Specification for Structural Steel Buildings

Allowable Stress Design and Plastic Design June 1, 1989

with Commentary

AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.
One East Wacker Drive, Suite 3100
Chicago, IL 60601-2001

PREFACE

The AISC *Specification for Structural Steel Buildings*—Allowable Stress Design (ASD) and Plastic Design has evolved through numerous versions from the 1st Edition, published June 1, 1923. Each succeeding edition has been based upon past successful usage, advances in the state of knowledge and changes in design practice. The data included has been developed to provide a uniform practice in the design of steel-framed buildings. The intention of the Specification is to provide design criteria for routine use and not to cover infrequently encountered problems which occur in the full range of structural design.

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

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

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

A Commentary has been included to provide background for these and other provisions.

This edition of the Specification has been developed primarily upon the basis of the criteria in the Specification dated November 1, 1978. That Specification, as well as earlier editions, was arranged essentially on the basis of type of stress with special or supplementary requirements for different kinds of members and details contained in succeeding sections. The provisions of the 1978 Specification have been reorganized using decision table logic techniques to provide an allowable stress design specification that is more logically arranged on the basis of type of member.

This arrangement is more convenient to the user because general design requirements are presented first, followed by chapters containing the information required to design members of each type. This organization is consistent with that used in the *Load and Resistance Factor Design Specification for Structural Steel Buildings*.

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

- Reorganization of provisions to be consistent with LRFD format.
- New provisions for built-up compression members.
- New provisions for the design of webs under concentrated forces.
- Updated provisions for slender web girders.
- Updated provisions for design for fatigue.
- Recommendations for the use of heavy rolled shapes and welded members made up of thick plates.

The reader is cautioned that independent professional judgment must be exercised when data or recommendations set forth in this Specification are applied. The publication of the material contained herein is not intended as a representation or warranty on the part of the American Institute of Steel Construction, Inc.—or any other person named herein—that this information is suitable for general or particular use, or freedom from infringement of any patent or patents. Anyone making use of this information assumes all liability arising from such use. The design of structures is within the scope of expertise of a competent licensed structural engineer, architect, or other licensed professional for the application of principles to a particular structure.

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Lyle L. Wilson
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Charles Peshek, Secretary

June 1989

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CHAPTER A

GENERAL PROVISIONS

A1. SCOPE

The *Specification for Structural Steel Buildings—Allowable Stress Design and Plastic Design* is intended as an alternate to the currently approved *Load and Resistance Factor Design Specification for Structural Steel Buildings* of the American Institute of Steel Construction., Inc.

A2. LIMITS OF APPLICABILITY

1. Structural Steel Defined

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

2. Types of Construction

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

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

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

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

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

Type 1 construction is unconditionally permitted under this Specification. Two different methods of design are recognized. Within the limitations laid down in Sect. N1, 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 Chapters A through M, to resist the stresses produced by the specified design loads, assuming moment distribution in accordance with the elastic theory.

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

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

1. Connections and connected members have adequate capacity to resist wind moments.
2. Girders are adequate to carry full gravity load as "simple beams."
3. Connections have adequate inelastic rotation capacity to avoid overstress of the fasteners or welds under combined gravity and wind loading.

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

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

A3. MATERIAL

1. Structural Steel

a. ASTM designations

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

Structural Steel, ASTM A36
Pipe, Steel, Black and Hot-dipped, Zinc-coated Welded and Seamless Steel Pipe, ASTM A53, Gr. B
High-strength Low-alloy Structural Steel, ASTM A242
High-strength Low-alloy Structural Manganese Vanadium Steel, ASTM A441
Cold-formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes, ASTM A500
Hot-formed Welded and Seamless Carbon Steel Structural Tubing, ASTM A501
High-yield Strength, Quenched and Tempered Alloy-Steel Plate, Suitable for Welding, ASTM A514
Structural Steel with 42 ksi Minimum Yield Point, ASTM A529
Steel, Sheet and Strip, Carbon, Hot-rolled, Structural Quality, ASTM A570 Gr. 40, 45 and 50

