 rolled shapes and built-up members having an axis of symmetry in the plane of loading and proportions meeting the requirements of Sect. 2.6, when the member is supported laterally at intervals no greater than 13 times its compression flange width

$$F_b = 0.66F_y$$

Beams and girders which meet the requirements of the preceding paragraph and are continuous over supports or are rigidly framed to columns by means of rivets, high strength bolts or welds, may be proportioned for $9/10$ of the negative moments produced by gravity loading which are maximum at points of support, provided that, for such members, the maximum positive moment shall be increased by $1/10$ of the average negative moments.   This reduction shall not apply to moments produced by loading on cantilevers.   If the negative moment is resisted by a column rigidly framed to the beam or girder, the $1/10$ reduction may be used in proportioning the column for the combined axial and bending loading, provided that the unit stress, due to any concurrent axial load on the member, does not exceed $0.15F_y$.

1.5.1.4.2   Tension and compression on extreme fibers of unsymmetrical members supported as in Sect. 1.5.1.4.1 in the region of compression stress

$$F_b = 0.60F_y$$

1.5.1.4.3   Tension and compression on extreme fibers of box-type members whose proportions do not meet the provisions of Sect. 2.6, but do conform to the provisions of Sect. 1.9

$$F_b = 0.60F_y$$

1.5.1.4.4   Tension on extreme fibers of other rolled shapes, built-up members and plate girders

$$F_b = 0.60F_y$$

1.5.1.4.5   Compression on extreme fibers of rolled shapes, plate girders and built-up members having an axis of symmetry in the plane of their web (other than box-type beams and girders), the larger value computed by Formulas (4) and (5), but not more than $0.60F_y$

$$F_b = \left[1.0 - \frac{(l/r)^2}{2C_c^2 C_b} \right] 0.60F_y* \qquad \text{Formula (4)}$$

$$F_b = \frac{12,000,000}{ld/A_f} \qquad \text{Formula (5)}$$

where l is the unbraced length of the compression flange; r is the radius of gyration of a tee section comprising the compression flange plus one-sixth of the web area, about an axis in the plane of the web; A_f is the area of the compression flange; C_c is defined in Sect. 1.5.1.3 and C_b, which can conservatively be taken as unity, is equal to

$$C_b = 1.75 - 1.05 \left(\frac{M_1}{M_2} \right) + 0.3 \left(\frac{M_1}{M_2} \right)^2, \text{ but not more than } 2.3$$

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 (single curvature bending) and negative when they are of opposite signs, (reverse curvature bending).   When the bending moment at any point within an unbraced length is larger than that at both ends of this length the ratio M_1/M_2 shall be taken as unity.   See Sect. 1.10 for further limitation in plate girder flange stress.

1.5.1.4.6   Compression on extreme fibers of channels, the value computed by Formula (5), but not more than

$$F_b = 0.60F_y$$

1.5.1.4.7   Tension and compression on extreme fibers of large pins

$$F_b = 0.90F_y$$

1.5.1.4.8   Tension and compression on extreme fibers of rectangular bearing plates

$$F_b = 0.75F_y$$

1.5.1.5   Bearing   (on contact area)

1.5.1.5.1   Milled surfaces and pins in reamed, drilled or bored holes, pounds per square inch

$$F_p = 0.90F_y**$$

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* Where l/r is less than 40, stress reduction according to Formula (4) may be neglected.

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

1.5.1.5.2   Finished stiffeners, pounds per square inch
$$F_p = 0.80F_y*$$
1.5.1.5.3   Expansion rollers and rockers, pounds per linear inch
$$F_p = \left(\frac{F_y* - 13,000}{20,000}\right)660d$$
where d is the diameter of roller or rocker in inches.

1.5.2   Rivets and Bolts

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

TABLE 1.5.2.1

Description of Fastener	Tension (F_t)	Shear (F_v)	
		Friction-Type Connections	Bearing-Type Connections
A141 hot-driven rivets	20,000		15,000
A195 and A406 hot-driven rivets	27,000		20,000
A307 bolts and threaded parts of A7 and A373 steel	14,000		10,000
Threaded parts of other steels	$0.40F_y$		$0.30F_y$
A325 bolts when threading is *not* excluded from shear planes	40,000	15,000	15,000
A325 bolts when threading is excluded from shear planes	40,000	15,000	22,000
A354, Grade BC, bolts when threading is *not* excluded from shear planes	50,000	20,000	20,000
A354, Grade BC, when threading is excluded from shear planes	50,000	20,000	24,000

1.5.2.2   Allowable bearing stress on projected area of bolts in bearing-type connections and on rivets
$$F_p = 1.35F_y$$
(Bearing stress not restricted in friction-type connections assembled with A325 and A354, Grade BC, bolts.)

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

1.5.3   Welds (stress in pounds per square inch of throat area)

1.5.3.1   Fillet, Plug, Slot and Partial Penetration Groove Welds

Fillet, plug, slot and partial penetration groove welds made with A233 Class E60 series electrodes and fillet welds made by submerged arc welding Grade SA-1   13,600

Fillet, plug, slot and partial penetration groove welds made with A233 Class E70 series electrodes and fillet welds made by submerged arc welding Grade SA-2   15,800

1.5.3.2   Complete Penetration Groove Welds

On complete penetration groove welds the allowable tension, compression, bending, shear and bearing stresses shall be the same as those allowed by Sect. 1.5 in the connected material.   (See Sect. 1.17.2 for electrodes to be employed on various grades of steel.)

1.5.4   Cast Steel and Steel Forgings

1.5.4.1   Tension (on net section)

$$F_t = 0.60F_y$$

1.5.4.2   Shear (on gross section)

$$F_v = 0.40F_y$$

1.5.4.3   Compression

Same as provided under Sect. 1.5.1.3

1.5.4.4   Bending (on extreme fibers)

$$F_b = 0.60F_y$$

1.5.4.5   Bearing

Same as provided under Sect. 1.5.1.5

1.5.5   Masonry Bearing

In the absence of Code regulations the following unit stresses in pounds per square inch shall apply:

On sandstone and limestone . . . . $F_p = 400$
On brick in cement mortar . . . . $F_p = 250$
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 at 28 days.

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, nor less than that required by Sect. 1.7, if it is applicable.

SECTION 1.6   COMBINED STRESSES

1.6.1   Axial Compression and Bending

Members subject to both axial compression and bending stresses shall be proportioned to meet the requirements of both Formula (6) and Formula (7).

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \dfrac{f_a}{F'_e}\right)F_b} \leq 1.0 \qquad \text{Formula (6)}$$

$$\frac{f_a}{0.6F_y} + \frac{f_b}{F_b} \leq 1.0 \ \text{(applicable only at braced points)} \qquad \text{Formula (7)}$$

where

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

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

$F'_e = \dfrac{149,000,000}{(l/r_b)^2}$ (May be increased one-third in accordance with Sect. 1.5.6)

l = actual unbraced length in the plane of bending

r_b = radius of gyration about axis of bending

f_a = computed axial stress

f_b = computed bending stress at the point under consideration

C_m = 0.85, except as follows:

1. When $f_a/F_a \leq 0.15$. (For this case the member selected shall meet the limitation that $f_a/F_a + f_b/F_b \leq 1.0$.)

2. For restrained compression members in frames braced against joint translation but not subject to transverse loading between their supports in the plane of loading, C_m may be taken as $0.6 + 0.4(M_1/M_2)$, where M_1/M_2 is the ratio of smaller to larger moments at the ends of the critical unbraced length of the member. M_1/M_2 is positive when the unbraced length is bent in single curvature and negative when it is bent in reverse curvature.

3. For restrained compression members in frames braced against joint translation in the plane of loading and subject to transverse loading between their supports (joints) in the plane of loading, a value of C_m may be determined by rational analysis.

1.6.2 Shear and Tension

Rivets and bolts subject to combined shear and tension due to force applied to the connected parts, shall be so proportioned that the tension stress produced by the force shall not exceed the following:

For A141 rivets . . . . $F_t = 28,000 - 1.6f_v \leq 20,000$

For A195 and A406 rivets . . . . $F_t = 38,000 - 1.6f_v \leq 27,000$

For A307 bolts . . . . $F_t = 20,000 - 1.6f_v \leq 14,000$

For A325 bolts in bearing-type joints . . . . $F_t = 50,000 - 1.6f_v \leq 40,000$

For A354, Grade BC, bolts in bearing-type joints . . . . $F_t = 60,000 - 1.6f_v \leq 50,000$

where f_v, the shear stress produced by the same force, 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 bolts . . . . $F_v \leq 15,000 \ (1 - f_t A_b/T_b)$

For A354, Grade BC, bolts . . . . . $F_v \leq 20,000 \ (1 - f_t A_b/T_b)$

where f_t is the tensile stress due to applied load and T_b is the proof load of the bolt.

SECTION 1.7 MEMBERS AND CONNECTIONS SUBJECT TO REPEATED VARIATION OF STRESS

1.7.1 Up to 10,000 Complete Stress Reversals

The stress carrying area of members, connection material and fasteners* need not be increased because of repeated variation or reversal of stress unless the maximum stress allowed by Sect. 1.5 and 1.6 is expected to occur over 10,000† times in the life of the structure.

1.7.2 10,000 to 100,000 Cycles of Maximum Load

Members, connection material and fasteners (except high strength bolts in friction-type joints) subject to more than 10,000 but not over 100,000†† applications of maximum design loading shall be proportioned, at unit stresses allowed in Sect. 1.5 and 1.6 for the kind of steel and fasteners used, to support the algebraic difference** of the maximum computed stress and two-thirds of the minimum computed stress, but the stress-carrying area shall not be less than that required in proportioning the member, connection material and fasteners to support either the maximum or minimum computed stress at the values allowed in Sect. 1.5 and 1.6 for the kind of steel and fasteners used.

1.7.3 100,000 to 2,000,000 Cycles of Maximum Load

Members, connection material and fasteners (except high strength bolts in friction-type joints) subject to more than 100,000 but not more than

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* As used in this Section, "fasteners" comprise welds, rivets and bolts.

** In determining the algebraic difference, tensile stress is designated as positive and compression stress as negative.

† Approximately equivalent to one application per day for 25 years.

†† Approximately equivalent to ten applications per day for 25 years.

2,000,000† applications of maximum design loading shall be proportioned at unit stresses allowed in Sect. 1.5 and 1.6 for A7 steel, A141 rivet steel and E60XX and submerged arc Grade SA-1 welds to support the algebraic difference of the maximum computed stress and $\frac{2}{3}$ of the minimum computed stress, but the stress-carrying area shall not be less than that required in proportioning the member, connection material and fasteners to support either the maximum or minimum computed stress at the values allowed in Sect. 1.5 and 1.6 for the kind of steel and fasteners used.

1.7.4 Over 2,000,000 Cycles of Maximum Load

Members, connection material and fasteners (except high strength bolts in friction-type joints) subject to more than 2,000,000 applications of maximum design loading shall be proportioned at two-thirds of the unit stress allowed in Sect. 1.5 and 1.6 for A7 steel, A141 rivet steel and E60XX and submerged arc Grade SA-1 welds to support the algebraic difference of the maximum computed stress and three-quarters of the minimum computed stress, but the stress-carrying area shall not be less than that required in proportioning the member, connection material and fastener to support either the maximum or minimum computed stress at the values allowed in Sect. 1.5 and 1.6 for the kind of steel and fasteners used.

1.7.5 Details

Members subject to the provisions of Sect. 1.7.2, 1.7.3 and 1.7.4 shall have no sharp notches, sharp copes or attachments of clips, brackets or similar details, at locations where the stress exceeds 75 percent of those allowed in this section.

1.7.6 High Strength Bolted Connections

High strength bolts in friction-type joints shall be proportioned at the unit stresses allowed in Sect. 1.5.2 and 1.6.2 to resist the largest static stress on the joint produced by any single application of the design loads.

SECTION 1.8 SLENDERNESS RATIOS

1.8.1 Definition

In determining the slenderness ratio of an axially loaded compression member, l shall be taken as its effective length and r the corresponding radius of gyration.

1.8.2 Sidesway Prevented

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

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† Approximately equivalent to 200 applications per day for 25 years.

1.8.3 Sidesway Not Prevented

The effective length of compression members in a frame which depends upon its own bending stiffness for lateral stability, shall be determined by a rational method and shall not be less than the actual unbraced length.

1.8.4 Maximum Ratios

The slenderness ratio of compression members shall not exceed 200.

The slenderness ratio 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 Projecting Elements Under Compression

Projecting elements of members subjected to axial compression or compression due to bending shall have ratios of width-to-thickness not greater than the following:

Single-angle struts; double-angle struts with separators. . $2,400/\sqrt{F_y}$
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 . . . . $3,000/\sqrt{F_y}$
Stems of tees . . . . $4,000/\sqrt{F_y}$

The width of plates shall be taken from the free edge to the first row of rivets, bolts or welds; the width of legs of angles, channels and zees, and of the stems of tees, shall be taken as the full nominal dimension; the width of flanges of beams 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.

When a projecting element exceeds the width-to-thickness ratio prescribed in the preceding paragraph, but would conform to same and would satisfy the stress requirements with a portion of its width considered as removed, the member will be acceptable.

1.9.2 Compression Elements Supported Along Two Edges

In compression members the unsupported width of web, cover or diaphragm plates, between the nearest lines of fasteners or welds, or between the roots of the flanges in case of rolled sections, shall not exceed $8,000/\sqrt{F_y}$ times its thickness.

When the unsupported width exceeds this limit, but a portion of its width no greater than $8,000/\sqrt{F_y}$ times the thickness would satisfy the stress requirements, the member will be considered acceptable.

The unsupported width of cover plates perforated with a succession of access holes, may exceed $8,000/\sqrt{F_y}$, but shall not exceed $10,000/\sqrt{F_y}$, times the thickness. The gross width of the plate less the width of the widest access hole shall be assumed available to resist compression.

SECTION 1.10 PLATE GIRDERS AND ROLLED BEAMS

1.10.1 Proportions

Riveted and welded plate girders, cover-plated beams and rolled 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.

1.10.2 Web

The clear distance between flanges in inches, shall not exceed

$$\frac{14,000,000}{\sqrt{F_y(F_y + 16,500)}}$$

times the web thickness.

1.10.3 Flanges

The thickness of outstanding parts of flanges shall conform to the requirements of Sect. 1.9.

Each flange of welded plate girders shall in general consist of a single plate rather than two or more plates superimposed. The single plate may comprise a series of shorter plates, laid end-to-end and joined by complete penetration butt welds.

Unstiffened cover plates on riveted girders shall not extend more than $3,000/\sqrt{F_y}$ times the thickness of the thinnest outside plate beyond the outer row of rivets or bolts connecting them to the angles. 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 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 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 ¾ 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 1½ times the width of the cover plate when there is a continuous weld smaller than ¾ 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 by the provisions of Sect. 1.10.10, at points of concentrated loads. Such stiffeners shall have a close bearing against the flange, or flanges, through which they receive their loads or reactions, and shall extend approximately to the edge of the flange plates or flange angles. They shall be designed as columns subject to the provisions of Sect. 1.5.1, assuming the column section to comprise the pair of stiffeners and a centrally located strip of the web whose width is equal to not more than 25 times its thickness at interior stiffeners or 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 angle fillet or the flange-to-web welds shall be considered effective in bearing.

1.10.5.2 The largest average web shear f_v in any panel between stiffeners (total shear force divided by web cross-sectional area), in pounds per square inch, computed for any condition of complete or partial loading, shall not exceed the value given by Formula (8) or (9),** as applicable.

$$F_v = \frac{F_y}{2.89}\left[C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}}\right] \qquad \text{Formula (8)}$$

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* 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 values of F_v corresponding to various stiffener spacing see Tables 3 in the Appendix.

when C_v is less than 1.0;

$$F_v = \frac{F_y}{2.89}(C_v) \qquad \text{Formula (9)}$$

but not more than $0.4F_y$, when C_v is more than 1.0 or when intermediate stiffeners are omitted;

where

a = clear distance between transverse stiffeners, in inches
h = clear distance between flanges, in inches
$C_v = \dfrac{45,000,000k}{F_y(h/t)^2}$, when C_v is less than 0.8

$\quad = \dfrac{6,000}{h/t}\sqrt{\dfrac{k}{F_y}}$, when C_v is more than 0.8

t = thickness of web, in inches

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

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

When a/h is more than 3 its value shall be taken as infinity. In this case Formula (8) reduces to Formula (9) and $k = 5.34$.

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 (9).

The spacing of intermediate stiffeners, when stiffeners are required, shall be such that the web shear stress will not exceed the value for F_v given by Formulas (8) or (9), as applicable, but the smaller panel dimension, a or h, shall not exceed 260 times the web thickness and the ratio a/h shall not exceed $\left(\dfrac{260}{h/t}\right)^2$ nor 3.0.

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 $\dfrac{11,000t}{\sqrt{f_v}}$.

1.10.5.4 The gross area, in square inches, of intermediate stiffeners spaced in accordance with Formula (8) (*total* area, when stiffeners are furnished in pairs) shall be not less than that computed by Formula (10).

$$A_{st} = \frac{1 - C_v}{2}\left[\frac{a}{h} - \frac{(a/h)^2}{\sqrt{1 + (a/h)^2}}\right]YDht \qquad \text{Formula (10)}$$

where

C_v, a, h and t are as defined in Sect. 1.10.5.2

$$Y = \frac{\text{yield point of web steel}}{\text{yield point of stiffener steel}}$$

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 (8) this gross area requirement may be reduced in like proportion.

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

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.

Intermediate stiffeners required by the provisions of Sect. 1.10.5.3 shall be connected for a total shear transfer, in pounds per linear inch of single stiffener or pair of stiffeners, not less than that computed by the formula

$$f_{vs} = h \sqrt{\left(\frac{F_y}{3,400}\right)^3}$$

where F_y = yield point 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 (8). 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.

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 $24,000/\sqrt{F_b}$, the maximum stress in the compression flange shall not exceed

$$f'_b \le F_b \left[1.0 - 0.0005 \frac{A_w}{A_f} \left(\frac{h}{t} - \frac{24,000}{\sqrt{F_b}} \right) \right] \quad \text{Formula (11)}$$

where

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

1.10.7 Combined Shear and Tension Stress

Plate girder webs, under combined shear and tension stress, shall be so proportioned that the bending tensile stress, due to moment in the plane of the girder web, shall not exceed $0.6F_y$ nor

$$\left(0.825 - 0.375 \frac{f_v}{F_v}\right)F_y \qquad \text{Formula (12)}$$

where

f_v = computed web shear stress (total shear divided by web area)
F_v = allowable web shear stress according to Formula (8) or (9)

1.10.8 Splices

Spliced cross-sections in plate girders and in beams, except butt welded splices, shall develop the strength required by the stresses, at the point of splice, but in no case less than 50 percent of the effective strength of the material spliced. Butt welded splices shall develop the full strength of the smaller spliced section.

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 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 $0.75F_y$ pounds per square inch allowed in Sect. 1.5.1; otherwise, bearing stiffeners shall be provided. The governing formulas shall be:

For interior loads,

$$\frac{R}{t(N + 2k)} = \text{not over } 0.75F_y \text{ pounds per square inch} \qquad \text{Formula (13)}$$

For end-reactions,

$$\frac{R}{t(N + k)} = \text{not over } 0.75F_y \text{ pounds per square inch} \qquad \text{Formula (14)}$$

where

R = concentrated load or reaction, in pounds
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

$$\left[5.5 + \frac{4}{(a/h)^2} \right] \frac{10,000,000}{(h/t)^2} \quad \text{pounds per square inch} \qquad \text{Formula (15)}$$

when the flange is restrained against rotation, nor

$$\left[2 + \frac{4}{(a/h)^2} \right] \frac{10,000,000}{(h/t)^2} \quad \text{pounds per square inch} \qquad \text{Formula (16)}$$

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 pounds per linear inch of length, shall be divided by the web thickness.

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 poured 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 $1\frac{1}{2}$ 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. 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 point of the steel beam. The bending stress produced by loads after the concrete has hardened shall be computed on the basis of the moment of inertia of the composite section. Concrete tension stresses below the neutral axis of the composite section shall be neglected. Alternatively, the steel beam alone may be proportioned to resist unassisted the 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. 1.5.1.4.1 or 1.5.1.4.4 as applicable. The moment of inertia I_{tr} of the composite section shall be computed in accordance with the elastic theory. Concrete tension stresses below the neutral axis of the composite section shall be neglected. The compression area of the concrete above the neutral axis shall be treated as an equivalent area of steel by dividing it by the modular ratio n.

For construction without temporary shoring the value of the section modulus of the composite section used in stress calculations (referred to the tension flange) shall not exceed

$$S_{tr} = \left(1.35 + 0.35\frac{M_L}{M_D}\right)S_s \qquad \text{Formula (17)}$$

where M_L and M_D are, respectively, the live load and dead load moments and S_s is the section modulus of the steel beam (referred to its tension flange) and provided that the steel beam alone, supporting the loads before the concrete has hardened, is not stressed to more than the applicable bending stress given in Sect. 1.5.1.

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. The total horizontal shear to be thus resisted between the point of maximum positive moment and each end of the steel beam (or between the point of maximum moment and a point of contraflexure in continuous beams) may be taken as the smaller value using the formulas

$$V_h = \frac{0.85f'_c\,A_c}{2} \qquad \text{Formula (18)}$$

and

$$V_h = \frac{A_s F_y}{2} \qquad \text{Formula (19)}$$

where

f'_c = specified compression strength of concrete at 28 days
A_c = actual area of effective concrete flange defined in Sect. 1.11.1
A_s = area of steel beam

The number of connectors resisting this shear, 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, or one pitch of a spiral bar, is as given in Table 1.11.4.

TABLE 1.11.4

Connector	Allowable Horizontal Shear Load (q) (kips) (Applicable only to stone concrete)		
	$f'_c = 3{,}000$	$f'_c = 3{,}500$	$f'_c = 4{,}000$
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
½″ diam. spiral bar	11.9	12.4	12.8
⅝″ diam. spiral bar	14.8	15.4	15.9
¾″ diam. spiral bar	17.8	18.5	19.1

w = length of channel in inches.

The required number of shear connectors may be spaced uniformly between the sections of maximum and zero moment.

Shear connectors shall have at least 1 inch of concrete cover in all directions.

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

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 will not exceed $\frac{1}{360}$ of the span.

The depth of beams and girders supporting flat roofs shall be not less than $F_y/1{,}000{,}000$ times their span length whether designed as simple or continuous spans.

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 Sect. 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.

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 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 shall preferably not be less than 5 times the thickness of the member or separated element at the pin hole.   If a smaller size is used, the bearing stress shall not exceed that allowed by Sect. 1.5.1.5.1. The diameter of the pin hole shall not be more than $\frac{1}{32}$ inch greater than the diameter of the pin.

1.14.7   Effective Areas of Weld Metal

The effective area of butt 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.

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

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 center-line of the weld through the center of the plane through the throat. However, in the case of overlapping fillets, the effective area shall not exceed the nominal cross-sectional area of the hole or slot, in the plane of the faying surface.

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

The effective length of a butt 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.

The effective throat thickness of a complete penetration butt weld (i.e., a butt 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-V or single-bevel groove welds having no root opening and having partial penetration into their joints shall be $\frac{1}{4}$ inch less than the depth of the V or bevel groove. The effective throat thickness of single-J or single-U groove welds having no root opening and having partial penetration into their joints shall be the depth of the J or U groove. The effective throat thickness of any of these partial penetration groove welds 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 6,000 pounds.

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, the 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 re-peated 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:

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

$$= \frac{f_b L}{3,600,000}, \text{when the beam is designed for full uniform load}$$
$$\text{producing the unit stress } f_b \text{ at mid-span}$$

where

e = the horizontal displacement of the end of the top flange, in the direction of the span, in inches

f_b = the flexural unit stress in the beam at mid-span, in pounds 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.