High-strength, Low-alloy Columbium-Vanadium Steels of Structural Quality, ASTM A572

High-strength Low-alloy Structural Steel with 50 ksi Minimum Yield Point to 4-in. Thick, ASTM A588

Steel, Sheet and Strip, High-strength, Low-alloy, Hot-rolled and Cold-rolled, with Improved Atmospheric Corrosion Resistance, ASTM A606

Steel, Sheet and Strip, High-strength, Low-alloy, Columbium or Vanadium, or both, Hot-rolled and Cold-rolled, ASTM A607

Hot-formed Welded and Seamless High-strength Low-alloy Structural Tubing, ASTM A618

Structural Steel for Bridges, ASTM A709

Quenched and Tempered Low-alloy Structural Steel Plate with 70 ksi Minimum Yield Strength to 4 in. thick, ASTM A852

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

b. Unidentified steel

Unidentified steel, if free from surface imperfections, is permitted 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.

c. Heavy shapes

For ASTM A6 Groups 4 and 5 rolled shapes to be used as members subject to primary tensile stresses due to tension or flexure, toughness need not be specified if splices are made by bolting. If such members are spliced using full penetration welds, the steel shall be specified in the contract documents to be supplied with Charpy V-Notch testing in accordance with ASTM A6, Supplementary Requirement S5. The impact test shall meet a minimum average value of 20 ft-lbs. absorbed energy at +70°F and shall be conducted in accordance with ASTM A673 with the following exceptions:

a. The center longitudinal axis of the specimens shall be located as near as practical to midway between the inner flange surface and the center of the flange thickness at the intersection with the web mid-thickness.

b. Tests shall be conducted by the producer on material selected from a location representing the top of each ingot or part of an ingot used to produce the product represented by these tests.

For plates exceeding 2-in. thick used for built-up members with bolted splices and subject to primary tensile stresses due to tension or flexure, material toughness need not be specified. If such members are spliced using full penetration welds, the steel shall be specified in the contract documents to be supplied with Charpy V-Notch testing in accordance with ASTM A6, Supplementary Requirement S5. The impact test shall be conducted by the producer in accordance with ASTM A673, Frequency P, and shall meet a minimum average value of 20 ft-lbs. absorbed energy at +70°F.

The above supplementary toughness requirements shall also be considered for welded full-penetration joints other than splices in heavy rolled and built-up members subject to primary tensile stresses.

Additional requirements for joints in heavy rolled and built-up members are given in Sects. J1.7, J1.8, J2.6, J2.7 and M2.2.

2. Steel Castings and Forgings

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

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

Steel forgings shall conform to the following standard specification:

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

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

Allowable stresses shall be the same as those provided for other steels, where applicable.

3. Rivets

Steel rivets shall conform to the following standard specification:

- Steel Structural Rivets, ASTM A502

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

4. Bolts, Washers and Nuts

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

- Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength, ASTM A307
- High-strength Bolts for Structural Steel Joints, ASTM A325
- Quenched and Tempered Steel Bolts and Studs, ASTM A449
- Heat-treated Steel Structural Bolts, 150 ksi Min. Tensile Strength, ASTM A490
- Carbon and Alloy Steel Nuts, ASTM A563
- Hardened Steel Washers, ASTM F436

A449 bolts are permitted only in connections requiring bolt diameters greater than 1½ in. and shall not be used in slip-critical connections.

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

5. Anchor Bolts and Threaded Rods

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

Structural Steel, ASTM A36
Carbon and Alloy Steel Nuts for Bolts for High-pressure and High-temperature Service, ASTM A194, Gr.7
Quenched and Tempered Alloy Steel Bolts, Studs and other Externally Threaded Fasteners, ASTM A354
Quenched and Tempered Steel Bolts and Studs, ASTM A449
High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality, ASTM A572
High-strength Low-alloy Structural Steel with 50,000 psi Minimum Yield Point to 4 in. Thick, ASTM A588
High-strength Non-headed Steel Bolts and Studs, ASTM A687

Threads on bolts and rods shall conform to Unified Standard Series of latest edition of ANSI B18.1 and shall have Class 2A tolerances.

Steel bolts conforming to other provisions of Sect. A3 are permitted as anchor bolts. A449 material is acceptable for high-strength anchor bolts and threaded rods of any diameter.

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

6. Filler Metal and Flux for Welding

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

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

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

*Approval of these welding electrode specifications is given without regard to weld metal notch toughness requirements, which are generally not critical for building construction. See Commentary, Sect. A3.

7. Stud Shear Connectors

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

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

A4. LOADS AND FORCES

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

1. Dead Load and Live 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.

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

2. 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 not less than:

For supports of elevators	100%
For cab-operated traveling crane support girders and their connections	25%
For pendant-operated traveling crane support girders and their connections	10%
For supports of light machinery, shaft or motor driven	20%
For supports of reciprocating machinery or power driven units	50%
For hangers supporting floors and balconies	33%

3. Crane Runway Horizontal Forces

The lateral force on crane runways to provide for the effect of moving crane trolleys shall be not less than 20% of the sum of weights of the lifted load and of the crane trolley, but exclusive of other parts of the crane. The force shall

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

be assumed to be applied at the top of the rails, acting in either direction normal to the runway rails, and shall be distributed with due regard for lateral stiffness of the structure supporting the rails.

The longitudinal tractive force shall be not less than 10% of the maximum wheel loads of the crane applied at the top of the rail, unless otherwise specified.

The crane runway shall also be designed for crane stop forces.

4. Wind

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

5. Other Forces

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

A5. DESIGN BASIS

1. Allowable Stresses

Except as provided in Chapter N, all structural members, connections and connectors shall be proportioned so the stresses due to the working loads do not exceed the allowable stresses specified in Chapters D through K. The allowable stresses specified in these chapters do not apply to peak stresses in regions of connections (see also Sect. B9), provided requirements of Chapter K are satisfied.

For provisions pertaining to plastic design, refer to Chapter N.

2. Wind and Seismic Stresses

Allowable stresses may be increased ⅓ above the values otherwise provided when produced by wind or seismic loading, acting alone or in combination with the design dead and live loads, provided the required section computed on this basis is not less than that required for the design dead and live load and impact (if any) computed without the ⅓ stress increase, and further provided that stresses are not otherwise* required to be calculated on the basis of reduction factors applied to design loads in combinations. The above stress increase does not apply to allowable stress ranges provided in Appendix K4.

3. Structural Analysis

The stresses in members, connections and connectors shall be determined by structural analysis for the loads defined in Sect. A4. Selection of the method of analysis is the prerogative of the responsible engineer.

*For example, see ANSI A58.1, Sect. 2.3.3.

4. Design for Serviceability and Other Considerations

The overall structure and the individual members, connections and connectors shall be checked for serviceability in accordance with Chapter L.

A6. REFERENCED CODES AND STANDARDS

Where codes and standards are referenced in this Specification, the editions of the following listed adoption dates are intended:

American National Standards Institute
- ANSI B18.1-72
- ANSI A58.1-82

American Society of Testing and Materials

ASTM A6-87d	ASTM A27-87	ASTM A36-87
ASTM A53-88	ASTM A148-84	ASTM A242-87
ASTM A307-86a	ASTM A325-86	ASTM A354-86
ASTM A441-85	ASTM A449-87	ASTM A490-85
ASTM A500-84	ASTM A501-84	ASTM A514-87a
ASTM A529-85	ASTM A563-84	ASTM A570-85
ASTM A572-85	ASTM A588-87	ASTM A606-85
ASTM A607-85	ASTM A618-84	ASTM A668-85a
ASTM A687-84	ASTM C33-86	ASTM C330-87
ASTM F436-86	ASTM A502-83A	ASTM A709-87b
ASTM A852-85		

American Welding Society

AWS D1.1-88	AWS A5.1-81	AWS A5.5-81
AWS A5.17-80	AWS A5.18-79	AWS A5.20-79
AWS A5.23-80	AWS A5.28-79	AWS A5.29-80

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

A7. DESIGN DOCUMENTS

1. Plans

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

Design documents shall indicate the type or types of construction as defined in Sect. A2.2 and shall include the loads and design requirements necessary for preparation of shop drawings including shears, moments and axial forces to be resisted by all members and their connections.