1.15.6   Fillers

When rivets or bolts carrying computed stress pass through fillers thicker than $\frac{1}{4}$ inch, except in friction-type connections assembled with high strength bolts, the fillers shall be extended beyond the splice material and the filler extension shall be secured by enough rivets or bolts to distribute the total stress in the member uniformly over the combined section of the member and the filler, or an equivalent number of fasteners shall be included in the connection.

In welded construction, any filler $\frac{1}{4}$ inch or more in thickness shall extend beyond the edges of the splice plate and shall be welded to the part on which it is fitted with sufficient weld to transmit the splice plate stress, applied at the surface of the filler as an eccentric load.   The welds joining the splice plate to the filler shall be sufficient to transmit the splice plate stress and shall be long enough to avoid overstressing the filler along the toe of the weld.   Any filler less than $\frac{1}{4}$ inch thick shall have its edges made flush with the edges of the splice plate and the weld size shall be the sum of the size necessary to carry the splice plate stress plus the thickness of the filler plate.

1.15.7   Connections of Tension and Compression Members in Trusses

The connections at ends of tension or compression members in trusses shall develop the strength required by the stress, but not less than 50 percent of the effective strength of the member.

1.15.8   Compression Members with Bearing Joints

Where compression members bear on bearing plates, and where tier-building columns are finished to bear, 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 (butt, 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 $2\frac{2}{3}$ 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 Bolts* as approved by the Research Council on Riveted and Bolted Structural Joints, except that A354, Grade BC, bolts tightened to their proof load, may be substituted for A325 bolts at the working stresses permitted in Sect. 1.5 and 1.6.

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 five diameters, shall have their number increased 1 percent for each additional $\frac{1}{16}$ inch in the grip.

1.16.4 Minimum Pitch

The minimum distance between centers of rivet and bolt holes shall be not less than $2\frac{2}{3}$ 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

The distance from the center of the end rivet or high strength bolt in a bearing-type connection of a tension member having not more than two fasteners in a line parallel to the direction of stress and that end of the connected member towards which the stress is directed, shall be not less than the shearing area of the fastener (single or double shear) divided by the plate thickness for riveted connections, or by ⅔ of the plate thickness for high strength bolted connections. This end distance may, however, be decreased in such proportion as the stress per fastener is less than that permitted under Sect. 1.5.2, but it shall not be less than the distance specified in Sect. 1.16.5 above.

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 and Welding Operator Qualifications

Welds shall be made only by welders and welding operators who have been previously qualified by tests as prescribed in the *Standard Code for Arc and Gas Welding in Building Construction* of the American Welding Society, to perform the type of work required, except that this provision need not apply to tack welds not later incorporated into finished welds carrying calculated stress.

1.17.2 Qualification of Weld and Joint Details

The details of all joints (including for butt welds the groove form, root face, root spacing, etc.) to be employed under this Specification without welding procedure qualification shall comply with all the requirements for joints which are accepted without procedure qualification under the *Standard Code for Arc and Gas Welding in Building Construction* or the *Standard Specifications for Welded Highway and Railway Bridges* of the American Welding Society. Additionally, single-V, single-bevel, single-J and single-U partial penetration groove welds, having no root opening and an effective throat thickness as defined in Sect. 1.14.7, are accepted without procedure qualification. Joint forms or procedures other than those included in the foregoing may be employed provided they shall have been qualified in accordance with the requirements of these AWS Standards.

E60 and E70 series electrodes for manual arc welding and Grade SA-1 or Grade SA-2 submerged arc process may be used for welding A7, A373 and A36 steel. Only E70 low hydrogen electrodes for manual arc welding or Grade SA-2 for submerged arc welding shall be used with A441 or weldable A242 steel, except that fillet welds or partial penetration groove welds used to connect parts of built-up members and not carrying calculated stress may be made with E60 series low hydrogen electrodes and Grade SA-1 submerged arc process. Welding A440 steel is not recommended.

1.17.3 Submerged Arc Welding

The bare electrodes and granular fusible flux used in combinations for submerged arc welding shall be capable of producing weld metal having the following tensile properties when deposited in a multiple pass weld:

Grade SA-1

Tensile strength	62,000 to 80,000 psi
Yield point, min.	45,000 psi
Elongation in 2 in., min.	25%
Reduction in area, min..	40%

Grade SA-2

Tensile strength	70,000 to 90,000 psi
Yield point, min..	50,000 psi
Elongation in 2 in., min..	22%
Reduction in area, min..	40%

1.17.4 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 1.17.4. Weld size is determined by the thicker of the two parts joined, except that the weld size need not exceed the thickness of the thinner part joined unless a larger size is required by calculated stress:

TABLE 1.17.4

Material Thickness of Thicker Part Joined (Inches)	Minimum Size of Fillet Weld (Inches)
To $\frac{1}{2}$ inclusive	$\frac{3}{16}$
Over $\frac{1}{2}$ to $\frac{3}{4}$	$\frac{1}{4}$
Over $\frac{3}{4}$ to $1\frac{1}{2}$	$\frac{5}{16}$
Over $1\frac{1}{2}$ to $2\frac{1}{4}$	$\frac{3}{8}$
Over $2\frac{1}{4}$ to 6	$\frac{1}{2}$
Over 6	$\frac{5}{8}$

1.17.5 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 $\frac{1}{4}$ inch thick, the maximum size may be equal to the thickness of the material.
2. Along edges of material $\frac{1}{4}$ inch or more in thickness, the maximum size shall be $\frac{1}{16}$ 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.

1.17.6 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.

1.17.7 Intermittent Fillet Welds

Intermittent fillet welds may be used to transfer calculated stress across a joint or faying surfaces when the strength required is less than that developed by a continuous fillet weld of the smallest permitted size, and to join components of built-up members. The effective length of any segment of intermittent fillet welding shall be not less than 4 times the weld size with a minimum of $1\frac{1}{2}$ inches.

1.17.8 Lap Joints

The minimum width of laps 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 edge of both lapped parts except where the deflection of the lapped parts is sufficiently restrained to prevent opening of the joint under maximum loading.

1.17.9   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.

1.17.10   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 be to considered plug or slot welds.

1.17.11   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 $2\frac{1}{4}$ times the thickness of the weld metal.

The minimum center-to-center spacing of plug welds shall be 4 times the diameter of the hole.

The length of slot for a slot weld shall not exceed 10 times the thickness of the weld.   The width of the slot shall be not less than the thickness of the part containing it, plus $5/16$ inch, rounded to the next greater odd $1/16$ inch, nor shall it be greater than $2\frac{1}{4}$ times the thickness of the weld.   The ends of the slot shall be semicircular or shall have the corners rounded to a radius not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

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

The thickness of plug or slot welds in material $5/8$ 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 Sect. 1.8 and 1.9.

1.18.2.2 At the ends of built-up compression members bearing on base plates or milled surfaces, all components in contact with one another shall be connected by rivets or bolts spaced longitudinally not more than 4 diameters apart for a distance equal to $1\frac{1}{2}$ times the maximum width of the member, or by continuous welds having a length not less than the maximum width of the member.

1.18.2.3 The longitudinal spacing for intermediate rivets, bolts or intermittent welds in built-up members shall be adequate to provide for the transfer of calculated stress. However, where a component of a built-up compression member consists of an outside plate, the maximum spacing shall not exceed the thickness of the thinner outside plate times $4{,}000/\sqrt{F_y}$ 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 $6{,}000/\sqrt{F_y}$ 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 $\frac{1}{50}$ 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) or (3) 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 net width of such plates across 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 $1\frac{1}{2}$ 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 plates 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 $\frac{1}{50}$ 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 shall be done by methods that will not injure the metal.

1.23.2   Gas Cutting

Gas cutting shall preferably be done by machine. Gas cut edges which will be subjected to substantial stress or which are to have weld metal deposited on them shall be free from gouges; any gouges 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.

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 and A354, Grade BC, bolts shall be tightened to a bolt tension not less than the proof load given in the applicable ASTM specification for the type of bolt used. Tightening shall be done with properly calibrated wrenches or by the turn-of-nut method.

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.

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 $\frac{3}{16}$ inch. If the separation is $\frac{1}{16}$ 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 $\frac{1}{16}$ inch. The fit of joints at contact surfaces which are not completely sealed by welds, shall be close enough to exclude water after painting.

Abutting parts to be butt welded shall be carefully aligned. Misalignments greater than $\frac{1}{8}$ inch shall be corrected and, in making the correction, the parts shall not be drawn into a sharper slope than 2 degrees ($\frac{7}{16}$ 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 butt 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 $\frac{5}{16}$ 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. Butt 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 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.

Butt welds shall be terminated at the ends of a joint in a manner that will insure 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.

No welding shall be done when the ambient temperature is lower than 0° F.

Base metal shall be preheated as required to the temperature called for in Table 1.23.6 prior to tack welding or welding. 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.

Where required, 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

TABLE 1.23.6

Thickness of Thickest Part at Point of Welding	Minimum Preheat and Interpass Temperatures					
	Other Than Low-Hydrogen Welding Processes[1]			Low-Hydrogen Welding Processes[2]		
	A373 Steel	A7, A36 Steel	A441 Steel	A373 Steel	A7, A36 Steel	A441 Steel[3]
To 1″, incl.	None[4]	None[4]	Welding with this process not recommended	None[4]	None[4]	None[4]
Over 1″ to 2″, incl.	100°F	200°F		None[4]	50°F	100°F
Over 2″	200°F	300°F		100°F	150°F	200°F

[1] Welding with ASTM A233 E60XX or E70XX electrodes other than a low-hydrogen class.

[2] Welding with properly dried ASTM A233 EXX15, 16, 18 or 28 electrodes or submerged arc welding with properly dried flux.

[3] Preheating for weldable A242 steel may need to be either higher or lower than these requirements, depending on composition of steel.

[4] Except when base metal temperature is below 32° F.

exercised to prevent scaling, or flaking of weld and base metal from over-peening.

The technique of welding employed, the appearance and quality of welds made, and the methods used in correcting defective work shall conform to Section 4—Workmanship, of the *Standard Code for Arc and Gas Welding in Building Construction* of the American Welding Society.

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 para-graph.   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 $\frac{1}{1000}$ 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 $\frac{1}{32}$ 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 $\frac{1}{16}$ inch for members 30 feet or less in length, and not greater than $\frac{1}{8}$ 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, 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 INSPECTION

1.26.1 General

Material and workmanship at all times shall be subject to the inspection of experienced engineers representing the purchaser.

1.26.2 Cooperation

All inspection as far as possible shall be made at the place of manufacture, and the contractor or manufacturer shall cooperate with the inspector, permitting access for inspection to all places where work is being done.

1.26.3 Rejections

Material or workmanship not conforming to the provisions of this Specification may be rejected at any time defects are found during the progress of the work.

1.26.4 Inspection of Welding

The inspection of welding shall be performed in accordance with the provisions of Section 5 of the *Standard Code for Arc and Gas Welding in Building Construction* of the American Welding Society.

PART 2

SECTION 2.1 SCOPE

Subject to the limitations contained herein, simple or continuous beams, one and two-story rigid frames classified as Type 1 construction in Sect. 1.2 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., of their maximum strength. This strength, as determined by rational analysis, shall not be less than that required to support 1.70 times the given live load and dead load for simple and continuous beams. For continuous frames it shall not be less than 1.85 times the given live load and dead load, nor 1.40 times these loads acting in conjunction with 1.40 times any specified wind or earthquake forces.

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:

> *Steel for Bridges and Buildings*, ASTM A7
> *Structural Steel for Welding*, ASTM A373
> *Structural Steel*, ASTM A36

SECTION 2.3 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 120, l being taken as the distance center-to-center of adjacent members connecting to the column or the distance from such a member to the base of the column. The slenderness ratio of columns covered by Formula (21) shall not exceed 100. The maximum axial load P at ultimate loading shall not exceed six-tenths P_y, where P_y is the product of yield point stress times column area.

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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.

Columns in continuous frames, where sidesway is not prevented (a) by diagonal bracing, (b) by attachment to an adjacent structure having ample lateral stability or (c) by floor slabs or roof decks secured horizontally by walls or bracing systems parallel to the plane of the continuous frames, shall be so proportioned that

$$\frac{2P}{P_y} + \frac{l}{70r} \leq 1.0 \qquad \text{Formula (20)}$$

Except as otherwise provided in this section, M_o/M_p, the ratio of allowable end moment to the full plastic bending strength of columns and other axially loaded members, shall not exceed the value given by the following formulas, where they are applicable:

CASE I.   For columns bent in double curvature by moments producing plastic hinges at both ends of the columns

$$M_o = M_p \quad \text{when } P/P_y \leq 0.15$$

$$\frac{M_o}{M_p} \leq 1.18 - 1.18\left(\frac{P}{P_y}\right) \leq 1.0 \text{ when } P/P_y > 0.15 \qquad \text{Formula (21)}$$

CASE II.   For pin-based columns required to develop a hinge at one end only, and double curvature columns required to develop a hinge at one end when the moment at the other end would be less than the hinge value

$$\frac{M_o}{M_p} \leq B - G\left(\frac{P}{P_y}\right) \leq 1.0 \qquad \text{Formula (22)}$$

the numerical values for B and G, for any given slenderness ratio in the plane of bending l/r, being those listed in Tables 4-33 and 4-36 of the Appendix. Where l/r in the plane of bending is less than 60, and P/P_y does not exceed 0.15, the full plastic strength of the member may be used ($M_o = M_p$).

CASE III.   For columns bent in single curvature

$$\frac{M_o}{M_p} \leq 1.0 - K\left(\frac{P}{P_y}\right) - J\left(\frac{P}{P_y}\right)^2 \qquad \text{Formula (23)}$$

the numerical values for K and J being those given in Tables 5-33 and 5-36 of the Appendix.

In no case shall the ratio of axial load to plastic load exceed that given by the following expression:

$$\frac{P}{P_y} = \frac{8,700}{(l/r)^2} \quad \text{when } \frac{l}{r} > 120 \qquad \text{Formula (24)}$$

where l and r are the unbraced length and radius of gyration of the column in the plane normal to that of the continuous frame under consideration.

SECTION 2.4 SHEAR

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

$$V_u \leq 0.55F_y wd$$

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

(Shear stresses are generally high within the boundaries of the connection of two or more members whose webs lie in a common plane. The foregoing provisions will be satisfied, without reinforcing the web within the connection, when its thickness w, in inches, is greater than $23,000M/A_{bc}F_y$, M being the algebraic sum of clockwise and counter-clockwise moment (in kip-feet) applied on opposite sides of the connection web boundary, and A_{bc} the planar area of the connection web, expressed in square inches, and F_y is given in pounds per square inch. When the thickness of this web is less than that given by the above formula the deficiency may be compensated by a pair of diagonal stiffeners or by a reinforcing plate in contact with the web over the connection area.)

SECTION 2.5 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, opposite these flanges, when

$$w < \frac{A_f}{t_f + 5k}$$

or when

$$t_f < 0.4\sqrt{A_f}$$

where

w = thickness of web to be stiffened
k = distance from outer face of flange to web toe of fillet of member to be stiffened
t_f = thickness of flange of member to be stiffened
A_f = area of flange delivering concentrated load

The end of such stiffeners shall be fully welded to the inside face of the flange opposite the concentrated tensile load. It 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 the full plastic strength of the stiffener cross-section.

SECTION 2.6   MINIMUM THICKNESS (WIDTH-THICKNESS RATIOS)

Projecting elements that would be subjected to compression involving plastic hinge rotation under ultimate loading, shall have width-thickness ratios no greater than the following:

> Flanges of rolled shapes and flange plates of similar built-up shapes: 8½, except that for rolled shapes an upward variation of 3 percent may be tolerated.   The thickness of sloping flanges may be taken as their average thickness.   Stiffeners and that portion of flange plates in box sections and cover plates included between the free edge and the first longitudinal row of fasteners or connecting welds: 8½.

The width-thickness ratio of flange plates in box sections and flange cover plates included between longitudinal lines of connecting rivets, high strength bolts or welds, shall not exceed 32.

The depth-thickness ratio of beam and girder webs subjected to plastic bending without axial loading shall not exceed 70 and, when subjected to combined axial force and plastic bending moment at ultimate loading, the value given by the formula

$$\frac{d}{w} \leq 70 \,-\, 100\frac{P}{P_y} \qquad\qquad \text{Formula (25)}$$

with a minimum value of 43.

SECTION 2.7   CONNECTIONS

All connections, the rigidity of which is essential to the continuity assumed as the basis of the design analysis, shall be capable of resisting the moments, shears and axial loads to which they would be subjected by the ultimate loading.

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.

Rivets, welds and A307 bolts shall be proportioned to resist the forces produced at ultimate load using unit stresses equal to 1.67 times those given in Part 1.

In general, groove welds are preferable to fillet welds, but their use is not mandatory when the strength of the latter at 1.67 times the stress given in Part 1 is sufficient to resist the ultimate load imposed upon a joint.

High strength bolts may be proportioned, on the basis of their minimum guaranteed proof load, to resist the tension produced by the ultimate loading. When used to transmit shear produced by the ultimate loading, one bolt may be substituted for a rivet of the same nominal diameter.   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 ultimate loading, of the plastic hinges assumed in the design.

SECTION 2.8 LATERAL BRACING

The maximum laterally unsupported length of members designed on the basis of ultimate loading need not be less than that which would be permitted for the same members designed under the provisions of Part 1, except at plastic hinge locations associated with the failure mechanism. Furthermore, the following 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. Other plastic hinge locations shall be adequately braced to resist lateral and torsional displacement.

The laterally unsupported distance l_{cr}, from such braced hinge locations to the nearest adjacent point on the frame similarly braced, need not be less than that given by the formula

$$l_{cr} = \left(60 - 40\frac{M}{M_p}\right)r_y \qquad \text{Formula (26)}$$

nor less than $35r_y$, 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 and
M/M_p, the end moment ratio, is positive when the segment is bent in single curvature and negative when bent in double curvature.

Any greater laterally unbraced length for these segments must be justified by an analysis based upon the predictable amount of restraint present at the ends of the segment in the plane of the computed bending moments.

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 2.9 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 ultimate loading. If used they shall be finished smooth by grinding, chipping or planing.

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

APPENDIX

For Steels with 33,000 psi Specified Yield Point

Applicable to ASTM A7 and A373 Structural Steel

Approved Welding Electrodes: E60 Series. Submerged Arc Grade SA-1.
E70 Series. Submerged Arc Grade SA-2.

PART 1

SECTION 1.5 ALLOWABLE UNIT STRESSES

1.5.1 Structural Steel

1.5.1.1 Tension

Tension on net section, except at pin holes . . . . $F_t = 20,000$ psi
Tension on net section at pin holes . . . . $F_t = 15,000$ psi

1.5.1.2 Shear

Shear on gross section (see Table 3-33 for reduced
values for girder webs) . . . . $F_v = 13,000$ psi

1.5.1.3 Compression

$$C_c = 131.7$$

For values of F_a given by Formulas (1),
(2) and (3) see Table 1-33.

1.5.1.4 Bending

1.5.1.4.1 Tension and compression for compact,
adequately braced beams having an axis of
symmetry in the plane of loading . . . . $F_b = 22,000$ psi
1.5.1.4.2 Tension and compression for unsymmetrical
rolled shapes continuously braced in the region
under compression stress . . . . $F_b = 20,000$ psi
1.5.1.4.3 Tension and compression for box-type
members not included in Sect. 1.5.1.4.1 . . . . $F_b = 20,000$ psi
1.5.1.4.4 Tension for other rolled shapes, built-up
members and plate girders . . . . $F_b = 20,000$ psi
1.5.1.4.5 Compression, except as provided by Sect.
1.5.1.4.1, 1.5.1.4.2, 1.5.1.4.3, 1.5.1.4.7 and 1.5.1.4.8:
the larger value given by Formulas (4) and (5).

$$F_b = 20,000 - \frac{0.571}{C_b}\left(\frac{l}{r}\right)^2 \qquad \text{Formula (4)}$$

$$F_b = \frac{12,000,000}{ld/A_f} \leq 20,000 \text{ psi} \qquad \text{Formula (5)}$$

$F_y = 33$ ksi

TABLE 1-33

ALLOWABLE STRESS (KSI)
FOR COMPRESSION MEMBERS OF 33 KSI SPECIFIED YIELD POINT STEEL

Main and Secondary Members l/r not over 120						Main Members l/r 121 to 200				Secondary Members l/r 121 to 200			
$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)
1	19.77	41	17.64	81	14.32	121	9.96	161	5.76	121	10.01	161	7.25
2	19.73	42	17.57	82	14.23	122	9.84	162	5.69	122	9.94	162	7.20
3	19.69	43	17.50	83	14.13	123	9.72	163	5.62	123	9.87	163	7.16
4	19.66	44	17.43	84	14.03	124	9.59	164	5.55	124	9.79	164	7.12
5	19.62	45	17.36	85	13.93	125	9.47	165	5.49	125	9.71	165	7.08
6	19.58	46	17.29	86	13.84	126	9.34	166	5.42	126	9.63	166	7.04
7	19.54	47	17.22	87	13.74	127	9.22	167	5.35	127	9.55	167	7.00
8	19.50	48	17.14	88	13.64	128	9.09	168	5.29	128	9.47	168	6.96
9	19.46	49	17.07	89	13.53	129	8.96	169	5.23	129	9.38	169	6.93
10	19.41	50	16.99	90	13.43	130	8.83	170	5.17	130	9.30	170	6.89
11	19.37	51	16.92	91	13.33	131	8.70	171	5.11	131	9.21	171	6.85
12	19.32	52	16.84	92	13.23	132	8.57	172	5.05	132	9.12	172	6.82
13	19.28	53	16.76	93	13.13	133	8.44	173	4.99	133	9.03	173	6.79
14	19.23	54	16.68	94	13.02	134	8.32	174	4.93	134	8.94	174	6.76
15	19.18	55	16.60	95	12.92	135	8.19	175	4.88	135	8.86	175	6.73
16	19.13	56	16.52	96	12.81	136	8.07	176	4.82	136	8.78	176	6.70
17	19.08	57	16.44	97	12.71	137	7.96	177	4.77	137	8.70	177	6.67
18	19.03	58	16.36	98	12.60	138	7.84	178	4.71	138	8.62	178	6.64
19	18.98	59	16.28	99	12.49	139	7.73	179	4.66	139	8.54	179	6.61
20	18.93	60	16.20	100	12.38	140	7.62	180	4.61	140	8.47	180	6.58
21	18.88	61	16.12	101	12.28	141	7.51	181	4.56	141	8.39	181	6.56
22	18.82	62	16.03	102	12.17	142	7.41	182	4.51	142	8.32	182	6.53
23	18.77	63	15.95	103	12.06	143	7.30	183	4.46	143	8.25	183	6.51
24	18.71	64	15.86	104	11.95	144	7.20	184	4.41	144	8.18	184	6.49
25	18.66	65	15.78	105	11.83	145	7.10	185	4.36	145	8.12	185	6.46
26	18.60	66	15.69	106	11.72	146	7.01	186	4.32	146	8.05	186	6.44
27	18.54	67	15.61	107	11.61	147	6.91	187	4.27	147	7.99	187	6.42
28	18.48	68	15.52	108	11.50	148	6.82	188	4.23	148	7.93	188	6.40
29	18.42	69	15.43	109	11.38	149	6.73	189	4.18	149	7.87	189	6.38
30	18.36	70	15.34	110	11.27	150	6.64	190	4.14	150	7.81	190	6.36
31	18.30	71	15.25	111	11.15	151	6.55	191	4.09	151	7.75	191	6.35
32	18.24	72	15.16	112	11.04	152	6.46	192	4.05	152	7.69	192	6.33
33	18.18	73	15.07	113	10.92	153	6.38	193	4.01	153	7.64	193	6.31
34	18.11	74	14.98	114	10.80	154	6.30	194	3.97	154	7.59	194	6.30
35	18.05	75	14.89	115	10.69	155	6.22	195	3.93	155	7.53	195	6.28
36	17.98	76	14.80	116	10.57	156	6.14	196	3.89	156	7.48	196	6.27
37	17.92	77	14.70	117	10.45	157	6.06	197	3.85	157	7.43	197	6.26
38	17.85	78	14.61	118	10.33	158	5.98	198	3.81	158	7.39	198	6.24
39	17.78	79	14.51	119	10.21	159	5.91	199	3.77	159	7.34	199	6.23
40	17.71	80	14.42	120	10.09	160	5.83	200	3.73	160	7.29	200	6.22