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

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

2. Standard Symbols and Nomenclature

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

3. Notation for Welding

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

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

CHAPTER B

DESIGN REQUIREMENTS

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

B1. GROSS AREA

The gross area 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.

For angles, the gross width shall be the sum of the widths of the legs less the thickness.

B2. NET AREA

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

The width of a bolt or rivet hole shall be taken as 1/16 in. greater than the nominal dimension of the hole.

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

$$s^2/4g$$

where

s = longitudinal center-to-center spacing (pitch) of any two consecutive holes, in.

g = transverse center-to-center spacing (gage) between fastener gage lines, in.

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

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

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

B3. EFFECTIVE NET AREA

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

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

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

where

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

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

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

where

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

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

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

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

When the load is transmitted to a plate by longitudinal welds along both edges at the end of the plate, the length of the welds shall not be less than the width of the plate. The effective net area A_e shall be computed by Equation (B3-2).

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

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

where

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

Bolted and riveted splice and gusset plates and other connection fittings subject to tensile force shall be designed in accordance with the provisions of Sect. D1, where the effective net area shall be taken as the actual net area, except that, for the purpose of design calculations, it shall not be taken as greater than 85% of the gross area.

B4. STABILITY

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

Consideration shall be given to significant load effects resulting from the deflected shape of the structure or of individual elements of the lateral load resisting system, including effects on beams, columns, bracing, connections and shear walls.

B5. LOCAL BUCKLING

1. Classification of Steel Sections

Steel sections are classified as compact, noncompact and slender element sections. For a section to qualify as compact, its flanges must be continuously connected to the web or webs and the width-thickness ratios of its compression elements must not exceed the applicable limiting width-thickness ratios from Table B5.1. Steel sections that do not qualify as compact are classified as noncompact if the width-thickness ratios of the compression elements do not exceed the values shown for noncompact in Table B5.1. If the width-thickness ratios of any compression element exceed the latter applicable value, the section is classified as a slender element section.

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

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

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

a. For webs of rolled, built-up or formed sections, h is the clear distance between flanges.
b. For webs of rolled, built-up or formed sections, d is the full nominal depth.
c. For flange or diaphragm plates in built-up sections, the width b is the distance between adjacent lines of fasteners or lines of welds.
d. For flanges of rectangular hollow structural sections, the width b is the clear distance between webs less the inside corner radius on each side. If the corner radius is not known, the flat width may be taken as the total section width minus three times the thickness.

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

2. Slender Compression Elements

For the design of flexural and compressive sections with slender compressive elements see Appendix B5.