TABLE 2

VALUES OF F'_e (KSI)

For use in Formula (6), Sect. 1.6.1, for all grades of steel

$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)
21	338.13	51	57.33	81	22.73	111	12.12	141	7.51	171	5.11
22	308.09	52	55.15	82	22.18	112	11.90	142	7.41	172	5.05
23	281.88	53	53.09	83	21.65	113	11.69	143	7.30	173	4.99
24	258.89	54	51.14	84	21.13	114	11.49	144	7.20	174	4.93
25	238.59	55	49.30	85	20.64	115	11.29	145	7.10	175	4.88
26	220.58	56	47.56	86	20.16	116	11.10	146	7.01	176	4.82
27	204.55	57	45.90	87	19.70	117	10.91	147	6.91	177	4.77
28	190.20	58	44.33	88	19.26	118	10.73	148	6.82	178	4.71
29	177.31	59	42.84	89	18.83	119	10.55	149	6.73	179	4.66
30	165.68	60	41.43	90	18.41	120	10.37	150	6.64	180	4.61
31	155.17	61	40.07	91	18.01	121	10.20	151	6.55	181	4.56
32	145.62	62	38.79	92	17.62	122	10.03	152	6.46	182	4.51
33	136.93	63	37.57	93	17.24	123	9.87	153	6.38	183	4.46
34	128.99	64	36.41	94	16.88	124	9.71	154	6.30	184	4.41
35	121.73	65	35.29	95	16.53	125	9.56	155	6.22	185	4.36
36	115.06	66	34.24	96	16.18	126	9.41	156	6.14	186	4.32
37	108.93	67	33.22	97	15.85	127	9.26	157	6.06	187	4.27
38	103.27	68	32.25	98	15.53	128	9.11	158	5.98	188	4.23
39	98.04	69	31.32	99	15.21	129	8.97	159	5.91	189	4.18
40	93.20	70	30.44	100	14.91	130	8.84	160	5.83	190	4.14
41	88.71	71	29.58	101	14.62	131	8.70	161	5.76	191	4.09
42	84.53	72	28.77	102	14.34	132	8.57	162	5.69	192	4.05
43	80.65	73	27.99	103	14.06	133	8.44	163	5.62	193	4.01
44	77.02	74	27.24	104	13.73	134	8.32	164	5.55	194	3.97
45	73.64	75	26.51	105	13.53	135	8.19	165	5.49	195	3.93
46	70.47	76	25.82	106	13.28	136	8.07	166	5.42	196	3.89
47	67.51	77	25.15	107	13.02	137	7.96	167	5.36	197	3.85
48	64.73	78	24.51	108	12.80	138	7.84	168	5.29	198	3.81
49	62.11	79	23.89	109	12.57	139	7.73	169	5.23	199	3.77
50	59.65	80	23.30	110	12.34	140	7.62	170	5.17	200	3.73

$$F'_e = \frac{149{,}000{,}000}{(l/r_b)^2}$$

$F_y = 33$ ksi

TABLE 3-33

ALLOWABLE SHEAR STRESSES (F_v) IN PLATE GIRDERS (KSI)
FOR 33 KSI SPECIFIED YIELD POINT STEEL

(Required Gross Area of Intermediate Stiffeners, as per cent of web area, shown in *italics*)

$F_y = 33$ ksi

Slenderness ratios h/t: web depth to web thickness

h/t	Aspect ratios a/h: stiffener spacing to web depth
	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	over 3
70								13.0	13.0	13.0	13.0	13.0	13.0	12.6
80						13.0	13.0	12.9	12.5	12.2	12.0	11.6	11.5	11.0
90					13.0	12.9	12.1	11.5	11.3 *0.4*	11.1 *0.7*	11.0 *0.8*	10.7 *0.9*	10.6 *0.9*	9.8
100				13.0	12.4	11.6	11.2 *0.8*	11.0 *1.3*	10.7 *1.6*	10.5 *1.7*	10.4 *1.8*	10.0 *1.7*	9.8 *1.6*	8.4
110			13.0	12.2	11.4 *0.3*	11.2 *1.2*	10.8 *2.0*	10.5 *2.4*	10.2 *2.7*	9.8 *2.9*	9.5 *3.0*	9.0 *2.9*	8.6 *2.7*	6.9
120		13.0	12.3	11.4 *0.5*	11.1 *1.6*	10.8 *2.3*	10.4 *3.2*	9.8 *3.9*	9.4 *4.2*	9.0 *4.3*	8.7 *4.2*	8.2 *3.9*	7.8 *3.5*	5.8
130		12.7	11.5 *0.3*	11.1 *1.6*	10.8 *2.6*	10.4 *3.6*	9.8 *4.8*	9.2 *5.3*	8.8 *5.4*	8.4 *5.3*	8.1 *5.2*	7.5 *4.6*	7.1 *4.1*	5.0
140	13.0	11.8	11.2 *1.3*	10.9 *2.6*	10.5 *3.9*	10.0 *5.1*	9.3 *6.1*	8.8 *6.3*	8.3 *6.3*	7.9 *6.2*	7.6 *5.9*	7.0 *5.2*	6.5 *4.6*	4.3
150	12.8	11.4 *0.7*	11.0 *2.2*	10.6 *3.8*	10.1 *5.3*	9.6 *6.4*	8.9 *7.1*	8.4 *7.2*	7.9 *7.1*	7.5 *6.8*	7.2 *6.5*	6.6 *5.7*	6.1 *5.0*	3.7
160	12.0	11.2 *1.5*	10.9 *3.1*	10.3 *5.1*	9.8 *6.5*	9.3 *7.4*	8.6 *7.9*	8.1 *7.9*	7.6 *7.7*	7.2 *7.4*	6.9 *7.0*	6.2 *6.1*	5.8 *5.3*	
170	11.5 *0.3*	11.1 *2.3*	10.6 *4.4*	10.0 *6.3*	9.5 *7.5*	9.1 *8.2*	8.4 *8.6*	7.8 *8.5*	7.3 *8.2*	6.9 *7.8*	6.6 *7.4*	5.9 *6.4*		
180	11.3 *1.1*	10.9 *3.0*	10.3 *5.6*	9.8 *7.2*	9.3 *8.3*	8.8 *8.9*	8.2 *9.2*	7.6 *9.0*	7.1 *8.6*	6.7 *8.2*	6.4 *7.7*			
200	11.1 *2.3*	10.5 *5.2*	9.9 *7.4*	9.4 *8.7*	8.9 *9.5*	8.5 *10.0*	7.8 *10.1*	7.3 *9.8*	6.8 *9.3*	6.4 *8.8*				
220	10.8 *4.0*	10.2 *6.8*	9.6 *8.7*	9.1 *9.8*	8.7 *10.5*	8.3 *10.8*	7.6 *10.7*	7.0 *10.3*						
240	10.5 *5.5*	9.9 *8.1*	9.4 *9.7*	8.9 *10.6*	8.5 *11.2*	8.1 *11.4*	7.4 *11.2*							
260	10.3 *6.8*	9.7 *9.0*	9.2 *10.4*	8.8 *11.3*	8.3 *11.7*	7.9 *11.9*								
280	10.1 *7.7*	9.6 *9.8*	9.1 *11.1*	8.6 *11.8*	8.2 *12.2*									
300	10.0 *8.5*	9.4 *10.4*	9.0 *11.6*	8.5 *12.2*										
320	9.8 *9.2*	9.3 *10.9*	8.9 *12.0*											
340	9.7 *9.7*	9.3 *11.3*												
360	9.7 *10.1*													

Girders so proportioned that the computed shear is less than that given in right-hand column do not require intermediate stiffeners.
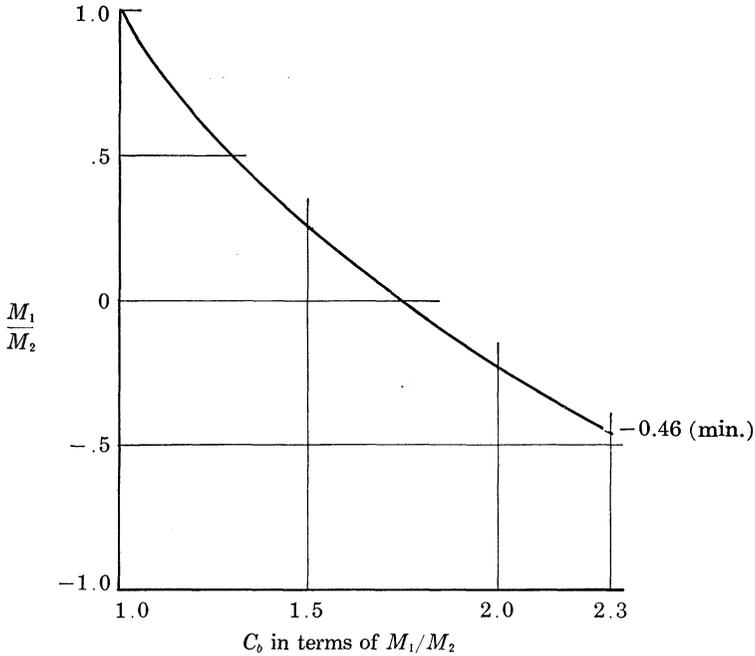
Fig. 1.5.1.4.5

1.5.1.4.6 Compression for channels: Use Formula (5) above.

1.5.1.4.7 Tension and compression for large pins......$F_b = 30,000$ psi

1.5.1.4.8 Tension and compression for rectangular bearing plates................................$F_b = 25,000$ psi

1.5.1.5 Bearing

1.5.1.5.1 On milled surfaces and pins in reamed, drilled or bored holes.............................$F_p = 30,000$ psi

1.5.1.5.2 On finished stiffeners......................$F_p = 26,000$ psi

1.5.1.5.3 On expansion rockers and rollers (in pounds per linear inch)...................................$F_p = 660d$

SECTION 1.6 COMBINED STRESSES

1.6.1 Axial Compression and Bending

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \dfrac{f_a}{F'_e}\right) F_b} \leq 1.0 \qquad \text{Formula (6)}$$

$$\frac{f_a}{20,000} + \frac{f_b}{F_b} \leq 1.0 \qquad \text{Formula (7)}$$

For values of F_a see Table 1-33.

For values of F'_e see Table 2.

$F_y = 33$ ksi

SECTION 1.9 WIDTH-THICKNESS RATIOS

SECTION 1.10 PLATE GIRDERS AND ROLLED BEAMS

1.10.2 Web

Maximum clear distance between flanges $\qquad h = 345t$

1.10.5 Stiffeners

1.10.5.3 For required stiffener spacing and gross area of intermediate stiffeners see Table 3-33.

1.10.5.4 Maximum shear between web and intermediate stiffeners in pounds per linear inch of stiffeners or pair of stiffeners $\qquad f_{vs} = 30h$

1.10.6 Reduction in Flange Stress

When h/t exceeds $24,000/\sqrt{F_b}$ the maximum compression flange stress shall not exceed

$$F_b \left[1.0 - 0.0005 \frac{A_w}{A_f} \left(\frac{h}{t} - \frac{24,000}{\sqrt{F_b}} \right) \right] \qquad \text{Formula (11)}$$

1.10.7 Combined Shear and Tension

$$F_b = 27,000 - 12,500 \left(\frac{f_v}{F_v} \right) \leq 20,000 \text{ psi} \qquad \text{Formula (12)}$$

1.10.10 Web Crippling

1.10.10.1 Use stiffeners under concentrated interior loads when

$$\frac{R}{t(N + 2k)} \text{ would exceed } 25,000 \text{ psi} \qquad \text{Formula (13)}$$

and under end reactions when

$$\frac{R}{t(N + k)} \text{ would exceed } 25,000 \text{ psi} \qquad \text{Formula (14)}$$

1.10.10.2 The compression stress, in pounds per square inch, produced by loads applied to girder webs, except through stiffeners, shall not exceed

$$\left[5.5 + \frac{4}{(a/h)^2} \right] \frac{10,000,000}{(h/t)^2} \qquad \text{Formula (15)}$$

when flange is restrained against rotation; otherwise

$$\left[2 + \frac{4}{(a/h)^2}\right] \frac{10,000,000}{(h/t)^2} \qquad \text{Formula (16)}$$

The compression stresses to be limited by formulas (15) and (16) shall be computed as follows:

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

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

$F_y = 33$ ksi

PART 2

TABLE 4-33

FOR 33 KSI SPECIFIED YIELD POINT STEEL

$F_y = 33$ ksi

Formula (22)

$$\frac{M_o}{M_p} = B - G\,\frac{P}{P_y}$$
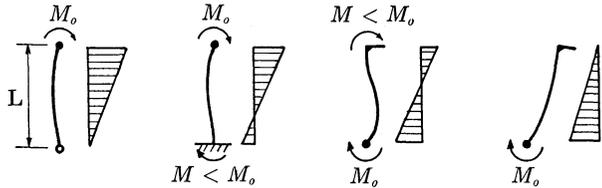
l/r	B	G	l/r	B	G	l/r	B	G
16	1.140	1.172	51	1.164	1.271	86	1.201	1.616
17	1.140	1.174	52	1.165	1.276	87	1.202	1.633
18	1.141	1.177	53	1.165	1.281	88	1.204	1.651
19	1.141	1.179	54	1.166	1.286	89	1.205	1.669
20	1.142	1.182	55	1.167	1.292	90	1.206	1.688
21	1.142	1.184	56	1.168	1.297	91	1.207	1.707
22	1.143	1.187	57	1.169	1.303	92	1.209	1.726
23	1.143	1.189	58	1.170	1.310	93	1.210	1.746
24	1.144	1.191	59	1.171	1.316	94	1.211	1.767
25	1.145	1.194	60	1.172	1.323	95	1.213	1.788
26	1.145	1.196	61	1.173	1.330	96	1.214	1.810
27	1.146	1.198	62	1.174	1.337	97	1.215	1.832
28	1.146	1.200	63	1.175	1.344	98	1.217	1.855
29	1.147	1.203	64	1.176	1.352	99	1.218	1.879
30	1.148	1.205	65	1.177	1.360	100	1.220	1.903
31	1.148	1.207	66	1.178	1.369	101	1.221	1.928
32	1.149	1.209	67	1.179	1.377	102	1.222	1.953
33	1.150	1.212	68	1.180	1.386	103	1.224	1.979
34	1.150	1.215	69	1.181	1.396	104	1.225	2.006
35	1.151	1.217	70	1.182	1.406	105	1.227	2.033
36	1.152	1.220	71	1.183	1.416	106	1.228	2.061
37	1.152	1.222	72	1.184	1.426	107	1.230	2.090
38	1.153	1.225	73	1.186	1.437	108	1.231	2.119
39	1.154	1.228	74	1.187	1.448	109	1.233	2.149
40	1.155	1.231	75	1.188	1.460	110	1.234	2.179
41	1.155	1.234	76	1.189	1.472	111	1.236	2.211
42	1.156	1.237	77	1.190	1.485	112	1.237	2.243
43	1.157	1.240	78	1.191	1.497	113	1.239	2.275
44	1.158	1.243	79	1.192	1.511	114	1.240	2.309
45	1.159	1.247	80	1.194	1.524	115	1.242	2.343
46	1.159	1.251	81	1.195	1.539	116	1.243	2.378
47	1.160	1.254	82	1.196	1.553	117	1.245	2.414
48	1.161	1.258	83	1.197	1.568	118	1.247	2.450
49	1.162	1.263	84	1.198	1.584	119	1.248	2.487
50	1.163	1.267	85	1.200	1.600	120	1.250	2.525

<div align="center">

TABLE 5-33

FOR 33 KSI SPECIFIED YIELD POINT STEEL

</div>

Formula (23)

$$\frac{M_o}{M_p} = 1.0 - K\left(\frac{P}{P_y}\right) - J\left(\frac{P}{P_y}\right)^2$$
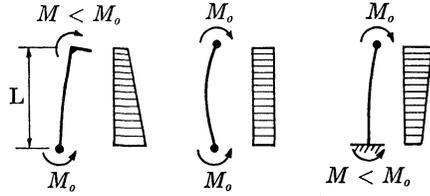
$F_y = 33\ ksi$

l/r	K	J	l/r	K	J	l/r	K	J
1	.434	.753	41	1.015	.149	81	1.824	− .738
2	.449	.736	42	1.032	.133	82	1.850	− .769
3	.463	.720	43	1.048	.116	83	1.877	− .801
4	.478	.703	44	1.064	.0998	84	1.903	− .833
5	.492	.687	45	1.081	.0832	85	1.930	− .866
6	.506	.671	46	1.097	.0663	86	1.958	− .900
7	.520	.655	47	1.114	.0492	87	1.986	− .934
8	.534	.640	48	1.131	.0318	88	2.014	− .969
9	.548	.624	49	1.148	.0143	89	2.042	−1.004
10	.562	.609	50	1.166	− .0036	90	2.071	−1.041
11	.576	.594	51	1.183	− .0217	91	2.101	−1.077
12	.590	.579	52	1.201	− .0401	92	2.130	−1.115
13	.604	.564	53	1.219	− .0588	93	2.161	−1.153
14	.619	.549	54	1.237	− .0777	94	2.191	−1.192
15	.633	.534	55	1.256	− .0970	95	2.222	−1.231
16	.647	.519	56	1.274	− .117	96	2.254	−1.272
17	.661	.504	57	1.293	− .137	97	2.286	−1.313
18	.675	.490	58	1.312	− .157	98	2.318	−1.354
19	.689	.475	59	1.332	− .177	99	2.350	−1.397
20	.703	.461	60	1.351	− .198	100	2.384	−1.440
21	.717	.447	61	1.371	− .220	101	2.417	−1.484
22	.731	.432	62	1.391	− .241	102	2.451	−1.529
23	.746	.418	63	1.411	− .263	103	2.486	−1.575
24	.760	.403	64	1.432	− .286	104	2.521	−1.621
25	.774	.389	65	1.452	− .309	105	2.556	−1.668
26	.789	.374	66	1.473	− .332	106	2.592	−1.716
27	.803	.360	67	1.495	− .356	107	2.628	−1.765
28	.818	.345	68	1.516	− .380	108	2.665	−1.814
29	.832	.331	69	1.538	− .404	109	2.703	−1.865
30	.847	.316	70	1.560	− .429	110	2.741	−1.916
31	.862	.301	71	1.583	− .455	111	2.779	−1.968
32	.877	.287	72	1.605	− .481	112	2.818	−2.021
33	.892	.272	73	1.628	− .507	113	2.857	−2.057
34	.907	.257	74	1.652	− .534	114	2.897	−2.123
35	.922	.242	75	1.675	− .562	115	2.937	−2.185
36	.937	.227	76	1.699	− .590	116	2.978	−2.242
37	.953	.211	77	1.724	− .618	117	3.020	−2.300
38	.968	.196	78	1.748	− .647	118	3.062	−2.358
39	.984	.180	79	1.773	− .677	119	3.104	−2.417
40	1.000	.165	80	1.799	− .707	120	3.147	−2.478

$F_y = 33$ ksi

PART 2

LOAD FACTOR

Live plus dead load for simple or continuous beams 1.70
Live plus dead load for continuous frames 1.85
Live plus dead load plus lateral forces for continuous frames 1.40

SECTION 2.3 COLUMNS

For reduction factors in accordance with Formulas (21), (22) and (23), to be applied to the tabulated M_p value furnished by members subject to axial loading, see Table 4-33 and Table 5-33.

SECTION 2.4 SHEAR

Allowable web shear (psi)

$$V_u \leq 18{,}000 \; wd$$

SECTION 2.6 WIDTH-THICKNESS RATIOS

(Applicable only to elements subject to compression involving plastic hinge rotation under ultimate loading.)

Beam flanges (based on one-half width) 8½
Cover plate projection outside of longitudinal row of fasteners
   or connecting welds 8½
Stiffeners 8½
Portion of flange cover plates and flanges of box sections between
   longitudinal rows of rivets, bolts or connecting welds 32
Webs of beams, girders and columns

$$\frac{d}{w} \leq 70 - 100 \frac{P}{P_y} \qquad \text{Formula (25)}$$

with a minimum of 43

SECTION 2.8 LATERAL BRACING

$$l_{cr} = \left(60 - 40 \frac{M}{M_p}\right) r_y \qquad \text{Formula (26)}$$

but not less than 35 r_y.

For Steels with 36,000 psi Specified Yield Point

Applicable to ASTM A36 Structural Steel

Approved Welding Electrodes: E60 Series. Submerged Arc Grade SA-1.
E70 Series. Submerged Arc Grade SA-2.

PART 1

SECTION 1.5   ALLOWABLE UNIT STRESSES

1.5.1   Structural Steel

1.5.1.1   Tension

Tension on net section, except at pin holes..........$F_t = 22,000$ psi
Tension on net section at pin holes................$F_t = 16,000$ psi

1.5.1.2   Shear

Shear on gross section (see Table 3-36 for reduced
values for girder webs)........................$F_v = 14,500$ psi

1.5.1.3   Compression

$$C_c = 126.1$$

For values of F_a given by Formulas (1),
(2) and (3) see Table 1-36.

1.5.1.4   Bending

1.5.1.4.1   Tension and compression for compact,
adequately braced beams having an axis of
symmetry in the plane of loading.................$F_b = 24,000$ psi

1.5.1.4.2   Tension and compression for unsymmetrical
rolled shapes continuously braced in the region
under compression stress........................$F_b = 22,000$ psi

1.5.1.4.3   Tension and compression for box-type
members not included in Sect. 1.5.1.4.1.............$F_b = 22,000$ psi

1.5.1.4.4   Tension for other rolled shapes, built-up
members and plate girders........................$F_b = 22,000$ psi

1.5.1.4.5   Compression, except as provided by Sect.
1.5.1.4.1, 1.5.1.4.2, 1.5.1.4.3, 1.5.1.4.7 and 1.5.1.4.8:
the larger value given by Formulas (4) and (5).

$$F_b = 22,000 - \frac{0.692}{C_b} \left(\frac{l}{r}\right)^2 \qquad \text{Formula (4)}$$

$$F_b = \frac{12,000,000}{ld/A_f} \le 22,000 \text{ psi} \qquad \text{Formula (5)}$$

$F_y = 36$ ksi

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TABLE 1-36

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

$F_y = 36$ ksi

Main and Secondary Members l/r not over 120						Main Members l/r 121 to 200				Secondary Members l/r 121 to 200			
$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (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

TABLE 2

VALUES OF F'_e (KSI)

For use in Formula (6), Sect. 1.6.1, for all grades of steel

$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)
21	338.13	51	57.33	81	22.73	111	12.12	141	7.51	171	5.11
22	308.09	52	55.15	82	22.18	112	11.90	142	7.41	172	5.05
23	281.88	53	53.09	83	21.65	113	11.69	143	7.30	173	4.99
24	258.89	54	51.14	84	21.13	114	11.49	144	7.20	174	4.93
25	238.59	55	49.30	85	20.64	115	11.29	145	7.10	175	4.88
26	220.58	56	47.56	86	20.16	116	11.10	146	7.01	176	4.82
27	204.55	57	45.90	87	19.70	117	10.91	147	6.91	177	4.77
28	190.20	58	44.33	88	19.26	118	10.73	148	6.82	178	4.71
29	177.31	59	42.84	89	18.83	119	10.55	149	6.73	179	4.66
30	165.68	60	41.43	90	18.41	120	10.37	150	6.64	180	4.61
31	155.17	61	40.07	91	18.01	121	10.20	151	6.55	181	4.56
32	145.62	62	38.79	92	17.62	122	10.03	152	6.46	182	4.51
33	136.93	63	37.57	93	17.24	123	9.87	153	6.38	183	4.46
34	128.99	64	36.41	94	16.88	124	9.71	154	6.30	184	4.41
35	121.73	65	35.29	95	16.53	125	9.56	155	6.22	185	4.36
36	115.06	66	34.24	96	16.18	126	9.41	156	6.14	186	4.32
37	108.93	67	33.22	97	15.85	127	9.26	157	6.06	187	4.27
38	103.27	68	32.25	98	15.53	128	9.11	158	5.98	188	4.23
39	98.04	69	31.32	99	15.21	129	8.97	159	5.91	189	4.18
40	93.20	70	30.44	100	14.91	130	8.84	160	5.83	190	4.14
41	88.71	71	29.58	101	14.62	131	8.70	161	5.76	191	4.09
42	84.53	72	28.77	102	14.34	132	8.57	162	5.69	192	4.05
43	80.65	73	27.99	103	14.06	133	8.44	163	5.62	193	4.01
44	77.02	74	27.24	104	13.73	134	8.32	164	5.55	194	3.97
45	73.64	75	26.51	105	13.53	135	8.19	165	5.49	195	3.93
46	70.47	76	25.82	106	13.28	136	8.07	166	5.42	196	3.89
47	67.51	77	25.15	107	13.02	137	7.96	167	5.36	197	3.85
48	64.73	78	24.51	108	12.80	138	7.84	168	5.29	198	3.81
49	62.11	79	23.89	109	12.57	139	7.73	169	5.23	199	3.77
50	59.65	80	23.30	110	12.34	140	7.62	170	5.17	200	3.73

$$F'_e = \frac{149,000,000}{(l/r_b)^2}$$

$F_y = 36$ ksi

TABLE 3-36

ALLOWABLE SHEAR STRESSES (F_v) IN PLATE GIRDERS (KSI)
FOR 36 KSI SPECIFIED YIELD POINT STEEL
(Required Gross Area of Intermediate Stiffeners, as percent of web area, shown in *italics*)

F_y = 36 ksi

Slenderness ratios h/t: web depth to web thickness

h/t	Aspect ratios a/h: stiffener spacing to web depth													
	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	over 3
70											14.3	14.0	13.7	13.1
80							14.2	13.5	13.1	12.8	12.6 *0.7*	12.3 *0.3*	12.1 *0.4*	11.5
90					14.4	13.9	12.6	12.3 *0.6*	12.1 *0.9*	11.9 *1.1*	11.7 *1.2*	11.4 *1.3*	11.2 *1.2*	10.2
100				14.0	13.0	12.4 *0.5*	12.0 *1.4*	11.7 *1.8*	11.4 *2.1*	11.2 *2.1*	11.0 *2.2*	10.4 *2.3*	10.1 *2.1*	8.4
110			14.0	12.7	12.3 *1.0*	12.0 *1.8*	11.6 *2.5*	11.1 *3.1*	10.6 *3.5*	10.3 *3.6*	9.9 *3.6*	9.3 *3.4*	8.9 *3.1*	6.9
120		14.4	12.8	12.3 *1.1*	12.0 *2.1*	11.6 *2.9*	10.9 *4.1*	10.4 *4.7*	9.9 *4.9*	9.5 *4.9*	9.1 *4.8*	8.5 *4.3*	8.0 *3.8*	5.8
130		13.3	12.4 *0.9*	12.0 *2.2*	11.6 *3.2*	11.1 *4.3*	10.4 *5.6*	9.8 *5.9*	9.3 *6.0*	8.9 *5.8*	8.5 *5.6*	7.8 *5.0*	7.4 *4.4*	5.0
140	14.3	12.5 *0.3*	12.1 *1.9*	11.7 *3.2*	11.1 *4.8*	10.6 *5.9*	9.9 *6.7*	9.3 *6.9*	8.8 *6.8*	8.4 *6.6*	8.0 *6.3*	7.3 *5.5*	6.8 *4.9*	4.3
150	13.4	12.3 *1.2*	11.9 *2.8*	11.3 *4.7*	10.8 *6.1*	10.3 *7.1*	9.5 *7.6*	8.9 *7.7*	8.4 *7.5*	8.0 *7.2*	7.6 *6.8*	6.9 *6.0*	6.4 *5.2*	3.7
160	12.6 *0.1*	12.1 *2.1*	11.6 *4.1*	11.0 *6.0*	10.4 *7.2*	10.0 *8.0*	9.2 *8.4*	8.6 *8.3*	8.1 *8.1*	7.7 *7.7*	7.3 *7.3*	6.6 *6.3*		
170	12.4 *0.9*	12.0 *2.8*	11.3 *5.3*	10.7 *7.0*	10.2 *8.1*	9.7 *8.7*	9.0 *9.0*	8.3 *8.9*	7.8 *8.5*	7.4 *8.1*	7.0 *7.7*			
180	12.3 *1.6*	11.7 *4.0*	11.0 *6.4*	10.5 *7.9*	10.0 *8.8*	9.5 *9.4*	8.8 *9.6*	8.1 *9.3*	7.6 *8.9*	7.2 *8.5*	6.8 *8.0*			
200	12.0 *2.9*	11.3 *6.0*	10.7 *8.0*	10.1 *9.2*	9.6 *10.0*	9.2 *10.4*	8.4 *10.4*	7.8 *10.0*	7.3 *9.5*					
220	11.6 *4.8*	10.9 *7.5*	10.4 *9.2*	9.8 *10.2*	9.4 *10.8*	8.9 *11.1*	8.2 *11.0*	7.5 *10.6*						
240	11.3 *6.2*	10.7 *8.6*	10.1 *10.1*	9.6 *11.0*	9.2 *11.5*	8.7 *11.7*	8.0 *11.5*							
260	11.1 *7.3*	10.5 *9.5*	10.0 *10.8*	9.5 *11.6*	9.0 *12.0*									
280	10.9 *8.2*	10.3 *10.2*	9.8 *11.4*	9.3 *12.1*	8.9 *12.4*									
300	10.8 *9.0*	10.2 *10.8*	9.7 *11.8*	9.2 *12.4*										
320	10.7 *9.5*	10.1 *11.2*	9.6 *12.2*	9.1 *12.8*										

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

Fig. 1.5.1.4.5

$F_y = 36$ *ksi*

1.5.1.4.6   Compression for channels: Use Formula (5) above.

1.5.1.4.7   Tension and compression for large pins......F_b = 33,000 psi

1.5.1.4.8   Tension and compression for rectangular bearing plates.................................F_b = 27,000 psi

1.5.1.5   Bearing

1.5.1.5.1   On milled surfaces and pins in reamed, drilled or bored holes.............................F_p = 33,000 psi

1.5.1.5.2   On finished stiffeners......................F_p = 30,000 psi

1.5.1.5.3   On expansion rockers and rollers (in pounds per linear inch).....................................F_p = 760d

SECTION 1.6   COMBINED STRESSES

1.6.1   Axial Compression and Bending

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \dfrac{f_a}{F'_e}\right) F_b} \leq 1.0 \qquad \text{Formula (6)}$$

$$\frac{f_a}{22,000} + \frac{f_b}{F_b} \leq 1.0 \qquad \text{Formula (7)}$$

For values of F_a see Table 1-36.

For values of F'_e see Table 2.

SECTION 1.9   WIDTH-THICKNESS RATIOS

$F_y = 36$ *ksi*

SECTION 1.10   PLATE GIRDERS AND ROLLED BEAMS

1.10.2   Web

Maximum clear distance between flanges $\qquad h = 320t$

1.10.5   Stiffeners

1.10.5.3   For required stiffener spacing and gross area of intermediate stiffeners see Table 3-36.

1.10.5.4   Maximum shear between web and intermediate stiffeners in pounds per linear inch of stiffeners or pair of stiffeners $\qquad f_{vs} = 35h$

1.10.6   Reduction in Flange Stress

When h/t exceeds $24,000/\sqrt{F_b}$ the maximum compression flange stress shall not exceed

$$F_b \left[1.0 - 0.0005 \frac{A_w}{A_f} \left(\frac{h}{t} - \frac{24,000}{\sqrt{F_b}} \right) \right] \qquad \text{Formula (11)}$$

1.10.7   Combined Shear and Tension

$$F_b = 29,500 - 13,500 \left(\frac{f_v}{F_v} \right) \leq 22,000 \, \text{psi} \qquad \text{Formula (12)}$$

1.10.10   Web Crippling

1.10.10.1   Use stiffeners under concentrated interior loads when

$$\frac{R}{t(N + 2k)} \text{ would exceed 27,000 psi} \qquad \text{Formula (13)}$$

and under end reactions when

$$\frac{R}{t(N + k)} \text{ would exceed 27,000 psi} \qquad \text{Formula (14)}$$

1.10.10.2   The compression stress, in pounds per square inch, produced by loads applied to girder webs, except through stiffeners, shall not exceed

$$\left[5.5 + \frac{4}{(a/h)^2} \right] \frac{10,000,000}{(h/t)^2} \qquad \text{Formula (15)}$$

when flange is restrained against rotation; otherwise

$$\left[2 + \frac{4}{(a/h)^2}\right] \frac{10{,}000{,}000}{(h/t)^2}$$ Formula (16)

The compression stresses to be limited by formulas (15) and (16) shall be computed as follows:

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

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

$F_y = 36\ ksi$

PART 2

TABLE 4-36

FOR 36 KSI SPECIFIED YIELD POINT STEEL

$F_y = 36$ ksi

Formula (22)

$$\frac{M_o}{M_p} = B - G\frac{P}{P_y}$$
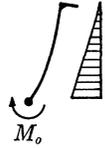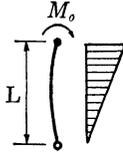
l/r	B	G	l/r	B	G	l/r	B	G
16	1.137	1.173	51	1.163	1.285	86	1.203	1.693
17	1.137	1.176	52	1.164	1.291	87	1.204	1.713
18	1.138	1.179	53	1.165	1.296	88	1.206	1.734
19	1.139	1.182	54	1.166	1.303	89	1.207	1.755
20	1.139	1.184	55	1.166	1.309	90	1.208	1.777
21	1.140	1.187	56	1.167	1.316	91	1.210	1.799
22	1.140	1.189	57	1.168	1.323	92	1.211	1.822
23	1.141	1.192	58	1.170	1.330	93	1.213	1.846
24	1.142	1.194	59	1.171	1.337	94	1.214	1.870
25	1.142	1.196	60	1.172	1.345	95	1.215	1.895
26	1.143	1.199	61	1.173	1.354	96	1.217	1.921
27	1.143	1.201	62	1.174	1.362	97	1.218	1.947
28	1.144	1.204	63	1.175	1.371	98	1.220	1.974
29	1.145	1.206	64	1.176	1.380	99	1.221	2.002
30	1.145	1.209	65	1.177	1.390	100	1.223	2.030
31	1.146	1.211	66	1.178	1.400	101	1.224	2.059
32	1.147	1.214	67	1.179	1.410	102	1.226	2.089
33	1.148	1.216	68	1.180	1.421	103	1.227	2.120
34	1.148	1.219	69	1.181	1.432	104	1.229	2.151
35	1.149	1.222	70	1.183	1.444	105	1.231	2.183
36	1.150	1.225	71	1.184	1.456	106	1.232	2.216
37	1.151	1.228	72	1.185	1.468	107	1.234	2.249
38	1.151	1.231	73	1.186	1.481	108	1.235	2.283
39	1.152	1.234	74	1.187	1.494	109	1.237	2.318
40	1.153	1.237	75	1.189	1.508	110	1.239	2.354
41	1.154	1.241	76	1.190	1.522	111	1.240	2.391
42	1.155	1.244	77	1.191	1.537	112	1.242	2.429
43	1.155	1.248	78	1.192	1.552	113	1.244	2.467
44	1.156	1.252	79	1.194	1.568	114	1.245	2.506
45	1.157	1.256	80	1.195	1.584	115	1.247	2.546
46	1.158	1.260	81	1.196	1.601	116	1.249	2.587
47	1.159	1.265	82	1.197	1.618	117	1.250	2.628
48	1.160	1.270	83	1.199	1.636	118	1.252	2.671
49	1.161	1.275	84	1.200	1.654	119	1.254	2.714
50	1.162	1.280	85	1.201	1.673	120	1.256	2.759

TABLE 5-36

FOR 36 KSI SPECIFIED YIELD POINT STEEL

Formula (23)

$$\frac{M_o}{M_p} = 1.0 - K\left(\frac{P}{P_y}\right) - J\left(\frac{P}{P_y}\right)^2$$

$$M < M_o \qquad M_o \qquad M_o$$

$$M_o \qquad M_o \qquad M < M_o$$

$F_y = 36\ ksi$
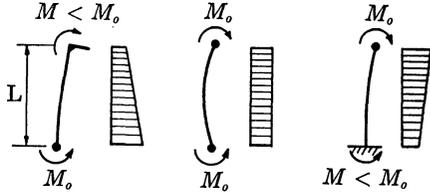
l/r	K	J	l/r	K	J	l/r	K	J
1	.435	.753	41	1.036	.137	81	1.904	− .817
2	.450	.736	42	1.053	.121	82	1.932	− .851
3	.464	.719	43	1.070	.104	83	1.961	− .886
4	.479	.702	44	1.087	.0867	84	1.990	− .922
5	.494	.686	45	1.105	.0692	85	2.020	− .958
6	.508	.670	46	1.122	.0516	86	2.050	− .996
7	.523	.654	47	1.140	.0336	87	2.080	−1.034
8	.537	.638	48	1.158	.0154	88	2.111	−1.072
9	.552	.622	49	1.176	− .0031	89	2.142	−1.112
10	.566	.607	50	1.195	− .0219	90	2.174	−1.152
11	.581	.591	51	1.213	− .0411	91	2.206	−1.193
12	.595	.576	52	1.232	− .0605	92	2.239	−1.234
13	.610	.561	53	1.251	− .0803	93	2.272	−1.277
14	.624	.546	54	1.271	− .100	94	2.306	−1.320
15	.639	.531	55	1.290	− .121	95	2.340	−1.364
16	.653	.516	56	1.310	− .142	96	2.375	−1.409
17	.668	.501	57	1.330	− .163	97	2.410	−1.455
18	.682	.486	58	1.351	− .185	98	2.445	−1.501
19	.697	.472	59	1.371	− .207	99	2.482	−1.549
20	.711	.457	60	1.392	− .229	100	2.518	−1.597
21	.726	.442	61	1.413	− .252	101	2.555	−1.646
22	.741	.428	62	1.435	− .275	102	2.593	−1.696
23	.755	.413	63	1.456	− .299	103	2.631	−1.747
24	.770	.398	64	1.478	− .323	104	2.670	−1.799
25	.785	.384	65	1.501	− .348	105	2.709	−1.852
26	.800	.369	66	1.523	− .373	106	2.749	−1.906
27	.815	.354	67	1.546	− .399	107	2.789	−1.960
28	.830	.340	68	1.570	− .425	108	2.830	−2.016
29	.845	.325	69	1.593	− .452	109	2.871	−2.073
30	.860	.310	70	1.617	− .479	110	2.914	−2.130
31	.876	.295	71	1.641	− .507	111	2.956	−2.189
32	.891	.280	72	1.666	− .535	112	2.999	−2.248
33	.907	.265	73	1.691	− .564	113	3.043	−2.309
34	.922	.249	74	1.716	− .593	114	3.087	−2.371
35	.938	.234	75	1.742	− .623	115	3.132	−2.433
36	.954	.218	76	1.768	− .654	116	3.178	−2.497
37	.970	.202	77	1.794	− .685	117	3.224	−2.562
38	.987	.186	78	1.821	− .717	118	3.271	−2.627
39	1.003	.170	79	1.848	− .750	119	3.318	−2.694
40	1.020	.154	80	1.876	− .783	120	3.366	−2.762

PART 2

LOAD FACTOR

Live plus dead load for simple or continuous beams............1.70
Live plus dead load for continuous frames....................1.85
Live plus dead load plus lateral forces for continuous frames......1.40

SECTION 2.3   COLUMNS

For reduction factors in accordance with Formulas (21), (22) and (23), to be applied to the tabulated M_p value furnished by members subject to axial loading, see Table 4-36 and Table 5-36.

SECTION 2.4   SHEAR

Allowable web shear (psi)

$$V_u \leq 20{,}000 \; wd$$

SECTION 2.6   WIDTH-THICKNESS RATIOS

(Applicable only to elements subject to compression involving plastic hinge rotation under ultimate loading.)

Beam flanges (based on one-half width)........................$8\frac{1}{2}$
Cover plate projection outside of longitudinal row of fasteners or
   connecting welds.......................................$8\frac{1}{2}$
Stiffeners..$8\frac{1}{2}$
Portion of flange cover plates and flanges of box sections between
   longitudinal rows of rivets, bolts or connecting welds.......... 32
Webs of beams, girders and columns

$$\frac{d}{w} \leq 70 - 100 \, \frac{P}{P_y} \qquad \qquad \text{Formula (25)}$$

with a minimum of 43

SECTION 2.8   LATERAL BRACING

$$l_{cr} = \left(60 - 40 \, \frac{M}{M_p} \right) r_y \qquad \qquad \text{Formula (26)}$$

but not less than 35 r_y.

*F*_{*y*} *= 36 ksi*

For Steels with 42,000 psi Specified Yield Point

Applicable to ASTM A242, A440 and A441 Structural Steel over $1\frac{1}{2}$ to 4 inches (inclusive) in thickness.

Approved Welding Electrodes for A242 and A441: E70 Low Hydrogen Series.

Submerged Arc Grade SA-2.

PART 1

SECTION 1.5 ALLOWABLE UNIT STRESSES

1.5.1 Structural Steel

1.5.1.1 Tension

Tension on net section, except at pin holes . . . . F_t = 25,000 psi

Tension on net section at pin holes . . . . F_t = 19,000 psi

1.5.1.2 Shear

Shear on gross section (see Table 3-42 for reduced values for girder webs) . . . . F_v = 17,000 psi

1.5.1.3 Compression

$$C_c = 116.7$$

For values of F_a given by Formulas (1), (2) and (3) see Table 1-42.

1.5.1.4 Bending

1.5.1.4.1 Tension and compression for compact, adequately braced beams having an axis of symmetry in the plane of loading . . . . F_b = 28,000 psi

1.5.1.4.2 Tension and compression for unsymmetrical rolled shapes continuously braced in the region under compression stress . . . . F_b = 25,000 psi

1.5.1.4.3 Tension and compression for box-type members not included in Sect. 1.5.1.4.1 . . . . F_b = 25,000 psi

1.5.1.4.4 Tension for other rolled shapes, built-up members and plate girders . . . . F_b = 25,000 psi

1.5.1.4.5 Compression, except as provided by Sect. 1.5.1.4.1, 1.5.1.4.2, 1.5.1.4.3, 1.5.1.4.7 and 1.5.1.4.8: the larger value given by Formulas (4) and (5).

$$F_b = 25,000 - \frac{0.925}{C_b}\left(\frac{l}{r}\right)^2 \qquad \text{Formula (4)}$$

$$F_b = \frac{12,000,000}{ld/A_f} \le 25,000 \text{ psi} \qquad \text{Formula (5)}$$

$F_y = 42$ ksi

TABLE 1-42

ALLOWABLE STRESS (KSI)

FOR COMPRESSION MEMBERS OF 42 KSI SPECIFIED YIELD POINT STEEL

$F_y = 42$ ksi

Main and Secondary Members l/r not over 120						Main Members l/r 121 to 200				Secondary Members l/r 121 to 200			
$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (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	24.22	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

TABLE 2

VALUES OF F'_e (KSI)

For use in Formula (6), Sect. 1.6.1, for all grades of steel

$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)
21	338.13	51	57.33	81	22.73	111	12.12	141	7.51	171	5.11
22	308.09	52	55.15	82	22.18	112	11.90	142	7.41	172	5.05
23	281.88	53	53.09	83	21.65	113	11.69	143	7.30	173	4.99
24	258.89	54	51.14	84	21.13	114	11.49	144	7.20	174	4.93
25	238.59	55	49.30	85	20.64	115	11.29	145	7.10	175	4.88
26	220.58	56	47.56	86	20.16	116	11.10	146	7.01	176	4.82
27	204.55	57	45.90	87	19.70	117	10.91	147	6.91	177	4.77
28	190.20	58	44.33	88	19.26	118	10.73	148	6.82	178	4.71
29	177.31	59	42.84	89	18.83	119	10.55	149	6.73	179	4.66
30	165.68	60	41.43	90	18.41	120	10.37	150	6.64	180	4.61
31	155.17	61	40.07	91	18.01	121	10.20	151	6.55	181	4.56
32	145.62	62	38.79	92	17.62	122	10.03	152	6.46	182	4.51
33	136.93	63	37.57	93	17.24	123	9.87	153	6.38	183	4.46
34	128.99	64	36.41	94	16.88	124	9.71	154	6.30	184	4.41
35	121.73	65	35.29	95	16.53	125	9.56	155	6.22	185	4.36
36	115.06	66	34.24	96	16.18	126	9.41	156	6.14	186	4.32
37	108.93	67	33.22	97	15.85	127	9.26	157	6.06	187	4.27
38	103.27	68	32.25	98	15.53	128	9.11	158	5.98	188	4.23
39	98.04	69	31.32	99	15.21	129	8.97	159	5.91	189	4.18
40	93.20	70	30.44	100	14.91	130	8.84	160	5.83	190	4.14
41	88.71	71	29.58	101	14.62	131	8.70	161	5.76	191	4.09
42	84.53	72	28.77	102	14.34	132	8.57	162	5.69	192	4.05
43	80.65	73	27.99	103	14.06	133	8.44	163	5.62	193	4.01
44	77.02	74	27.24	104	13.73	134	8.32	164	5.55	194	3.97
45	73.64	75	26.51	105	13.53	135	8.19	165	5.49	195	3.93
46	70.47	76	25.82	106	13.28	136	8.07	166	5.42	196	3.89
47	67.51	77	25.15	107	13.02	137	7.96	167	5.36	197	3.85
48	64.73	78	24.51	108	12.80	138	7.84	168	5.29	198	3.81
49	62.11	79	23.89	109	12.57	139	7.73	169	5.23	199	3.77
50	59.65	80	23.30	110	12.34	140	7.62	170	5.17	200	3.73

$$F'_e = \frac{149,000,000}{(l/r_b)^2}$$

$F_y = 42\ ksi$

TABLE 3-42

ALLOWABLE SHEAR STRESSES (F_v) IN PLATE GIRDERS (KSI)
FOR 42 KSI SPECIFIED YIELD POINT STEEL
(Required Gross Area of Intermediate Stiffeners, as per cent of web area, shown in *italics*)