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

Description of Element	Width-Thickness Ratio	Limiting Width-Thickness Ratios: Compact	Limiting Width-Thickness Ratios: Noncompact[c]
Flanges of I-shaped rolled beams and channels in flexure[a]	b/t	$65/\sqrt{F_y}$	$95/\sqrt{F_y}$
Flanges of I-shaped welded beams in flexure	b/t	$65/\sqrt{F_y}$	$95/\sqrt{F_{yf}/k_c}$[e]
Outstanding legs of pairs of angles in continuous contact; angles or plates projecting from rolled beams or columns; stiffeners on plate girders	b/t	NA	$95/\sqrt{F_y}$
Angles or plates projecting from girders, built-up columns or other compression members; compression flanges of plate girders	b/t	NA	$95/\sqrt{F_y/k_c}$
Stems of tees	d/t	NA	$127/\sqrt{F_y}$
Unstiffened elements simply supported along one edge, such as legs of single-angle struts, legs of double-angle struts with separators and cross or star-shaped cross sections	b/t	NA	$76/\sqrt{F_y}$
Flanges of square and rectangular box and hollow structural sections of uniform thickness subject to bending or compression[d]; flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$190/\sqrt{F_y}$	$238/\sqrt{F_y}$
Unsupported width of cover plates perforated with a succession of access holes[b]	b/t	NA	$317/\sqrt{F_y}$
All other uniformly compressed stiffened elements, i.e., supported along two edges	b/t h/t_w	NA	$253/\sqrt{F_y}$
Webs in flexural compression[a]	d/t	$640/\sqrt{F_y}$	—
	h/t_w	—	$760/\sqrt{F_b}$
Webs in combined flexural and axial compression	d/t_w	for $f_a/F_y \leq 0.16$ $\frac{640}{\sqrt{F_y}}\left(1 - 3.74\frac{f_a}{F_y}\right)$	—
		for $f_a/F_y > 0.16$ $257/\sqrt{F_y}$	—
	h/t_w	—	$760/\sqrt{F_b}$
Circular hollow sections In axial compression In flexure	D/t	 $3{,}300/F_y$ $3{,}300/F_y$	 — —

[a]For hybrid beams, use the yield strength of the flange F_{yf} instead of F_y.
[b]Assumes net area of plate at widest hole.
[c]For design of slender sections that exceed the noncompact limits see Appendix B5.
[d]See also Sect. F3.1.
[e]$k_c = \dfrac{4.05}{(h/t)^{0.46}}$ if $h/t > 70$, otherwise $k_c = 1.0$.

B6. ROTATIONAL RESTRAINT AT POINTS OF SUPPORT

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

B7. LIMITING SLENDERNESS RATIOS

For members whose design is based on compressive force, the slenderness ratio Kl/r preferably should not exceed 200. If this limit is exceeded, the allowable stress shall not exceed the value obtained from Equation (E2-2).

For members whose design is based on tensile force, the slenderness ratio L/r preferably should not exceed 300. The above limitation does not apply to rods in tension. Members which have been designed to perform as tension members in a structural system, but experience some compression loading, need not satisfy the compression slenderness limit.

B8. SIMPLE SPANS

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

B9. 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 Chapters D through F, except that some non-elastic but self-limiting deformation of a part of the connection is permitted when this is essential to avoid overstressing of fasteners.

B10. PROPORTIONS OF BEAMS AND GIRDERS

Rolled or welded shapes, plate girders and cover-plated beams shall, in general, be proportioned by the moment of inertia of the gross section. No deduction shall be made for shop or field bolt or rivet holes in either flange provided that

$$0.5F_u A_{fn} \geq 0.6F_y A_{fg} \tag{B10-1}$$

where A_{fg} is the gross flange area and A_{fn} is the net flange area, calculated in accordance with the provisions of Sects. B1 and B2.

If

$$0.5F_u A_{fn} < 0.6F_y A_{fg} \tag{B10-2}$$

the member flexural properties shall be based on an effective tension flange area A_{fe}

$$A_{fe} = \frac{5}{6} \frac{F_u}{F_y} A_{fn} \tag{B10-3}$$

Hybrid girders may be proportioned by the moment of inertia of their gross section,* subject to the applicable provisions in Sect. G1, provided they are not required to resist an axial force greater than $0.15F_y$ times the area of the gross section, where F_y is the yield stress of the flange material. To qualify as hybrid girders, the flanges at any given section shall have the same cross-sectional area and be made of the same grade of steel.

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

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

High-strength bolts, rivets 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 bolts, rivets or intermittent welds shall be in proportion to the intensity of the shear. However, the longitudinal spacing shall not exceed the maximum permitted for compression or tension members in Sect. D2 or E4, respectively. Bolts, rivets or welds connecting flange to web shall also be proportioned to transmit to the web any loads applied directly to the flange, unless provision is made to transmit such loads by direct bearing.