$F_y = 42$ ksi — 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
70						17.0	17.0	16.7	16.1	15.7	15.5	15.0	14.8	14.2
80				17.0	17.0	16.4	15.3	14.6 *0.1*	14.4 *0.5*	14.2 *0.7*	14.0 *0.8*	13.6 *0.9*	13.4 *0.9*	12.4
90			17.0	16.8	15.5	14.7 *0.1*	14.2 *1.0*	13.8 *1.5*	13.5 *1.8*	13.3 *1.9*	13.1 *1.9*	12.6 *1.9*	12.2 *1.8*	10.4
100		17.0	16.6	15.1	14.4 *0.7*	14.1 *1.5*	13.6 *2.3*	13.2 *2.7*	12.6 *3.2*	12.2 *3.4*	11.8 *3.4*	11.1 *3.2*	10.6 *2.9*	8.4
110		17.0	15.1	14.4 *1.0*	14.0 *2.0*	13.6 *2.7*	12.9 *3.9*	12.2 *4.5*	11.6 *4.7*	11.1 *4.7*	10.7 *4.6*	10.0 *4.2*	9.5 *3.8*	6.9
120	17.0	15.5	14.4 *0.9*	14.0 *2.1*	13.6 *3.2*	13.0 *4.5*	12.1 *5.5*	11.4 *5.9*	10.8 *5.9*	10.4 *5.8*	9.9 *5.6*	9.2 *5.0*	8.6 *4.4*	5.8
130	16.6	14.6 *0.3*	14.1 *2.0*	13.6 *3.3*	13.0 *4.9*	12.4 *6.0*	11.5 *6.8*	10.8 *6.9*	10.2 *6.8*	9.7 *6.6*	9.3 *6.3*	8.5 *5.6*	7.9 *4.9*	5.0
140	15.5	14.3 *1.4*	13.9 *2.9*	13.2 *4.9*	12.5 *6.3*	11.9 *7.2*	11.1 *7.7*	10.4 *7.9*	9.8 *7.6*	9.3 *7.3*	8.8 *6.9*	8.0 *6.0*	7.4 *5.3*	4.3
150	14.6 *0.2*	14.1 *2.2*	13.5 *4.3*	12.7 *6.2*	12.1 *7.4*	11.5 *8.1*	10.7 *8.5*	10.0 *8.5*	9.4 *8.2*	8.9 *7.8*	8.4 *7.4*	7.6 *6.4*	7.0 *5.6*	3.7
160	14.4 *1.1*	13.9 *3.1*	13.1 *5.6*	12.4 *7.3*	11.8 *8.3*	11.2 *8.9*	10.4 *9.2*	9.7 *9.0*	9.1 *8.7*	8.5 *8.2*	8.1 *7.8*	7.2 *6.7*		
170	14.3 *1.8*	13.6 *4.4*	12.8 *6.7*	12.1 *8.1*	11.5 *9.1*	11.0 *9.6*	10.3 *9.7*	9.5 *9.5*	8.9 *9.1*	8.3 *8.6*	7.8 *8.1*			
180	14.1 *2.5*	13.3 *5.5*	12.5 *7.6*	11.9 *8.9*	11.3 *9.8*	10.8 *10.1*	9.9 *10.2*	9.2 *9.9*	8.6 *9.4*	8.0 *8.9*	7.6 *8.3*			
200	13.6 *4.4*	12.8 *7.2*	12.1 *9.0*	11.5 *10.1*	11.0 *10.7*	10.4 *11.0*	9.6 *10.9*	8.9 *10.5*	8.2 *9.9*					
220	13.3 *6.1*	12.5 *8.5*	11.9 *10.0*	11.3 *10.9*	10.7 *11.4*	10.2 *11.6*	9.3 *11.4*	8.6 *10.9*						
240	13.0 *7.3*	12.3 *9.5*	11.6 *10.8*	11.1 *11.6*	10.5 *12.0*	10.0 *12.1*	9.2 *11.8*							
260	12.7 *8.3*	12.1 *10.2*	11.5 *11.4*	10.9 *12.1*	10.4 *12.4*	9.9 *12.5*								
280	12.6 *9.0*	11.9 *10.8*	11.3 *11.9*	10.8 *12.5*	10.2 *12.7*									

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

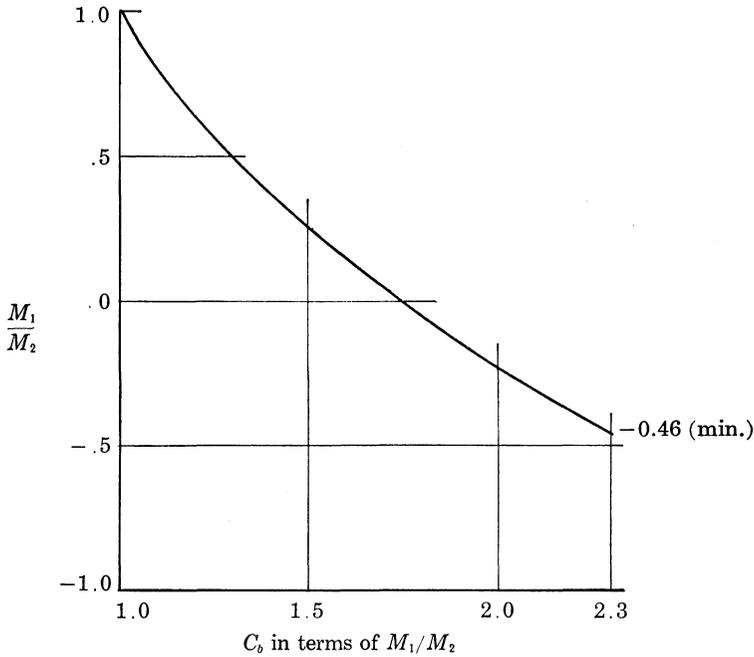$$\frac{M_1}{M_2}$$

C_b in terms of M_1/M_2

Fig. 1.5.1.4.5

−0.46 (min.)

$F_y = 42$ ksi

1.5.1.4.6   Compression for channels: Use Formula (5) above.

1.5.1.4.7   Tension and compression for large pins . . . . . .$F_b = 38{,}000$ psi

1.5.1.4.8   Tension and compression for rectangular bearing plates . . . .$F_b = 31{,}500$ psi

1.5.1.5   Bearing

1.5.1.5.1   On milled surfaces and pins in reamed, drilled or bored holes . . . .$F_p = 38{,}000$ psi

1.5.1.5.2   On finished stiffeners . . . .$F_p = 33{,}500$ psi

1.5.1.5.3   On expansion rockers and rollers (in pounds per linear inch) . . . .$F_p = 960d$

SECTION 1.6   COMBINED STRESSES

1.6.1   Axial Compression and Bending

$$\frac{f_a}{F_a} + \frac{C_m\, f_b}{\left(1 - \dfrac{f_a}{F'_e}\right) F_b} \leq 1.0 \qquad\qquad \text{Formula (6)}$$

$$\frac{f_a}{25{,}000} + \frac{f_b}{F_b} \leq 1.0 \qquad\qquad \text{Formula (7)}$$

For values of F_a see Table 1-42.

For values of F'_e see Table 2.

$F_y = 42$ ksi

SECTION 1.9 WIDTH-THICKNESS RATIOS

SECTION 1.10 PLATE GIRDERS AND ROLLED BEAMS

1.10.2 Web

Maximum clear distance between flanges $\qquad h = 282t$

1.10.5 Stiffeners

1.10.5.3 For required stiffener spacing and gross area of intermediate stiffeners see Table 3-42.

1.10.5.4 Maximum shear between web and intermediate stiffeners in pounds per linear inch of stiffeners or pair of stiffeners $\qquad f_{vs} = 43h$

1.10.6 Reduction in Flange Stress

When h/t exceeds $24{,}000/\sqrt{F_b}$ the maximum compression flange stress shall not exceed

$$F_b \left[1.0 - 0.0005 \frac{A_w}{A_f} \left(\frac{h}{t} - \frac{24{,}000}{\sqrt{F_b}} \right) \right] \qquad \text{Formula (11)}$$

1.10.7 Combined Shear and Tension

$$F_b = 34{,}500 - 15{,}500 \left(\frac{f_v}{F_v} \right) \leq 25{,}000 \text{ psi} \qquad \text{Formula (12)}$$

1.10.10 Web Crippling

1.10.10.1 Use stiffeners under concentrated interior loads when

$$\frac{R}{t(N + 2k)} \text{ would exceed } 31{,}500 \text{ psi} \qquad \text{Formula (13)}$$

and under end reactions when

$$\frac{R}{t(N + k)} \text{ would exceed } 31{,}500 \text{ psi} \qquad \text{Formula (14)}$$

1.10.10.2 The compression stress, in pounds per square inch, produced by loads applied to girder webs, except through stiffeners, shall not exceed

$$\left[5.5 + \frac{4}{(a/h)^2} \right] \frac{10{,}000{,}000}{(h/t)^2} \qquad \text{Formula (15)}$$

when flange is restrained against rotation; otherwise

$$\left[2 + \frac{4}{(a/h)^2} \right] \frac{10{,}000{,}000}{(h/t)^2} \qquad \text{Formula (16)}$$

The compression stresses to be limited by formulas (15) and (16) shall be computed as follows:

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

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

$F_y = 42$ ksi

For Steels with 46,000 psi Specified Yield Point

Applicable to ASTM A242, A440 and A441 Structural Steel over $\frac{3}{4}$ to $1\frac{1}{2}$ inches (inclusive) in thickness.

Approved Welding Electrodes for A242 and A441: E70 Low Hydrogen Series.
Submerged Arc Grade SA-2.

PART 1

SECTION 1.5   ALLOWABLE UNIT STRESSES

1.5.1   Structural Steel

1.5.1.1   Tension

Tension on net section, except at pin holes . . . . $F_t = 27{,}500$ psi

Tension on net section at pin holes . . . . $F_t = 20{,}500$ psi

1.5.1.2   Shear

Shear on gross section (see Table 3-46 for reduced
values for girder webs) . . . . $F_v = 18{,}500$ psi

1.5.1.3   Compression

$$C_c = 111.6$$

For values of F_a given by Formulas (1),
(2) and (3) see Table 1-46.

1.5.1.4   Bending

**1.5.1.4.1   Tension and compression for compact,
adequately braced beams having an axis of
symmetry in the plane of loading** . . . . $F_b = 30{,}500$ psi

**1.5.1.4.2   Tension and compression for unsymmetrical
rolled shapes continuously braced in the region
under compression stress** . . . . $F_b = 27{,}500$ psi

**1.5.1.4.3   Tension and compression for box-type
members not included in Sect. 1.5.1.4.1** . . . . $F_b = 27{,}500$ psi

**1.5.1.4.4   Tension for other rolled shapes, built-up
members and plate girders** . . . . $F_b = 27{,}500$ psi

**1.5.1.4.5   Compression, except as provided by Sect.
1.5.1.4.1, 1.5.1.4.2, 1.5.1.4.3, 1.5.1.4.7 and 1.5.1.4.8:
the larger value given by Formulas (4) and (5)**

$$F_b = 27{,}500 \; - \; \frac{1.110}{C_b} \left(\frac{l}{r}\right)^2 \qquad \text{Formula (4)}$$

$$F_b = \frac{12{,}000{,}000}{ld/A_f} \leq 27{,}500 \text{ psi} \qquad \text{Formula (5)}$$
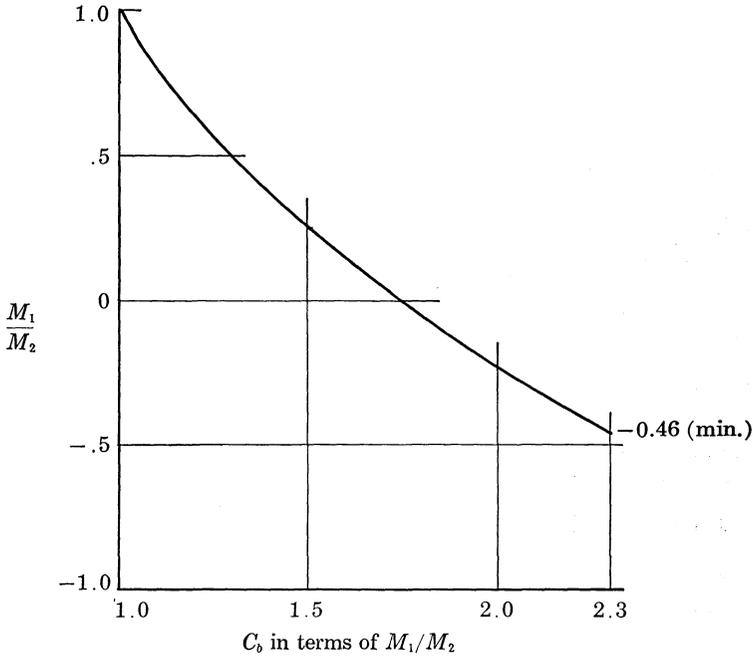
$F_y = 46$ ksi

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Fig. 1.5.1.4.5

1.5.1.4.6 Compression for channels: Use Formula (5) above.

1.5.1.4.7 Tension and compression for large pins......F_b = 41,500 psi

1.5.1.4.8 Tension and compression for rectangular bearing plates..................................F_b = 34,500 psi

1.5.1.5 Bearing

1.5.1.5.1 On milled surfaces and pins in reamed, drilled or bored holes.............................F_p = 41,500 psi

1.5.1.5.2 On finished stiffeners......................F_p = 37,000 psi

1.5.1.5.3 On expansion rockers and rollers (in pounds per linear inch).................................F_p = 1,090d

SECTION 1.6 COMBINED STRESSES

1.6.1 Axial Compression and Bending

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \frac{f_a}{F'_e}\right) F_b} \leq 1.0 \qquad \text{Formula (6)}$$

$$\frac{f_a}{27,500} + \frac{f_b}{F_b} \leq 1.0 \qquad \text{Formula (7)}$$

For values of F_a see Table 1-46.

For values of F'_e see Table 2.

TABLE 1-46

ALLOWABLE STRESS (KSI)
FOR COMPRESSION MEMBERS OF 46 KSI SPECIFIED YIELD POINT STEEL

$F_y = 46$ ksi

Main and Secondary Members l/r not over 120						Main Members l/r 121 to 200				Secondary Members l/r 121 to 200			
$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)
1	27.54	41	23.85	81	17.91	121	10.20	161	5.76	121	10.25	161	7.25
2	27.48	42	23.73	82	17.74	122	10.03	162	5.69	122	10.13	162	7.20
3	27.42	43	23.60	83	17.56	123	9.87	163	5.62	123	10.02	163	7.16
4	27.36	44	23.48	84	17.39	124	9.71	164	5.55	124	9.91	164	7.12
5	27.30	45	23.35	85	17.21	125	9.56	165	5.49	125	9.80	165	7.08
6	27.23	46	23.22	86	17.03	126	9.41	166	5.42	126	9.70	166	7.04
7	27.16	47	23.09	87	16.85	127	9.26	167	5.35	127	9.59	167	7.00
8	27.09	48	22.96	88	16.67	128	9.11	168	5.29	128	9.49	168	6.96
9	27.02	49	22.83	89	16.48	129	8.97	169	5.23	129	9.40	169	6.93
10	26.95	50	22.69	90	16.30	130	8.84	170	5.17	130	9.30	170	6.89
11	26.87	51	22.56	91	16.12	131	8.70	171	5.11	131	9.21	171	6.85
12	26.79	52	22.42	92	15.93	132	8.57	172	5.05	132	9.12	172	6.82
13	26.72	53	22.28	93	15.74	133	8.44	173	4.99	133	9.03	173	6.79
14	26.63	54	22.14	94	15.55	134	8.32	174	4.93	134	8.94	174	6.76
15	26.55	55	22.00	95	15.36	135	8.19	175	4.88	135	8.86	175	6.73
16	26.47	56	21.86	96	15.17	136	8.07	176	4.82	136	8.78	176	6.70
17	26.38	57	21.72	97	14.97	137	7.96	177	4.77	137	8.70	177	6.67
18	26.29	58	21.57	98	14.78	138	7.84	178	4.71	138	8.62	178	6.64
19	26.21	59	21.43	99	14.58	139	7.73	179	4.66	139	8.54	179	6.61
20	26.11	60	21.28	100	14.39	140	7.62	180	4.61	140	8.47	180	6.58
21	26.02	61	21.13	101	14.19	141	7.51	181	4.56	141	8.39	181	6.56
22	25.93	62	20.98	102	13.99	142	7.41	182	4.51	142	8.32	182	6.53
23	25.83	63	20.83	103	13.79	143	7.30	183	4.46	143	8.25	183	6.51
24	25.73	64	20.68	104	13.58	144	7.20	184	4.41	144	8.18	184	6.49
25	25.64	65	20.53	105	13.38	145	7.10	185	4.36	145	8.12	185	6.46
26	25.54	66	20.37	106	13.17	146	7.01	186	4.32	146	8.05	186	6.44
27	25.43	67	20.22	107	12.96	147	6.91	187	4.27	147	7.99	187	6.42
28	25.33	68	20.06	108	12.75	148	6.82	188	4.23	148	7.93	188	6.40
29	25.23	69	19.90	109	12.54	149	6.73	189	4.18	149	7.87	189	6.38
30	25.12	70	19.74	110	12.33	150	6.64	190	4.14	150	7.81	190	6.36
31	25.01	71	19.58	111	12.12	151	6.55	191	4.09	151	7.75	191	6.35
32	24.90	72	19.42	112	11.90	152	6.46	192	4.05	152	7.69	192	6.33
33	24.79	73	19.26	113	11.69	153	6.38	193	4.01	153	7.64	193	6.31
34	24.68	74	19.10	114	11.49	154	6.30	194	3.97	154	7.59	194	6.30
35	24.56	75	18.93	115	11.29	155	6.22	195	3.93	155	7.53	195	6.28
36	24.45	76	18.76	116	11.10	156	6.14	196	3.89	156	7.48	196	6.27
37	24.33	77	18.60	117	10.91	157	6.06	197	3.85	157	7.43	197	6.26
38	24.21	78	18.43	118	10.72	158	5.98	198	3.81	158	7.39	198	6.24
39	24.10	79	18.26	119	10.55	159	5.91	199	3.77	159	7.34	199	6.23
40	23.97	80	18.08	120	10.37	160	5.83	200	3.73	160	7.29	200	6.22

TABLE 2

VALUES OF F'_e (KSI)

For use in Formula (6), Sect. 1.6.1, for all grades of steel

$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)
21	338.13	51	57.33	81	22.73	111	12.12	141	7.51	171	5.11
22	308.09	52	55.15	82	22.18	112	11.90	142	7.41	172	5.05
23	281.88	53	53.09	83	21.65	113	11.69	143	7.30	173	4.99
24	258.89	54	51.14	84	21.13	114	11.49	144	7.20	174	4.93
25	238.59	55	49.30	85	20.64	115	11.29	145	7.10	175	4.88
26	220.58	56	47.56	86	20.16	116	11.10	146	7.01	176	4.82
27	204.55	57	45.90	87	19.70	117	10.91	147	6.91	177	4.77
28	190.20	58	44.33	88	19.26	118	10.73	148	6.82	178	4.71
29	177.31	59	42.84	89	18.83	119	10.55	149	6.73	179	4.66
30	165.68	60	41.43	90	18.41	120	10.37	150	6.64	180	4.61
31	155.17	61	40.07	91	18.01	121	10.20	151	6.55	181	4.56
32	145.62	62	38.79	92	17.62	122	10.03	152	6.46	182	4.51
33	136.93	63	37.57	93	17.24	123	9.87	153	6.38	183	4.46
34	128.99	64	36.41	94	16.88	124	9.71	154	6.30	184	4.41
35	121.73	65	35.29	95	16.53	125	9.56	155	6.22	185	4.36
36	115.06	66	34.24	96	16.18	126	9.41	156	6.14	186	4.32
37	108.93	67	33.22	97	15.85	127	9.26	157	6.06	187	4.27
38	103.27	68	32.25	98	15.53	128	9.11	158	5.98	188	4.23
39	98.04	69	31.32	99	15.21	129	8.97	159	5.91	189	4.18
40	93.20	70	30.44	100	14.91	130	8.84	160	5.83	190	4.14
41	88.71	71	29.58	101	14.62	131	8.70	161	5.76	191	4.09
42	84.53	72	28.77	102	14.34	132	8.57	162	5.69	192	4.05
43	80.65	73	27.99	103	14.06	133	8.44	163	5.62	193	4.01
44	77.02	74	27.24	104	13.73	134	8.32	164	5.55	194	3.97
45	73.64	75	26.51	105	13.53	135	8.19	165	5.49	195	3.93
46	70.47	76	25.82	106	13.28	136	8.07	166	5.42	196	3.89
47	67.51	77	25.15	107	13.02	137	7.96	167	5.36	197	3.85
48	64.73	78	24.51	108	12.80	138	7.84	168	5.29	198	3.81
49	62.11	79	23.89	109	12.57	139	7.73	169	5.23	199	3.77
50	59.65	80	23.30	110	12.34	140	7.62	170	5.17	200	3.73

$$F'_e = \frac{149{,}000{,}000}{(l/r_b)^2}$$

$F_y = 46\ ksi$

TABLE 3-46

ALLOWABLE SHEAR STRESSES (F_v) IN PLATE GIRDERS (KSI)
FOR 46 KSI SPECIFIED YIELD POINT STEEL

(Required Gross Area of Intermediate Stiffeners, as per cent of web area, shown in *italics*)

	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							18.5	18.5	18.5	18.5	18.5	18.3	18.0	17.3
70					18.5	18.5	18.3	17.5	16.9	16.5	16.2	15.8	15.3	14.9
												0.2	*0.3*	
80				18.5	18.3	17.2	16.1	15.7	15.4	15.1	14.9	14.5	14.2	13.0
							0.1	*0.7*	*1.0*	*1.2*	*1.3*	*1.3*	*1.2*	
90			18.5	17.6	16.3	15.8	15.3	14.8	14.5	14.2	13.8	13.1	12.6	10.4
						0.1	*1.6*	*2.0*	*2.2*	*2.3*	*2.5*	*2.5*	*2.3*	
100		18.5	17.4	16.0	15.6	15.2	14.6	13.9	13.3	12.8	12.4	11.6	11.0	8.4
				0.3	*1.3*	*2.1*	*2.9*	*3.6*	*4.0*	*4.0*	*4.0*	*3.7*	*3.3*	
110		17.8	16.0	15.5	15.1	14.6	13.6	12.9	12.3	11.7	11.3	10.5	9.9	6.9
			0.3	*1.6*	*2.6*	*3.6*	*4.8*	*5.3*	*5.4*	*5.3*	*5.2*	*4.6*	*4.1*	
120	18.5	16.3	15.6	15.1	14.5	13.8	12.9	12.1	11.5	10.9	10.5	9.6	9.0	5.8
		1.5	*2.7*	*4.2*	*5.4*	*6.2*	*6.5*	*6.5*	*6.3*	*6.0*	*5.3*	*4.7*		
130	17.4	15.8	15.3	14.6	13.9	13.2	12.3	11.5	10.9	10.3	9.9	9.0	8.3	5.0
		1.0	*2.5*	*4.3*	*5.8*	*6.7*	*7.4*	*7.5*	*7.3*	*7.0*	*6.7*	*5.9*	*5.1*	
140	16.2	15.5	14.9	14.1	13.4	12.8	11.8	11.1	10.4	9.8	9.4	8.4	7.8	4.3
		1.9	*3.8*	*5.8*	*7.0*	*7.8*	*8.3*	*8.2*	*8.0*	*7.6*	*7.2*	*6.3*	*5.5*	
150	15.9	15.3	14.5	13.7	13.0	12.4	11.5	10.7	10.0	9.4	9.0	8.0	7.4	3.7
	0.8	*2.8*	*5.3*	*7.0*	*8.0*	*8.7*	*9.0*	*8.9*	*8.5*	*8.1*	*7.7*	*6.7*	*5.8*	
160	15.7	14.9	14.1	13.4	12.7	12.1	11.2	10.4	9.7	9.1	8.6	7.7		
	1.7	*4.1*	*6.4*	*7.9*	*8.9*	*9.4*	*9.6*	*9.4*	*9.0*	*8.5*	*8.0*	*6.9*		
170	15.5	14.6	13.8	13.1	12.4	11.8	10.9	10.1	9.4	8.9	8.4			
	2.4	*5.3*	*7.4*	*8.7*	*9.6*	*10.0*	*10.1*	*9.8*	*9.3*	*8.8*	*8.3*			
180	15.2	14.3	13.5	12.8	12.2	11.6	10.7	9.9	9.2	8.6	8.1			
	3.2	*6.3*	*8.2*	*9.4*	*10.1*	*10.5*	*10.5*	*10.1*	*9.6*	*9.1*	*8.5*			
200	14.7	13.9	13.1	12.5	11.9	11.3	10.4	9.6	8.9					
	5.3	*7.9*	*9.5*	*10.5*	*11.0*	*11.3*	*11.2*	*10.7*	*10.1*					
220	14.3	13.6	12.9	12.2	11.6	11.1	10.1	9.3						
	6.7	*9.0*	*10.4*	*11.3*	*11.7*	*11.9*	*11.6*	*11.1*						
240	14.0	13.3	12.6	12.0	11.4	10.9	9.9							
	7.9	*9.9*	*11.2*	*11.9*	*12.2*	*12.3*	*12.0*							
260	13.8	13.1	12.5	11.8	11.3	10.7								
	8.8	*10.6*	*11.7*	*12.3*	*12.6*	*12.7*								

(Left vertical axis: Slenderness ratios h/t: web depth to web thickness)

(Side label: $F_y = 46$ ksi)

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

SECTION 1.9 WIDTH-THICKNESS RATIOS

SECTION 1.10 PLATE GIRDERS AND ROLLED BEAMS

1.10.2 Web

Maximum clear distance between flanges $h = 260t$

1.10.5 Stiffeners

1.10.5.3 For required stiffener spacing and gross area of intermediate stiffeners see Table 3-46.

1.10.5.4 Maximum shear between web and intermediate stiffeners in pounds per linear inch of stiffeners or pair of stiffeners $f_{vs} = 50h$

1.10.6 Reduction in Flange Stress

When h/t exceeds $24{,}000/\sqrt{F_b}$ the maximum compression flange stress shall not exceed

$$F_b \left[1.0 \,-\, 0.0005 \, \frac{A_w}{A_f} \left(\frac{h}{t} \,-\, \frac{24{,}000}{\sqrt{F_b}} \right) \right] \qquad \text{Formula (11)}$$

1.10.7 Combined Shear and Tension

$$F_b \,=\, 38{,}000 \,-\, 17{,}000 \left(\frac{f_v}{F_v} \right) \leq 27{,}500 \text{ psi} \qquad \text{Formula (12)}$$

1.10.10 Web Crippling

1.10.10.1 Use stiffeners under concentrated interior loads when

$$\frac{R}{t(N + 2k)} \text{ would exceed 34,500 psi} \qquad \text{Formula (13)}$$

and under end reactions when

$$\frac{R}{t(N + k)} \text{ would exceed 34,500 psi} \qquad \text{Formula (14)}$$

1.10.10.2 The compression stress, in pounds per square inch, produced by loads applied to girder webs, except through stiffeners, shall not exceed

$$\left[5.5 \,+\, \frac{4}{(a/h)^2} \right] \frac{10{,}000{,}000}{(h/t)^2} \qquad \text{Formula (15)}$$

$F_y = 46$ ksi

when flange is restrained against rotation; otherwise

$$\left[2 + \frac{4}{(a/h)^2}\right] \frac{10,000,000}{(h/t)^2} \qquad \text{Formula (16)}$$

The compression stresses to be limited by formulas (15) and (16) shall be computed as follows:

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

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

F_y = 46 ksi

For Steels with 50,000 psi Specified Yield Point

Applicable to ASTM A242, A440 and A441 Structural Steel $3/4$ inch and less in thickness

Approved Welding Electrodes for A242 and A441: E70 Low Hydrogen Series.
Submerged Arc Grade SA-2.

PART 1

SECTION 1.5 ALLOWABLE UNIT STRESSES

1.5.1 Structural Steel

1.5.1.1 Tension

Tension on net section, except at pin holes..........$F_t = 30,000$ psi
Tension on net section at pin holes................$F_t = 22,500$ psi

1.5.1.2 Shear

Shear on gross section (see Table 3-50 for reduced
values for girder webs)............................$F_v = 20,000$ psi

1.5.1.3 Compression

$$C_c = 107.0$$

For values of F_a given by Formulas (1),
(2) and (3) see Table 1-50.

1.5.1.4 Bending

1.5.1.4.1 Tension and compression for compact,
adequately braced beams having an axis of
symmetry in the plane of loading..................$F_b = 33,000$ psi
1.5.1.4.2 Tension and compression for unsymmetrical
rolled shapes continuously braced in the region
under compression stress..........................$F_b = 30,000$ psi
1.5.1.4.3 Tension and compression for box-type
members not included in Sect. 1.5.1.4.1............$F_b = 30,000$ psi
1.5.1.4.4 Tension for other rolled shapes, built-up
members and plate girders.........................$F_b = 30,000$ psi
1.5.1.4.5 Compression, except as provided by Sect.
1.5.1.4.1, 1.5.1.4.2, 1.5.1.4.3, 1.5.1.4.7 and 1.5.1.4.8:
the larger value given by Formulas (4) and (5).

$$F_b = 30,000 - \frac{1.310}{C_b} \left(\frac{l}{r}\right)^2 \qquad \text{Formula (4)}$$

$$F_b = \frac{12,000,000}{ld/A_f} \leq 30,000 \text{ psi} \qquad \text{Formula (5)}$$

$F_y = 50$ ksi

TABLE 1-50

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

Main and Secondary Members l/r not over 120						Main Members l/r 121 to 200				Secondary Members l/r 121 to 200			
$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (ksi)	$\frac{l}{r}$	F_a (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

$F_y = 50\ ksi$

TABLE 2
VALUES OF F'_e (KSI)
For use in Formula (6), Sect. 1.6.1, for all grades of steel

$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)	$\frac{l}{r}$	F'_e (ksi)
21	338.13	51	57.33	81	22.73	111	12.12	141	7.51	171	5.11
22	308.09	52	55.15	82	22.18	112	11.90	142	7.41	172	5.05
23	281.88	53	53.09	83	21.65	113	11.69	143	7.30	173	4.99
24	258.89	54	51.14	84	21.13	114	11.49	144	7.20	174	4.93
25	238.59	55	49.30	85	20.64	115	11.29	145	7.10	175	4.88
26	220.58	56	47.56	86	20.16	116	11.10	146	7.01	176	4.82
27	204.55	57	45.90	87	19.70	117	10.91	147	6.91	177	4.77
28	190.20	58	44.33	88	19.26	118	10.73	148	6.82	178	4.71
29	177.31	59	42.84	89	18.83	119	10.55	149	6.73	179	4.66
30	165.68	60	41.43	90	18.41	120	10.37	150	6.64	180	4.61
31	155.17	61	40.07	91	18.01	121	10.20	151	6.55	181	4.56
32	145.62	62	38.79	92	17.62	122	10.03	152	6.46	182	4.51
33	136.93	63	37.57	93	17.24	123	9.87	153	6.38	183	4.46
34	128.99	64	36.41	94	16.88	124	9.71	154	6.30	184	4.41
35	121.73	65	35.29	95	16.53	125	9.56	155	6.22	185	4.36
36	115.06	66	34.24	96	16.18	126	9.41	156	6.14	186	4.32
37	108.93	67	33.22	97	15.85	127	9.26	157	6.06	187	4.27
38	103.27	68	32.25	98	15.53	128	9.11	158	5.98	188	4.23
39	98.04	69	31.32	99	15.21	129	8.97	159	5.91	189	4.18
40	93.20	70	30.44	100	14.91	130	8.84	160	5.83	190	4.14
41	88.71	71	29.58	101	14.62	131	8.70	161	5.76	191	4.09
42	84.53	72	28.77	102	14.34	132	8.57	162	5.69	192	4.05
43	80.65	73	27.99	103	14.06	133	8.44	163	5.62	193	4.01
44	77.02	74	27.24	104	13.73	134	8.32	164	5.55	194	3.97
45	73.64	75	26.51	105	13.53	135	8.19	165	5.49	195	3.93
46	70.47	76	25.82	106	13.28	136	8.07	166	5.42	196	3.89
47	67.51	77	25.15	107	13.02	137	7.96	167	5.36	197	3.85
48	64.73	78	24.51	108	12.80	138	7.84	168	5.29	198	3.81
49	62.11	79	23.89	109	12.57	139	7.73	169	5.23	199	3.77
50	59.65	80	23.30	110	12.34	140	7.62	170	5.17	200	3.73

$$F'_e = \frac{149{,}000{,}000}{(l/r_b)^2}$$

$F_y = 50$ ksi

TABLE 3-50

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

(Required Gross Area of Intermediate Stiffeners, as per cent of web area, shown in *italics*)

h/t	Aspect ratios a/h: stiffener spacing to web depth														
	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	over 3
60							20.0	20.0	20.0	20.0	19.7	19.1	18.8	18.1
70					20.0	20.0	19.1	18.2	17.6	17.3 *0.2*	17.1 *0.4*	16.7 *0.6*	16.5 *0.6*	15.5
80				20.0	19.1	17.9	17.1 *0.6*	16.7 *1.2*	16.4 *1.4*	16.1 *1.6*	15.8 *1.6*	15.4 *1.6*	15.0 *1.5*	13.1
90			20.0	18.3	17.3 *0.4*	16.9 *1.3*	16.3 *2.1*	15.8 *2.5*	15.3 *2.8*	14.8 *3.1*	14.3 *3.1*	13.5 *3.0*	13.0 *2.8*	10.4
100		20.0	18.1	17.2 *0.9*	16.7 *1.9*	16.3 *2.6*	15.4 *3.8*	14.6 *4.4*	13.9 *4.6*	13.4 *4.6*	12.9 *4.5*	12.0 *4.1*	11.4 *3.7*	8.4
110	20.0	18.5	17.2 *0.9*	16.7 *2.2*	16.2 *3.2*	15.4 *4.5*	14.4 *5.5*	13.6 *5.9*	12.9 *5.9*	12.3 *5.8*	11.8 *5.6*	10.9 *5.0*	10.3 *4.4*	6.9
120	19.7	17.4 *0.4*	16.8 *2.0*	16.2 *3.4*	15.4 *5.0*	14.7 *6.1*	13.7 *6.9*	12.8 *7.0*	12.1 *6.9*	11.5 *6.7*	11.0 *6.4*	10.1 *5.8*	9.4 *4.9*	5.8
130	18.2	17.0 *1.5*	16.4 *3.1*	15.6 *5.1*	14.8 *6.5*	14.1 *7.4*	13.1 *7.9*	12.2 *7.9*	11.5 *7.7*	10.9 *7.4*	10.4 *7.0*	9.4 *6.1*	8.7 *5.3*	5.0
140	17.3 *0.5*	16.7 *2.5*	15.9 *4.7*	15.1 *6.5*	14.3 *7.7*	13.6 *8.4*	12.6 *8.7*	11.8 *8.6*	11.0 *8.3*	10.4 *7.9*	9.9 *7.5*	8.9 *6.5*	8.2 *5.7*	4.3
150	17.1 *1.4*	16.4 *3.6*	15.5 *6.0*	14.6 *7.6*	13.9 *8.6*	13.3 *9.2*	12.2 *9.4*	11.4 *9.2*	10.7 *8.8*	10.0 *8.4*	9.5 *7.9*	8.5 *6.8*	7.7 *5.9*	3.7
160	16.9 *2.2*	16.0 *4.9*	15.1 *7.1*	14.3 *8.5*	13.6 *9.4*	13.0 *9.8*	11.9 *9.9*	11.1 *9.7*	10.3 *9.2*	9.7 *8.7*	9.2 *8.2*	8.1 *7.1*		
170	16.7 *2.9*	15.6 *6.0*	14.8 *8.0*	14.0 *9.2*	13.3 *10.0*	12.7 *10.4*	11.7 *10.4*	10.8 *10.0*	10.1 *9.6*	9.5 *9.0*	8.9 *8.5*			
180	16.3 *4.1*	15.4 *7.0*	14.5 *8.8*	13.8 *10.0*	13.1 *10.5*	12.5 *10.9*	11.5 *10.8*	10.6 *10.4*	9.9 *9.8*	9.2 *9.3*	8.7 *8.7*			
200	15.8 *5.9*	14.9 *8.4*	14.1 *9.9*	13.4 *10.8*	12.8 *11.4*	12.2 *11.6*	11.1 *11.4*	10.3 *10.9*	9.5 *10.3*					
220	15.4 *7.3*	14.6 *9.5*	13.8 *10.8*	13.2 *11.6*	12.5 *12.0*	11.9 *12.1*	10.9 *11.8*	10.0 *11.3*						
240	15.1 *8.3*	14.3 *10.3*	13.6 *11.5*	13.0 *12.1*	12.3 *12.4*	11.7 *12.5*	10.7 *12.1*							

(Leftmost axis label: Slenderness ratios h/t: web depth to web thickness)

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

$F_y = 50$ ksi

Fig. 1.5.1.4.5

1.5.1.4.6   Compression for channels: Use formula (5) above.

1.5.1.4.7   Tension and compression for large pins......$F_b = 45,000$ psi

1.5.1.4.8   Tension and compression for rectangular
bearing plates......................................$F_b = 37,500$ psi

1.5.1.5   Bearing

1.5.1.5.1   On milled surfaces and pins in reamed,
drilled or bored holes.............................$F_p = 45,000$ psi

1.5.1.5.2   On finished stiffeners......................$F_p = 40,000$ psi

1.5.1.5.3   On expansion rockers and rollers (in pounds
per linear inch)...................................$F_p = 1,220d$

SECTION 1.6   COMBINED STRESSES

1.6.1   Axial Compression and Bending

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \dfrac{f_a}{F'_e}\right) F_b} \leq 1.0 \qquad \text{Formula (6)}$$

$$\frac{f_a}{30,000} + \frac{f_b}{F_b} \leq 1.0 \qquad \text{Formula (7)}$$

For values of F_a see Table 1-50.
For values of F'_e see Table 2.

$F_y = 50$ ksi

SECTION 1.9   WIDTH-THICKNESS RATIOS

SECTION 1.10   PLATE GIRDERS AND ROLLED BEAMS

1.10.2   Web

Maximum clear distance between flanges $\qquad h = 243t$

1.10.5   Stiffeners

1.10.5.3   For required stiffener spacing and gross area of intermediate stiffeners see Table 3-50.

1.10.5.4   Maximum shear between web and intermediate stiffeners, in pounds per linear inch of stiffeners or pair of stiffeners $\qquad f_{vs} = 56h$

1.10.6   Reduction in Flange Stress

When h/t exceeds $24{,}000/\sqrt{F_b}$ the maximum compression flange stress shall not exceed

$$F_b \left[1.0 \ - \ 0.0005 \ \frac{A_w}{A_f} \left(\frac{h}{t} - \frac{24{,}000}{\sqrt{F_b}} \right) \right] \qquad \text{Formula (11)}$$

1.10.7   Combined Shear and Tension

$$F_b \ = \ 41{,}000 \ - \ 18{,}500 \left(\frac{f_v}{F_v} \right) \le 30{,}000 \text{ psi} \qquad \text{Formula (12)}$$

1.10.10   Web Crippling

1.10.10.1   Use stiffeners under concentrated interior loads when

$$\frac{R}{t(N + 2k)} \text{ would exceed } 37{,}500 \text{ psi} \qquad \text{Formula (13)}$$

and under end reactions when

$$\frac{R}{t(N + k)} \text{ would exceed } 37{,}500 \text{ psi} \qquad \text{Formula (14)}$$

1.10.10.2   The compression stress, in pounds per square inch, produced by loads applied to girder webs, except through stiffeners, shall not exceed

$$\left[5.5 + \frac{4}{(a/h)^2} \right] \frac{10{,}000{,}000}{(h/t)^2} \qquad \text{Formula (15)}$$

$F_y = 50$ ksi

when flange is restrained against rotation; otherwise

$$\left[2 + \frac{4}{(a/h)^2}\right]\frac{10,000,000}{(h/t)^2} \qquad \text{Formula (16)}$$

The compression stresses to be limited by formulas (15) and (16) shall be computed as follows:

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

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

$F_y = 50 \text{ ksi}$
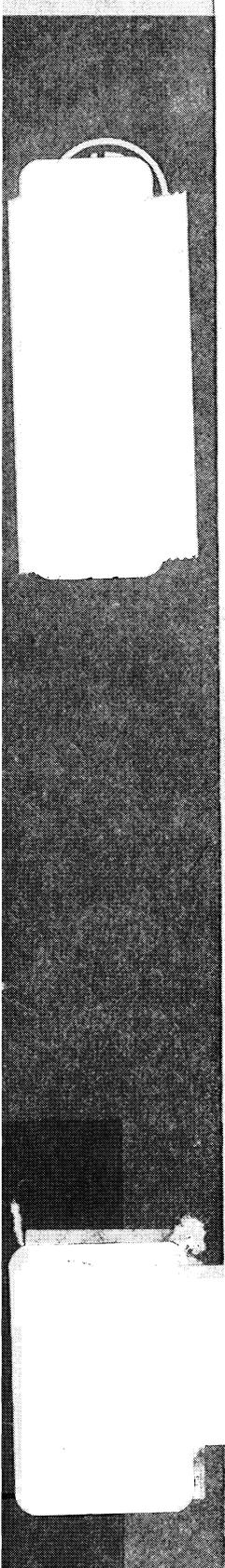
COMMENTARY

ON THE

SPECIFICATION

FOR THE

DESIGN,

FABRICATION

& ERECTION

OF

STRUCTURAL

STEEL FOR

BUILDINGS

NOVEMBER 30, 1961

AMERICAN INSTITUTE
OF STEEL CONSTRUCTION
101 PARK AVENUE, NEW YORK 17, N.Y.

COMMENTARY

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.

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.

Part 1 of the Specification includes all of the provisions necessary for a working-stress design covering all three types of construction. It corresponds to the earlier AISC Specification in force when the *Supplementary Rules for Plastic Design and Fabrication* were issued by the American Institute of Steel Construction in December, 1958. Part 2 of the Specification consists of these Rules with some minor modifications.

SECTION 1.3 LOADS AND FORCES

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

SECTION 1.4 MATERIALS

The increasing use of high strength steels no longer permits the continuation of a standard design specification based upon the exclusive use of one strength grade of steel. However, the unrestricted acceptance of all steels is not desirable. Physical properties are not the sole measure of acceptability. Metallurgical properties, which affect both fabrication and serviceability, must also be considered.

The steels permitted by the Specification afford as much as a 50% increase in strength as compared with the older A7 grade and offer no particular problems in their proper utilization. They by no means cover the entire range of steels, however, and undoubtedly in time others will be added.

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It should be noted that the increase in yield point above 36,000 pounds per square inch is governed by the thickness of the component being considered. For material no thicker than $\frac{3}{4}$ inch a yield point of 50,000 pounds per square inch is available, while for material ranging between $\frac{3}{4}$ inch and $1\frac{1}{2}$ inch in thickness the comparable specified minimum yield point is 46,000 pounds per square inch and for material over $1\frac{1}{2}$ inch up to 4 inch in thickness it is 42,000 pounds per square inch.

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 have been added.

SECTION 1.5   ALLOWABLE UNIT STRESSES

1.5.1   Structural Steel

Because of the introduction of steels having a specified minimum yield point other than 33,000 pounds per square inch it is convenient to express permissible working stresses in terms of yield point F_y. For ready reference, numerical values are presented in an Appendix for each of the yield points represented in Sect. 1.4.1. Since any greater precision would be unwarranted, these are presented in round numbers which are easily remembered, except where they have to be given in tabular form.

1.5.1.1   Tension

The same factor of safety with respect to yield point stress heretofore recommended for A7 steel has been used in determining the basic working stress for the newer and stronger steels. A working stress at the net section at pin holes has been added, based upon research* and experience with eye-bars.

1.5.1.2   Shear

No change has been made in the recommended working stress for shear except in the case of slender girder webs discussed under Sect. 1.10.

While the shear yield point of structural steel has been variously estimated as between one-half and five-eighths of the tension and compression yield point 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 tensile stress, substantially as it has been since the first edition of the AISC Specification was 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

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* Pin-Connected Plate Links, 1939 *ASCE Transactions*.

webs lie in a common plane. Such webs should be reinforced when the web thickness is less than

$$\frac{32,000M}{A_{bc}F_y}$$

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 maximum thickness of the web not requiring reinforcement can be computed from the equation

$$\text{allowable shear stress} = 0.40F_y = \frac{12M}{0.95A_{bc}\, w_{\max}}$$

1.5.1.3 Compression

1.5.1.3.1 The new Formula (1), for columns whose mode of failure is by inelastic buckling, like that for slender columns, is 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 point of a given grade of structural steel, as

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

A varied 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. Similar provisions have been included in the British and German design standards for some time and can be justified by the insensitivity of such members to accidental eccentricities. For longer columns, approaching the Euler slenderness range, the factor of safety is increased 15 percent, to approximately the value used in earlier recommendations. 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 abscissae are the ratio of given l/r values to the limiting value C_c, and whose ordinates vary from 1.67 when l/r equals 0 to 1.92 when l/r equals C_c.

While the new formula is somewhat more complex than heretofore, it permits a more economical use of material in relatively short columns. Tables giving the permissible stress for columns and other compression members for each of the approved structural steels are included in the Appendix to the Specification for the convenience of the designer.

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* *Guide to Design Criteria for Metal Compression Members*, Eq. (2.19) and (2.2).

1.5.1.3.2   Formula (2), covering columns slender enough to fail by elastic buckling, is based upon a constant factor of safety of 1.92 with respect to the elastic (Euler) column strength.   Allowable working stresses given by Formula (2) are substantially the same as those given by the more complex Rankine-Gordon formula which in the past were reduced by the factor $\left(1.6 - \dfrac{l}{200r}\right)$ for main compression members.

1.5.1.3.3   By dividing the values obtained from Formulas (1) and (2) by the factor $\left(1.6 - \dfrac{l}{200r}\right)$ when l/r exceeds 120, to obtain Formula (3), substantially the same allowable stresses are still recommended for bracing and secondary members as those formerly given by the Rankine-Gordon formula which has been included in the AISC Specification since its first adoption in 1923.   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 (3) takes advantage of end restraint, the full unbraced length of the member 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 Sect. 1.9 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-point 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 "compact" sections, i.e., those meeting the provisions of Sect. 2.6, before the full plastic moment is reached.   Practically all WF- and I-shapes meet these provisions.   It is obvious, therefore, 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 WF- and I-beams is generally in excess of 1.12, the allowable bending stress for such members has been raised 10 percent from $0.60F_y$ to $0.66F_y$.

The compression flange of such shapes may be considered as adequately supported laterally when the distance between bracing points is as much as 13 times the width of the flange.

Also reflecting the results of the research on ultimate strength of structures is the further provision in Sect. 1.5.1.4.1 permitting a limited redistribution of moments produced by gravity loading.   Taken in conjunction with the 10 percent increase in bending stress, permission to proportion flexural members for nine-tenths of the negative moment produced by gravity loads at points of support affords the same reduction in required bending strength as was provided by the 20 percent stress increase provision in the previous

edition of the Specification. However, it is now limited to compact shapes having an axis of symmetry in the plane of loading and subject to only minor concurrent axial stress. Also, it is now required that beams, proportioned for nine-tenths of the calculated moment at reaction points instead of the full computed moment, have sufficient bending strength to resist the maximum calculated moment between supports, increased by one-tenth of the average of reaction point moments, without exceeding the allowable bending stress. Fig. C 1.5.1 illustrates the application of this latter provision by comparing calculated moment diagrams with the diagrams as altered by this provision.