Partial length cover plates shall be extended beyond the theoretical cutoff point and the extended portion shall be attached to the beam or girder by high-strength bolts in a slip-critical connection, rivets or fillet welds adequate, at the applicable stresses allowed in Sects. J2.4, J3.4, or K4, to develop the cover plate's portion of the flexural stresses in the beam or girder at the theoretical cutoff point.

In addition, for welded cover plates, the welds connecting the cover plate termination to the beam or girder in the length a', defined below, shall be adequate, at the allowed stresses, to develop the cover plate's portion of the flexural stresses in the beam or girder at the distance a' from the end of the cover plate. The length a', measured from the end of the cover plate, shall be:

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

B11. PROPORTIONING OF CRANE GIRDERS

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

*No limit is placed on the web stresses produced by the applied bending moment for which a hybrid girder is designed, except as provided in Sect. K4 and Appendix K4.

CHAPTER C

FRAMES AND OTHER STRUCTURES

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

C1. GENERAL

In addition to meeting the requirements of member strength and stiffness, frames and other continous structures shall be designed to provide the needed deformation capacity and to assure over-all frame stability.

C2. FRAME STABILITY

1. Braced Frames

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

2. Unbraced Frames

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

CHAPTER D

TENSION MEMBERS

This section applies to prismatic members subject to axial tension caused by forces acting through the centroidal axis. For members subject to combined axial tension and flexure, see Sect. H2. For members subject to fatigue, see Sect. K4. For tapered members, see Appendix F7. For threaded rods see Sect. J3.

I. ALLOWABLE STRESS

The allowable stress F_t shall not exceed $0.60F_y$ on the gross area nor $0.50F_u$ on the effective net area. In addition, pin-connected members shall meet the requirements of Sect. D3.1 at the pin hole.

Block shear strength shall be checked at end connections of tension members in accordance with Sect. J4.

Eyebars shall meet the requirements of Sect. D3.1.

D2. BUILT-UP MEMBERS

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

24 times the thickness of the thinner plate, nor 12 in. for painted members or unpainted members not subject to corrosion.

14 times the thickness of the thinner plate, nor 7 in. for unpainted members of weathering steel subject to atmospheric corrosion.

In a tension member the longitudinal spacing of fasteners and intermittent welds connecting two or more shapes in contact shall not exceed 24 inches. Tension members composed of two or more shapes or plates separated 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 300.

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

The spacing of tie plates shall be such that the slenderness ratio of any component in the length between tie plates should preferably not exceed 300.

D3. PIN-CONNECTED MEMBERS

1. Allowable Stress

The allowable stress on the net area of the pin hole for pin-connected members is 0.45 F_y. The bearing stress on the projected area of the pin shall not exceed the stress allowed in Sect. J8.

The allowable stress on eyebars meeting the requirements of Sect. D3.3 is 0.60 F_y on the body area.

2. Pin-connected Plates

The minimum net area beyond the pin hole, parallel to the axis of the member, shall not be less than ⅔ of the net area across the pin hole.

The distance used in calculations, transverse to the axis of pin-connected plates or any individual element of a built-up member, from the edge of the pin hole to the edge of the member or element shall not exceed 4 times the thickness at the pin hole. For calculation purposes, the distance from the edge of the pin hole to the edge of the plate or to the edge of a separated element of a built-up member at the pin hole, shall not be assumed to be more than 0.8 times the diameter of the pin hole.

For pin-connected members in which the pin is expected to provide for relative movement between connected parts while under full load, the diameter of the pin hole shall not be more than 1/32 in. greater than the diameter of the pin.

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

3. Eyebars

Eyebars shall be of uniform thickness, without reinforcement at the pin holes, and have circular heads whose periphery is concentric with the pin hole. The radius of the transition between the circular head and the eyebar body shall not be less than the diameter of the head.

For calculation purposes, the width of the body of an eyebar shall not exceed 8 times its thickness.