In order to assure better 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 Members asymmetrical about the plane of loading but having continuous lateral support in regions of compression stress may be designed for the full basic stress. They cannot be treated in the same manner as other compact members as in Sect. 1.5.1.4.1, however.

1.5.1.4.3 Box-type members, even those whose width-thickness ratios are such that they cannot be classified as compact members, are torsionally very stiff. Hence, reduction of the full stress, as provided by Formulas (4) and (5) for open-type sections, is not required.

1.5.1.4.4 and 1.5.1.4.5 While a $0.6F_y$ bending stress may be used for tension in proportioning flexural members not covered by Sect. 1.5.1.4.1, 1.5.1.4.2 or 1.5.1.4.3, compression stresses in their extreme fibers may be subject to limitation as provided by Formula (4) or (5). Box-type beams and girders are an exception. The torsional properties of a "closed" profile are so much better than those of the "open" I-beam, that lateral stability is not a problem with box beams and girders.

Formula (4) treats the compression flange, plus one-sixth the area of the web adjacent to that flange, as a column, supported in the plane of the web but free to bend, between points of lateral support, about its axis in the plane of the web. It should be noted that for shapes symmetrical about their x-axis of bending, substitution of r_y of the entire section for that of the compression flange plus one-sixth of the web is conservative. Through the introduction of the modifier C_b,* some liberalization in stress is permissible where there is moment gradient over the unbraced length, but use of this liberalization is optional with the designer. Formula (4) is based on the assumption that only the bending stiffness of the compression flange is available to prevent the lateral displacement of that element between bracing points.

Rational expressions for the elastic buckling strength of the beam, which take into account its torsional rigidity about its longitudinal axis as well as the bending stiffness of its compression flange, are too complex for general design office use. They become even more complex if the location of the supported loads (above, at or below the neutral axis) and the shape of the moment diagram are taken into consideration.

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* See *Guide to Design Criteria for Metal Compression Members*, Eq. (4.13), p. 61.
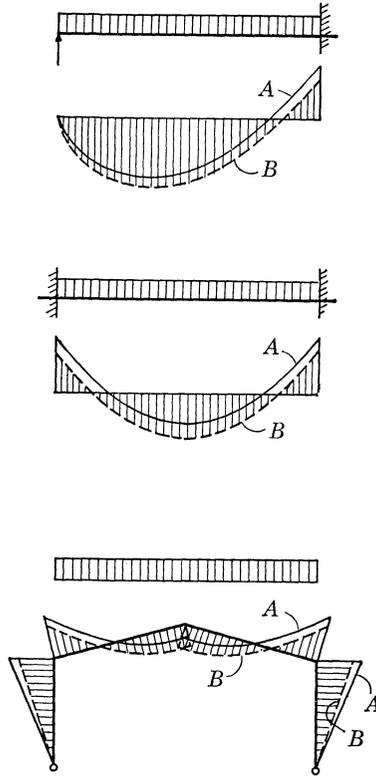
A = Actual moment diagram
B = Modified diagram corresponding to 10 percent moment reduction allowance at interior supports

Fig. C 1.5.1

Formula (5) is a convenient approximation to such expressions, which is conservative for all cases. Its agreement with these expressions is closest in the case of sections having superior torsional properties, identifiable by a relatively low d/A_f ratio. In plate girders, which usually have a much higher d/A_f ratio than rolled I- and WF-shapes, it may err grossly on the conservative side. For such members the larger stress permitted by Formula (4) is the better estimate of buckling strength. While it under-estimates this strength somewhat because it ignores the torsional rigidity of the profile, this rigidity for such sections is relatively small and the margin of over-conservatism, therefore, is likewise small.

Formula (5) assumes the most critical condition of load application and bending moment usually encountered in engineering structures. For less severe conditions it can be made to yield better estimates of bending strength when multiplied by a factor corresponding to the given loading conditions.*

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* Buckling Considerations in the Design of Steel Beams and Plate Girders, p. 429, October, 1954 *Journal of the Boston Society of Civil Engineers.*

It should be noted, however, that Formula (5), like the more precise, complex expressions it replaces, is written for the case of elastic buckling. For shorter unbraced lengths where inelastic buckling would govern, these expressions, like the Euler column formula, become unconservative and must be replaced by values providing a gradual transition between elastic buckling values and the full bending strength. A similar transition is not provided for Formula (5) 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 (4).

1.5.1.4.6 Rolled shapes such as channels, zees and angles, when used as minor flexural members, generally receive lateral support from the slab, deck, wall or siding which they support and hence can usually be designed for the full permissible bending stress. When concentrated loads are introduced on a channel beam by other members framing into it, these usually provide enough torsional and lateral support to the beam so that the reduction in permissible stress, required for laterally unbraced segments of I-shaped beams, can be safely applied in its design. However, it should be remembered that the shear center of a channel profile is eccentric to its center of gravity and even to the plane of its web. Hence, when transverse loading is applied without at the same time providing lateral or torsional support, the effect of eccentricity of loading must be considered in the stress analysis.

The analysis of other types of unsymmetrical profiles having only intermittent points of lateral support, particularly those produced by fabricating components into built-up members, is too complex to be covered by simple rules for common usage. For further discussion of the subject see the *Guide to Design Criteria for Metal Compression Members*, Chapter 4.

1.5.1.4.8 The increase in allowable bending stress for bearing plates can be justified on the basis of shape factor, which for a rectangular profile is 1.50. Additionally, the actual pressure at the edge of the overhanging plate must be less than the average value used in design computation, based upon considerations of elastic behavior. Hence, the computed bending stress overestimates the actual requirement and an increase in working stress to $0.75F_y$ is still conservative.

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 no longer is taken the same as for

rivets. Whereas the latter has been increased slightly in keeping with recent research on riveted joints, the value for pins has been reduced to nine-tenths of the yield point of the part containing the pin hole as a further safeguard against instability of the plate beyond the hole,* which is considerably larger than a rivet hole.

1.5.2 Rivets and Bolts

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 the bolted connections, permissible stresses for bolts are now given in terms applicable to their nominal body area, i.e., the area of the unthreaded shank.

The tension stress permitted for A307 bolts and threaded parts of A7 and A373 steel is equivalent to 20,000 pounds per square inch applied at the root area of the threads, as in earlier specifications. A similar basis is reflected in the provisions for tension on threaded parts made from the other steels listed in Sect. 1.4.1. In recognition of the protection against notch effect in the threading, assured by the required initial tightening, 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.**

1.5.2.1 Shear

In keeping with the recommendations*** of the Research Council on Riveted and Bolted Structural Joints, two working shear values are given for high strength bolts. When slip between the connected parts cannot be tolerated and must be prevented by friction produced by high clamping force, the allowable shear value is the same as that permitted on A141 rivets.† In bearing-type connections (where slip is permissible), when no precaution is taken to exclude the threading from shear planes at the faying surfaces of the connected parts, the allowable shear value is also the same as that permitted on A141 rivets.

The shear value permitted on A307 bolts, as heretofore, also recognizes the possibility of threading in the shear planes.

When care is taken to exclude the threads of high strength bolts from all shear planes in bearing-type joints, a shear stress of 22,000 pounds per square

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* See Pin-Connected Plate Links, 1939 *ASCE Transactions*.
** See Research on Bolted Connections, 1956 *ASCE Transactions*, p. 1265.
*** See *Specifications for Structural Joints Using ASTM A325 Bolts*.
† Increased one-third for A354, Grade BC, bolts tightened to their proof load, which is approximately one-third greater than that of A325 bolts.

inch is permitted for A325 bolts. This value is based on extensive research.*
The allowable stresses for A354, Grade BC, bolts are proportionately higher
in keeping with their higher tensile strength.

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 $2\frac{1}{4}$ 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 equal to $2\frac{1}{4}$ times the tensile working stress recommended for
determining required net area.

1.5.3 Welds

The permissible stress for fillet welds, without regard to the direction of
applied force, was established before the advent of high strength steels in
building construction by applying a factor of safety of 3 to ultimate strength
test results. For convenience the working stress was then rounded off to an
even 500 pounds per linear inch per one-sixteenth inch of weld size. Im-
provements in electrode manufacture and use, particularly the development
of coated electrodes for shielded metal-arc welding, have allowed the 500
pound value to be increased to 600 pounds. This value is well established in
welding done with E60 electrodes and is therefore retained in the Specification.
Since the yield strength of weld metal deposited by E70 series electrodes
ranges about 15 percent higher than that of the earlier types, a corresponding
increase in stress is permitted. Again, for convenience, the working value
has been rounded off, in this case to 700 pounds per linear inch per one-
sixteenth inch of weld size.

The submerged arc process, employing bare wire electrodes and a granular
flux, has been used in the fabrication of structural steel for over two decades
with excellent results. In the absence of a standard electrode specification,
provisions for two strength levels—Grades SA-1 and SA-2—are included in
Sect. 1.17 of the Specification.

By requiring (Sect. 1.17.2) that only E70 series electrodes or Grade SA-2
submerged arc be used in the welding of high-strength low-alloy steels, weld
strength equal to that of the connected parts is assured.

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* See *Specification for Structural Joints Using ASTM A325 Bolts.*
** Effect of Bearing Ratio on Static Strength of Riveted Joints, 1958 *ASCE
Transactions.*

1.5.4 Cast Steel

In keeping with the inclusion of high-strength low-alloy steels, the Specification recognizes high-strength steel castings. Hence, allowable working stresses are now expressed in terms of the specified minimum yield point for castings.

SECTION 1.6 COMBINED STRESSES

1.6.1 Axial Compression and Bending

The earlier "straight line" interaction formula for allowable compression stress acting in conjunction with concurrent bending stress has been revised in two respects, both of which have been the subject of considerable discussion* in recent years.

1. The bending stress at any cross-section subject to lateral displacement must now be amplified by the factor

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

This is in the direction of greater conservatism. It recognizes the fact that such displacement, caused by applied moment, generates a secondary moment equal to the product of the resulting eccentricity and the applied axial load, which is not reflected in the computed stress f_b. Under certain combinations of bending and axial stress and column slenderness, designs meeting the requirement of a straight line interaction formula become somewhat unconservative. However, under other combinations this amplification factor overestimates the influence of secondary moment. To take care of this situation the amplification factor is modified by a reduction factor C_m.

2. Depending upon the slenderness ratio of a given unbraced length, the combined stress computed at one of its laterally braced ends may exceed the combined stress at all points where lateral displacement is created by the applied moments, even when the bending stress at these points has been amplified. To provide for this case the former straight line interaction expression has been liberalized in Formula (7) by substituting $0.6F_y$ for F_a.

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 lateral support. Three categories and the appropriate provisions of Sect. 1.6.1 are listed in Table C1.6.1.1.

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 lateral support, f_b is computed from the larger of the moments at the ends of an unbraced length. When intermediate transverse loading is present, the moment at the end of the unbraced length is used to compute f_b for use in Formula (7).

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* See *Guide to Design Criteria for Metal Compression Members*, Chapter 5.

TABLE C 1.6.1.1

Category	Loading conditions	f_b	C_m	Remarks
A	Computed moments maximum at end; no transverse loading; joint translation not prevented	$\dfrac{M_2}{S}$	0.85	$M_1 < M_2$; $\dfrac{M_1}{M_2}$ positive as shown Check Formulas (6) & (7)
B	Computed moments maximum at end; no transverse loading; joint translation prevented	$\dfrac{M_2}{S}$	$0.4\,\dfrac{M_1}{M_2} + 0.6 \geq 0.4$	Check both Formulas (6) & (7)
C	Transverse loading; joint translation prevented	$\dfrac{M_2}{S}$ Using Formula (7) ——— $\dfrac{M_3}{S}$ Using Formula (6)	$1 + \psi\,\dfrac{f_a}{F'_e}$	Check both Formulas (6) & (7)
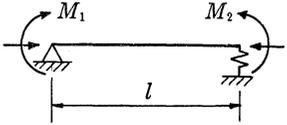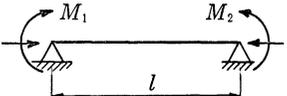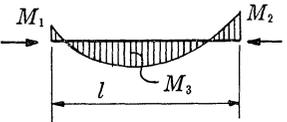
The maximum moment between points of lateral bracing, however, is used to compute the bending stress for use in Formula (6).

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, the effective length of such members, as discussed hereinafter under Sect. 1.8, is never less than the actual unbraced length, and may be greater than this length. The actual length is used in computing moments and determining the value of F'_e. For this case the value of C_m can be conservatively taken as equal to

$$1 - 0.18\,f_a/F'_e$$

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 $0.18f_a/F'_e$. 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, 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 and determining the value of F'_e.

For this category, the greatest eccentricity, and hence the greatest

amplification, occurs when M_1 and M_2 are equal and cause single curvature. It is least when they are 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 like signs which would cause failure at the same concurrent axial load as would the given unequal end moments.

Then $\dfrac{M_e}{M_2}$ can be written,* in terms of $\dfrac{M_1}{M_2}$, as

$$\frac{M_e}{M_2} = C_m = \sqrt{0.3 \left(\frac{M_1}{M_2}\right)^2 + 0.4 \left(\frac{M_1}{M_2}\right) + 0.3}$$

It has been noted** that the simpler formulation

$$C_m = 0.6 + 0.4 \left(\frac{M_1}{M_2}\right) \geq 0.4$$

affords a good approximation to this expression. When M_1/M_2 is less than -0.5 the combined axial and bending stress is usually limited by general yielding rather than by stability, in which case Formula (7) would govern. Therefore, a tentatively selected column section should be tested by both Formulas (6) and (7).

When bending occurs simultaneously about both axes of a column the second (bending) term in Formula (6) may conservatively be treated as the sum of two terms, as

$$\frac{C_m f_b}{\left(1 - \dfrac{f_a}{F'_e}\right) F_b} = \frac{C_{mx} f_{bx}}{\left(1 - \dfrac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \dfrac{f_a}{F'_{ey}}\right) F_{by}}$$

where the subscripts x and y refer to the principal axes of bending of the column profile.

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

$$\psi = \frac{\pi^2 \sigma_o E I}{M_o L^2} - 1$$

σ_o = maximum deflection due to transverse loading

M_o = 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.

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* See *Guide to Design Criteria for Metal Compression Members*, p. 80.

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

*** See *Guide to Design Criteria for Metal Compression Members*, p. 76.

TABLE C 1.6.1.2
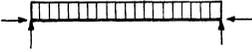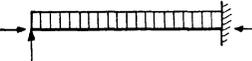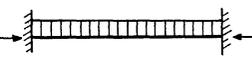
Case	ψ	C_m
	0	1.0
	-0.3	$1 - .3\dfrac{f_a}{F'_e}$
	-0.4	$1 - .4\dfrac{f_a}{F'_e}$
	-0.2	$1 - .2\dfrac{f_a}{F'_e}$
	-0.4	$1 - .4\dfrac{f_a}{F'_e}$
	-0.6	$1 - .6\dfrac{f_a}{F'_e}$
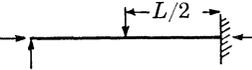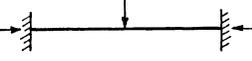
For cases where Formula (6) would govern the design and the value of the axial stress term f_a/F_a is less than 0.15, it is required that the member selected also satisfy the former straight line interaction formula

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0$$

This is to insure that f_b in no case will exceed F_b.

1.6.2 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) the three straight lines shown in Fig. C 1.6.2.

* How Much Combined Stress Can A Rivet Take? *Engineering News-Record,* Dec. 4, 1952.

$$f_t = \frac{F_t}{F_v} \sqrt{F_v{}^2 - f_v{}^2}$$
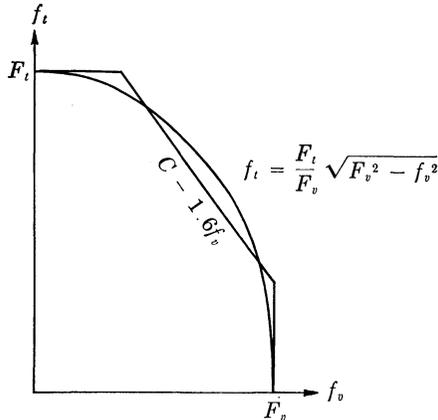
Fig. C1.6.2

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. 1.6.2, 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.

SECTION 1.7 MEMBERS AND CONNECTIONS SUBJECT TO REPEATED VARIATION OF STRESS

Few members in building frames, or the connections for such members, need be designed for "fatigue", which can be defined as a reduction in strength, due to repeated fluctuation in stress involving a large variation in stress. Where fatigue is a problem its severity is enhanced by an increase in the number of load applications and also by an increase in the magnitude of the stress variations. It is aggravated by the presence of sharp notches and other stress raisers in the region of maximum stress.

The magnitude of stress variations, i.e., the range of stress, associated with one repetition of load application, in most members is less than the full allowable (max.) design stress because of the continued presence of dead load (min.) stress.

For grades of steel recommended in the Specification no reduction in working stress is required when less than 10,000 repetitions of maximum design stress are expected to occur in the lifetime of a member, even if the nature of loading is such as to cause an alternating reversal of stress. This is the equivalent of one maximum loading and one complete reversal a day for about 25 years.

The requirements covered by the provisions of this Section are summarized in Table C 1.7.

TABLE C 1.7

Section No.	Application of Design Loads	Calculated Stress Used as Basis for Design	Allowable Stress as Given in Sect. 1.5 and 1.6
1.7.1	Under 10,000 times, with or without stress reversal	Critical static loading (max. static stress produced by any application of specified loads)	Same as for steel and fasteners used
1.7.2	10,000 to 100,000 times, with or without stress reversal	(Max. − $\frac{2}{3}$ min.) or critical static loading	Same as for steel and fasteners used
1.7.3	100,000 to 2,000,000 times, with or without stress reversal	Max. − $\frac{2}{3}$ min.	Allowable stress for A7 steel,* A141 rivet steel, E60XX and submerged arc Grade SA-1 welds
		Critical static loading	Same as for steel and fasteners used
1.7.4	Over 2,000,000 times, with or without stress reversal	Max. − $\frac{3}{4}$ min.	$\frac{2}{3}$ those permitted for A7 steel,* A141 rivet steel, E60XX and submerged arc Grade SA-1 welds
		Critical static loading	Same as for steel and fasteners used

* Regardless of yield point of steel furnished.

When the fluctuations range from tension to compression, or compression to tension, the algebraic difference of maximum and minimum stress becomes the arithmetic sum of these stresses and is characterized for design purposes as the same kind of stress as that which is maximum. When both are tension or both are compression stresses their algebraic difference is less than the maximum. But the area, determined on the basis of the computed difference, can never be less than that required for the critical static loading condition.

No reduction in stress is required, even at 2,000,000 cycles of loading, in proportioning high strength bolts in friction-type joints. The stress in the bolts is not affected by variations in stress in the connected parts. Tests have shown that the high clamping force required to resist slip in a friction-type connection improves somewhat the fatigue strength of these parts.

SECTION 1.8   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. 2.6 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, 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.
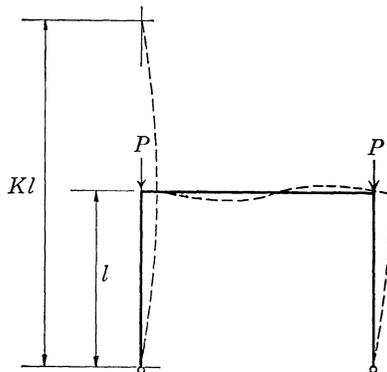
Fig. C1.8.1

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 1.8.2. 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.

If the column base in case (f) of Table C 1.8.2 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

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

TABLE C 1.8.2

	(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		Translation fixed		
		Rotation free		Translation fixed		
		Rotation fixed		Translation free		
		Rotation free		Translation free		

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. 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 1.8.2 to very complex analytical procedures. Once a trial selection of framing members has been made, the use of the following alignment chart (Fig. C 1.8.3) affords a fairly rapid method for determining suitable K-values.

Where the design of a building frame is based primarily upon the effect of large side loading or upon a "drift" limitation, the effective column length may generally be taken as the actual unbraced length. 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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* See 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.

$$G_A \qquad K \qquad G_B$$

The subscripts A and B refer to the joints at the two ends of the column section being considered. G is defined as

$$G = \frac{\sum \dfrac{I_c}{L_c}}{\sum \dfrac{I_g}{L_g}}$$

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. C 1.8.3

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

Elements of members having width-thickness ratios no greater than those specified, can be stressed approximately to yield point without failure by

local buckling. Under favorable conditions of support, premature buckling will not occur even under more slender ratios, but the analyses needed to determine the proper limiting values are too complex for common use.

As the allowable stress on compression elements is increased in proportion to the increase in the specified minimum yield point of the material, width-thickness ratios must be further restricted in order to prevent local buckling. For various grades of steel the critical ratios are inversely proportional to $\sqrt{F_y}$.

SECTION 1.10 PLATE GIRDERS AND ROLLED BEAMS

1.10.1 Proportioning

As in the earlier AISC Specification, it is recommended* 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. However, holes not filled by rivets are no longer treated separately. Test observations have clearly shown that the stress distribution around such holes is the same whether they are filled with a fastener or not.

1.10.2 Web

An upper limit is placed upon the web depth-thickness ratio which, for steel having a yield point of 33,000 pounds per square inch, is 345. For steels having a higher yield point, this limit is proportionately less. Analytical studies, corroborated by test results, have indicated that up to this limit the web is capable of providing vertical support for the compression flange. If more slender girder webs were permitted there would be a possibility that the compression flange might buckle before the intended ultimate load had been reached.

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 total cover plate stress 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

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* See Effective Moment of Inertia of a Riveted Plate Girder, *1940 ASCE Transactions.*

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 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

More liberal spacing of intermediate transverse stiffeners is now permitted than heretofore. Earlier provisions governing the design of plate girders 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 (8). Use of tension field action is not counted upon when

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

or where

$$a/h > 3.0$$

When the computed average shear stress in the web is less than that permitted by Formula (9), intermediate stiffeners are not required. Such

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* See *Welding Research Council Bulletin* No. 63, page 48.

** See Strength of Plate Girders in Shear, *ASCE Journal of the Structural Division*, Oct. 1961.

*** See Web Buckling Tests on Welded Plate Girders, *Welding Research Council Bulletin* No. 64.

girders do not depend upon tension field action. However, the depth of these girders is limited to not more than 260 times the web thickness.

When intermediate stiffeners are 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 of the Appendix for each of the 5 specified yield points 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 (9). For such cases tension field action is not counted upon.

At the ends of the girder, the spacing between adjacent stiffeners is limited to $11,000t/\sqrt{f_v}$, as heretofore. So spaced, the web is capable of resisting the full shear without tension field action, thus providing 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.

As in earlier Specifications, all stiffeners are required to have a moment of inertia at least equal to $\left(\dfrac{h}{50}\right)^4$. 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 (10). 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

$$f_{vs} = h \sqrt{\left(\frac{F_y}{3,400}\right)^3}$$

affords a conservative estimate of required shear transfer under any condition of stress permitted by Formula (8). 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.

In order to facilitate handling during fabrication and erection, an upper limit of 260 times the web thickness is placed upon the lesser of the panel dimensions a or h. For the same reason, where intermediate stiffeners are required so that the computed average shear stress in the web will not exceed the value permitted by Formulas (8) or (9), the panel aspect ratio a/h is arbitrarily limited to

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

with a maximum spacing of 3 times the girder depth.

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 so 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 (11), sufficient bending capacity is provided in the flange to compensate for any loss of bending strength in the web due to its lateral displacement.

1.10.7 Combined Shear and Tension Stress

It can be shown that plate girder webs 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 (12).

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 (13) or (14) 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 (15) or (16). If the flange is prevented from rotation about its longitudinal axis by its contact with a rigid slab, Formula (15) will govern; otherwise, the more conservative Formula (16) 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.

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. Heretofore, it has been a prerequisite that the beam be fully encased in concrete poured integrally with the slab. The Specification now has been expanded to include provisions covering the use of mechanical shear connectors to obtain composite action when the beams are not encased.

1.11.2   Design Assumptions

Beams encased in concrete are deemed to be interconnected by means of the natural bond of the concrete to the steel beam. Unless temporary shores are used, however, the beam 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*,** however, when shear

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

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

connectors are used to obtain composite action, this action may be used within certain limits in proportioning the beam for the moments created by both live and dead loads. This liberalization is based upon an ultimate strength concept. Safe working limits are established by applying a factor of safety to the ultimate bending strength of the composite beam, rather than to the load at which, theoretically, yielding would commence in the steel beam.

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-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.*

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 the past.

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 action is analogous to that which takes place in connections having a large number of fasteners in the line of stress. The important consideration is that the total number of connectors, either side of the point of maximum moment, be sufficient to develop full composite action at that point. The provisions of the Specification are based upon this concept of composite action.

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

SECTION 1.13 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.

Movement under varying applied loads which would be intolerable to persons standing on a structure may be in no way objectionable in the case of a shed-type building whose only function is to provide shelter. Where human comfort is the criterion for limiting motion, as in the case of perceptible

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* Progress Report of the Joint ASCE -ACI Committee on Composite Construction *ASCE Journal of the Structural Division*, December, 1960.   Equation (3).

** (Report to be published.)

vibrations, the limit of tolerable amplitude is dependent upon the frequency of vibrations.

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 $F_y/800,000$ times the span, and where subject to shock or vibration not less than $F_y/650,000$ 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 $F_y/1,000,000$ times the span, except in the case of roofs with a slope not less than 3 in 12.

Minimum depth ratios for restrained and continuous spans should, if practicable, be such that the deflection at critical points will not be greater than that of a simple beam designed for the same loading.

In the case of flat roofs, the Specification limits the depth-span ratio of supporting beams and girders to $F_y/1,000,000$, regardless of any condition of continuity. This is done in order to minimize the effect of "ponding", wherein the deflection of supporting beams results in the retention of rain water which, in turn, causes additional deflection. In the case of continuous framing, unequal dead load deflections in adjacent spans can result in a greater accumulation in one span which, in turn, tends to unload the adjacent spans, thus reducing the restraint at the ends of the more heavily loaded span. To prevent this occurrence the same depth-span limitation is stipulated for continuous framing as for simple spans.

SECTION 1.14   GROSS AND NET SECTIONS

1.14.3   Net Section

Tests have shown that the ultimate strength of a tension member containing holes usually will not exceed 85 percent of a similar member without a hole, even when the net section, computed in accordance with the prescribed rules, is more than 85 percent of the gross section. Hence, a limitation of this amount has been added. Otherwise the provisions relating to gross and net area are the same as formerly.

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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* See Pin-Connected Plate Links, *1939 ASCE Transactions.*

1.14.7   Effective Areas of Weld Metal

The effective throat thickness of partial penetration single-V, single-bevel, single-J and single-U groove welds having no root opening has been added.   The first two of these have been discounted by $\frac{1}{4}$ inch because of the difficulty of ensuring full penetration at the bottom of the groove, as indicated in Fig. C 1.14.1.

Partial penetration groove welds of this type are frequently used in column splices and in connecting elements of built-up members, pedestals, grillages and similar assemblies where the stress to be transferred is substantially less than that requiring a complete penetration butt weld.   As in the case of fillet welds, the minimum permissible size of welds is a function of the thickness of the material being welded.

Closed Single-Bevel   Closed Single-J

Closed Single-V   Closed Single-U

Incomplete Penetration Single-Bevel   Incomplete Penetration Single-U
Groove Welds in Column Splices   Groove Welds in Built-Up Column
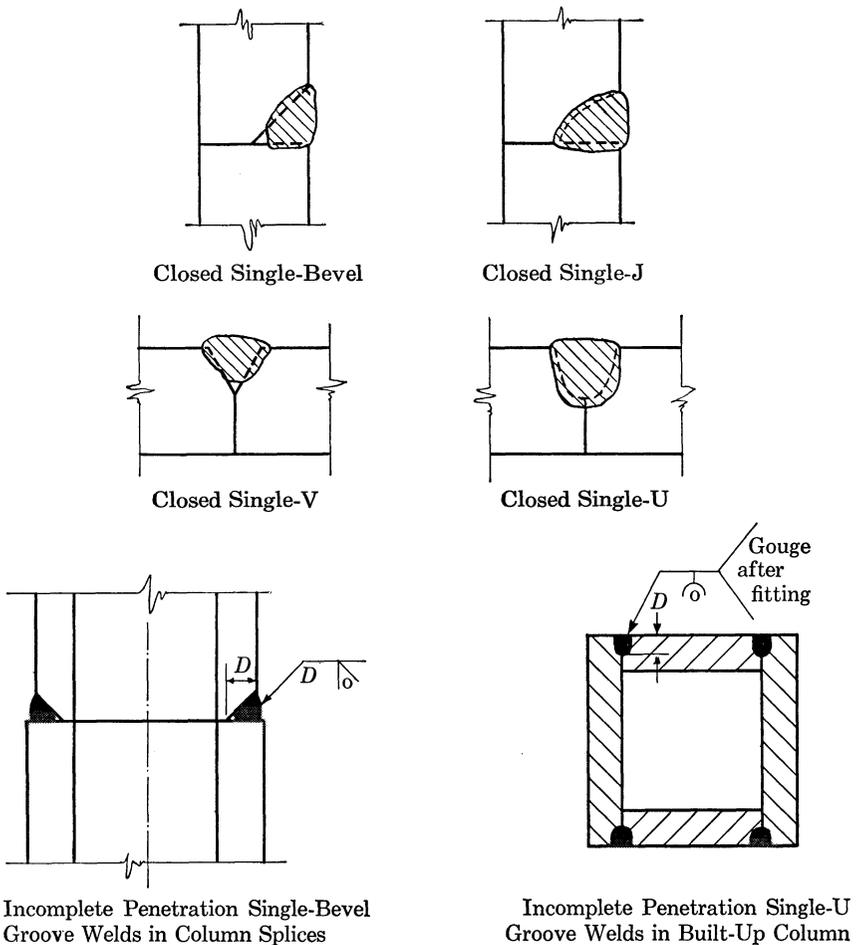
Incomplete Penetration Groove Welds

Fig. C 1.14.1

SECTION 1.15   CONNECTIONS

1.15.1   Minimum Connections

The requirement that connections carrying calculated stress be designed for not less than 10,000 pounds, which was based on the assumption that only rivets (for practical reasons requiring two as a minimum) would be used in the connection, has been reduced somewhat in keeping with the current use of fillet welds and A307 bolts.   The new requirement is still adequate to take care of temporary stress due to handling and erection.

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.   Much more attention has been paid to the matter in welded construction, resulting at times in somewhat awkward details.   Tests have shown* this practice to be unwarranted in statically loaded structures and the Specification has been revised to reflect these findings.

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, as in earlier editions of the AISC Specification, is not recommended in new work.   High strength bolts used in bearing-type connections also should not be required to share the 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.

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* *The Welding Journal*, January, 1942, pp. 44–5.

SECTION 1.16   RIVETS AND BOLTS

1.16.1   High Strength Bolts

The addition of this Section citing the specification of the Research Council on Riveted and Bolted Structural Joints merely confirms action taken by the American Institute of Steel Construction as long ago as 1950 in endorsing that standard.

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.

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 Standard Code for Arc and Gas Welding in Building Construction* and the *AWS Standard Specifications for Welded Highway and Railroad Bridges*.   Other welding processes, such as inert gas welding, have been developed and are being used, but have not been standardized sufficiently to be included specifically in the Specification.   They may be used provided they are qualified to the satisfaction of the designer and the building code authority.

SECTION 1.18   BUILT-UP MEMBERS

Requirements dealing with the detailing of built-up members, which cannot be stated in terms of calculated stress, have been assembled in a single section of the Specification.   Many of them 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

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* Long Structural Joints of A7 Steel, *Fritz Engineering Laboratory Report No. 271.18.*

ensure a close fit-up over the entire faying surface of components designed to be in contact with one another.

Provisions based on this latter consideration, like those giving maximum spacing of stitch fasteners for separated components of built-up tension members, are of little structural significance. Hence, some latitude is warranted in relating them to the given dimensions of a particular member.

The provisions governing the proportioning of perforated cover plates are based upon extensive experimental research.*

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.

Recently 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.

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* Perforated Cover Plates for Steel Columns, *Research Paper RP1861*, National Bureau of Standards, and

Study of Columns with Perforated Cover Plates, *AREA Bulletin No. 531*, Sept.-Oct., 1956.

SECTION 1.23 FABRICATION

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 and the Welding Research Council publication *Weldability of Steels.*

SECTION 1.24 SHOP PAINTING

The shop painting of structural steel not to be encased in concrete is no longer 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.*

As in the past, 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 prescription would not suffice.**

PART 2

SECTION 2.1 SCOPE

Pending the completion of current research, the use of plastic design is limited largely to low building frames wherein axial stress in the columns is relatively small. However, beams in the floors of multi-story buildings, in which sidesway is prevented and resistance to lateral forces is provided by means other than the bending stiffness of these beams, may be designed in accordance with the provisions of Part 2, provided the columns in such structures are designed in accordance with the provisions of Part 1.

The adoption of a load factor of 1.70 for beams recognizes a fundamental premise of plastic design, namely that a plastically designed continuous beam should provide the same margin of strength as that inherent in a simply supported beam designed under an allowable working stress type of specification to support the same load.

The plastic 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

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* See 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.

its profile; a non-compact member (meeting the provisions of Sect. 1.9, but not those of Sect. 2.6), usually has but 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 load factor F to be used in plastic design is defined* as

$$F = \frac{\partial_y}{\partial_w} \ (f)$$

where ∂_y and ∂_w are, respectively, the yield stress and allowable working stress. A load equal to $\frac{33,000}{20,000} \times 1.0 = 1.65$ has proven adequate for the design of non-compact sections for many years.

The superior bending strength of compact sections is now 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 having compact profiles is given by the equation $F = \frac{F_y}{0.66F_y} \ (f)$. 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, as well as for the design of non-compact rolled beams.

Extension of plastic design methods to simple beams is merely for the convenience of the designer. When the analysis of other members in a framework is made on an ultimate strength basis using factored loads, it is not necessary to convert these loads back to actual requirements in order to proportion simple beams according to an allowable bending stress provision. Usually the same selection of member size results from solving for the required M_p at ultimate load.

SECTION 2.2 STRUCTURAL STEEL

The plastic design rules have been revised to include the use of A36 structural steel. Since the elastic-plastic behavior of steels having yield points in the range of but higher than 36,000 pounds per square inch (characterized by the idealized stress-strain curve shown in Fig. C 2.2) is essentially
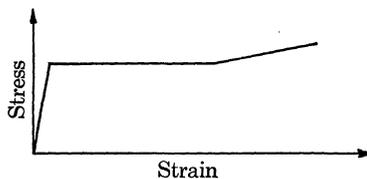
Fig. C 2.2

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* See *ASCE Manual of Engineering Practice No. 41*, Commentary on Plastic Design in Steel, page 18.

the same as that of A7 and A36 steel, applications of plastic design would not be improper. However, before this is recommended, the stability problems (local buckling, lateral buckling and column buckling) must be restudied at the higher stress level corresponding to the higher yield point. Hence, for the present, the provisions are limited to steels having a specified yield point of 36,000 pounds per square inch or less.

SECTION 2.3 COLUMNS

The limitations which have been placed on slenderness and on the intensity of axial loading reflect the fact that the effective plastic bending strength of a member and its ability to rotate plastically decreases as slenderness and concurrent axial stress increase, and reaches a point where it is either nonexistent or so small as to render a shape too uneconomical for the requirements of a given problem. However, these limits are ample enough to include the full range of practical problems encountered within the scope presently recommended for plastic design.

Three readily distinguishable conditions of end moment are recognized: Cases I, II and III. Each is covered by an interaction formula giving the effective moment M_o furnished by a particular shape in the presence of a given axial load P, in terms of the full plastic moment M_p of its profile and the axial load that it would support in the absence of bending moment.

Within the limits given, the interaction expression for Case I (Formula 21) is independent of slenderness ratio. The effective moment M_o is reduced from the full plastic moment capacity M_p furnished by the profile only by the amount of the profile area required to support the given axial load P at yield stress.* For Case II and Case III columns, the axial load which the profile could support is dependent upon slenderness ratio. The corresponding interaction expressions (Formulas 22 and 23) become rather complex and can best be expressed in terms of coefficients, the numerical values for which, corresponding to values of l/r, are given in Tables 4-33, 4-36 and 5-33, 5-36 in the Appendix to the Specification.

By virtue of the provisions of Formula (20), use of Formulas (21), (22) and (23) is limited to frames in which sidesway is not a problem. Substantially the same interaction expressions as those given by Formulas (21), (22) and (23) could be written using Formulas (6) and (7), expressed in terms of ultimate load rather than working stress.** These would have the advantage of affording solutions for the cases where one of the computed end moments was neither zero nor equal numerically to the other end moment. However, the amount of design time involved in testing the suitability of a trial profile for the given load, moments and unbraced length by these expressions, as compared with use of the tables noted above, would seldom be justified by the slight economy in the use of steel that might be achieved.

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* See *ASCE Manual of Engineering Practice No. 41*, Commentary on Plastic Design in Steel, pp. 75–81.

** *Ibid.*, Eqs. (7.13) and (7.15).

SECTION 2.4   SHEAR

The capacity of an unreinforced web to resist shear has been defined as an average shear stress equal to $F_y/\sqrt{3}$.* The effective depth of a beam has been taken as 0.95 times its actual depth to allow for the presence of plastic strain in the flanges, due to concurrent bending. Thus

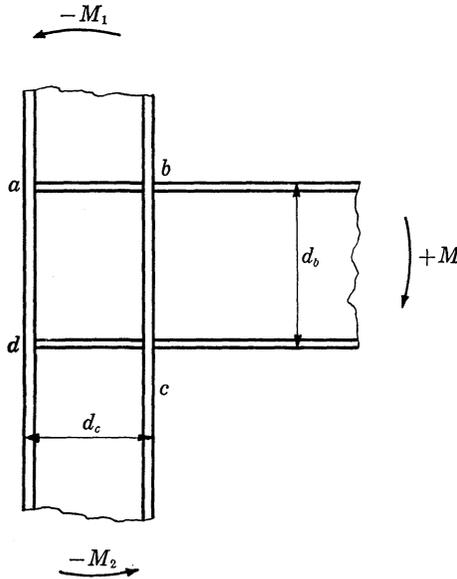$$V_u = \frac{0.95\,F_y}{\sqrt{3}}\,wd = 0.55F_y\,wd$$

Fig. C 2.4

Assuming the moment $+M$, in Fig. C 2.4, expressed in pound-feet, to be resisted by a couple of forces at the centroid of the beam flange, the shear produced in beam-to-column connection web *abcd* can be computed as

$$V = \frac{+12M}{0.95d_b}$$

when

$$V = V_u = 0.55F_y w d_c$$

$$\text{Req'd } w = \frac{12M}{0.95d_b \times 0.55F_y d_c}$$

$$= \frac{23{,}000M}{AF_y}$$

where A is the planar area *acbd* and F_y is expressed in pounds per square inch.

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* See *ASCE Manual of Engineering Practice No. 41*, Commentary on Plastic Design in Steel, p. 39.

SECTION 2.5   WEB CRIPPLING

Usually stiffeners are needed, as at *ab* and *dc* in Fig. C 2.4, 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.

The formulas given for least web thickness w^* and flange thickness t_f,** below which stiffeners are required, have been developed and corroborated by tests*** to ensure that yielding will not occur at these points before the full M_p value of the connected beam has been reached.

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 2.5 may prove advantageous.

Stiffener Plates
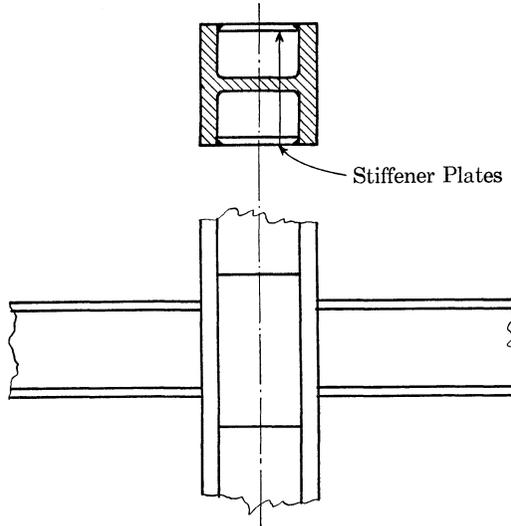
Fig. C 2.5

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* *ASCE Manual of Engineering Practice No. 41*, Commentary on Plastic Design in Steel, Eq. (8.27).

** *Ibid.*, (Eq. 8.26).

*** See *Welded Interior Beam-to-Column Connections*, American Institute of Steel Construction.

SECTION 2.6 MINIMUM THICKNESS (WIDTH - THICKNESS RATIOS)

The width-thickness ratios of compression elements of a profile subject to rotation due to plastic hinge action are more restrictive than similar ratios given in Sect. 1.9 of the Specification. The latter are required merely to *reach* yield stress without buckling. To ensure adequate hinge rotation capacity the proportions required for compression elements in regions of maximum moment in plastically designed framing are such that these elements can compress plastically to strain-hardening.*

The web depth-thickness ratio of beams and girders required to develop a plastic hinge at ultimate load is limited to 70. In the presence of concurrent axial loading this ratio is to be reduced in accordance with Formula (25), but not below 43. None of the rolled shapes have a web depth-thickness ratio in excess of 70.

SECTION 2.7 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 ultimate load, the permissible stresses to be used in proportioning parts of the connection will be in the ratio $F_y/0.6F_y$, or 1.67 times those given in Sect. 1.5 and 1.6, except that high strength bolts required to resist tension may be proportioned on the basis of their proof load.

The same procedure is valid in proportioning connections located in the region of a plastic hinge, with two added restrictions. 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 shall meet the requirements given in Part 2, and sheared edges and punched holes shall 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.

Haunched connections designed in accordance with the following procedures will meet all of the requirements of Sect. 2.7.

Tapered Haunches (See Fig. C 2.7.1.)

1. Make web thickness not less than that of adjoining members.
2. Proportion flange area so that the moment at any point due to ultimate loading, divided by the corresponding plastic modulus of the section taken normal to the connected member, would not exceed the yield point.

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* See *ASCE Manual No. 41*, Commentary on Plastic Design in Steel, Section 6.2.

** See *Plastic Design in Steel*, American Institute of Steel Construction, pp. 36 and 37.
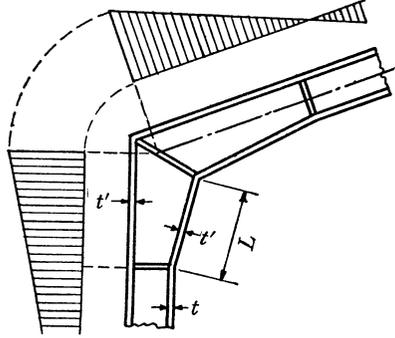
Fig. C 2.7.1

3. a. If the taper is such that the stress, computed as in (2) above, is approximately yield point at both ends, limit the unbraced length l to not more than 6 times the flange width b or, alternatively, multiply the flange thickness t', used in computing the plastic moduli, by the factor

$$1 + 0.1 \left(\frac{l}{b} - 6 \right)$$

b. If the proportions of the haunch are such that the stress at one end, computed as in (2) above, is approximately the yield point and the computed stress f at the other end, using the section modulus instead of the plastic modulus, is less than yield point, limit the unbraced length to

$$l = (17.5 - 0.40f)b$$

but not less than $6b$.

c. If the bending stress, computed on the basis of the section modulus, is less than the yield point at all transverse sections, check to be sure that the maximum computed value does not exceed

$$\frac{12,000 \times 1.67}{\dfrac{ld}{A_f}}$$

where l is the distance between bracing points and d is the greatest depth of section between these points.

4. Provide stiffeners at both ends of tapered haunches, making the total cross-sectional area of these stiffeners not less than three-fourths that of the flange area.

Curved Haunches (See Fig. 2.7.2.)

1. Provide web thickness not less than that of adjoining members.

2. With the aid of the graph in Fig. C 2.7.3, determine the required thickness t' for a haunch flange having a width b equal to that of the connected member in which the hinge would form.

$$t' = (1 + m)t$$
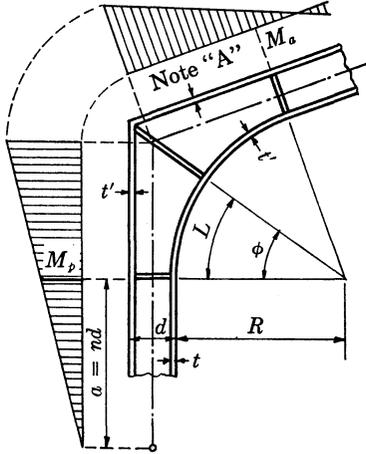
Note "A"—Area of this flange need not exceed that of connected flange if bending stress, M_a/S, is less than f_y.

Fig. C 2.7.2

3. If the unbraced length l, equal to $R\phi$ where ϕ is expressed in radians, is greater than $6b$, increase the haunch flange thickness, computed as in (2), by an amount equal to

$$0.1 \left(\frac{l}{b} - 6\right) t'$$

Alternatively, the haunch area bt' may be furnished by a plate having a width not less than $l/6$ and thickness not less than t'.

4. Limit width-thickness ratio b/t' of curved inner flange to $2R/b$ or 17, whichever is the smaller value.

5. Provide stiffeners at, and midway between, points of tangency, making the total cross-sectional area of stiffeners at the mid-brace point not less than three-fourths that of the curved flange area.
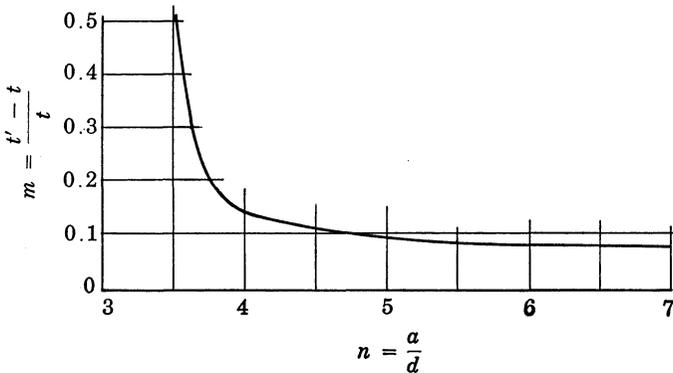

Fig. C 2.7.3

SECTION 2.8   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 1.67, they must also strain inelastically to provide the necessary hinge rotation.   This is not true at the last hinge to form, since the ultimate load is assumed to have been reached when this hinge starts to rotate.   When bending takes place about the strong axis, any I- or WF-shaped 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.

Unbraced lengths no greater than those determined by Formula (26) ensure ample hinge rotation capacity when the width-thickness ratios of compression elements are within the limits provided in Sect. 2.6 so as to prevent local buckling.   Values of l_{cr} computed by the formula are usually somewhat conservative because no credit is given to the restraining influence of segments of the frame adjacent to the length under consideration.   More accurate procedures are available* for computing the critical unbraced length, but they are unnecessarily involved for ordinary usage.

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* See *ASCE Manual No. 41*, Commentary on Plastic Design in Steel, Sect. 6.3.