The thickness may be less than ½-in. only if external nuts are provided to tighten pin plates and filler plates into snug contact. For calculation purposes, the distance from the hole edge to plate edge perpendicular to the direction of the applied load shall not be less than ⅔ nor greater than ¾ times the width of the eyebar body.

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

The pin-hole diameter shall be no more than 1/32-in. greater than the pin diameter.

For steel having a yield stress greater than 70 ksi, the hole diameter shall not exceed 5 times the plate thickness and the width of the eyebar shall be reduced accordingly.

CHAPTER E

COLUMNS AND OTHER COMPRESSION MEMBERS

This section applies to prismatic members with compact and noncompact sections subject to axial compression through the centroidal axis. For members with slender elements, see Appendix B5.2. For members subject to combined axial compression and flexure, see Chap. H. For tapered members, see Appendix F7.

E1. EFFECTIVE LENGTH AND SLENDERNESS RATIO

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

In determining the slenderness ratio of an axially loaded compression member, the length shall be taken as its effective length Kl and r as the corresponding radius of gyration. For limiting slenderness ratios, see Sect. B7.

E2. ALLOWABLE STRESS

On the gross section of axially loaded compression members whose cross sections meet the provisions of Table B5.1, when Kl/r, the largest effective slenderness ratio of any unbraced segment is less than C_c, the allowable stress is:

$$F_a = \frac{\left[1 - \dfrac{(Kl/r)^2}{2C_c^2}\right] F_y}{\dfrac{5}{3} + \dfrac{3(Kl/r)}{8C_c} - \dfrac{(Kl/r)^3}{8C_c^3}} \qquad \text{(E2-1)}$$

where

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

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

$$F_a = \frac{12\pi^2 E}{23(Kl/r)^2} \qquad \text{(E2-2)}$$

E3. FLEXURAL-TORSIONAL BUCKLING

Singly symmetric and unsymmetric columns, such as angles or tee-shaped columns, and doubly symmetric columns such as cruciform or built-up columns with very thin walls, may require consideration of flexural-torsional and torsional buckling.

E4. BUILT-UP MEMBERS

All parts of built-up compression members and the transverse spacing of their lines of fasteners shall meet the requirements of Sect. B7.

For spacing and edge distance requirements for weathering steel members, see Sect. J3.10.

At the ends of built-up compression members bearing on base plates or milled surfaces, all components in contact with one another shall be connected by rivets or bolts spaced longitudinally not more than 4 diameters apart for a distance equal to 1½ times the maximum width of the member, or by continuous welds having a length not less than the maximum width of the member.

The longitudinal spacing for intermediate bolts, rivets or intermittent welds in built-up members shall be adequate to provide for the transfer of calculated stress. The maximum longitudinal spacing of bolts, rivets or intermittent welds connecting two rolled shapes in contact shall not exceed 24 in. In addition, for painted members and unpainted members not subject to corrosion where the outside component consists of a plate, the maximum longitudinal spacing shall not exceed:

$127/\sqrt{F_y}$ times the thickness of the outside plate nor 12 in. when fasteners are not staggered along adjacent gage lines.

$190/\sqrt{F_y}$ times the thickness of the outside plate nor 18 in. when fasteners are staggered along adjacent gage lines.

Compression members composed of two or more rolled shapes separated by intermittent fillers shall be connected at these fillers at intervals such that the slenderness ratio Kl/r of either shape, between the fasteners, does not exceed ¾ times 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. At least two intermediate connectors shall be used along the length of the built-up member.

All connections, including those at the ends, shall be welded or shall utilize high-strength bolts tightened to the requirements of Table J3.7.

Open sides of compression members built up from plates or shapes shall be provided with lacing having tie plates at each end and at intermediate points if the lacing is interrupted. Tie plates shall be as near the ends as practicable. In main members carrying calculated stress, the end tie plates shall have a length of not less than the distance between the lines of fasteners or welds connecting them to the components of the member. Intermediate tie plates shall have a length not less than ½ of this distance. The thickness of tie plates shall not be less than 1/50 of the distance between the lines of fasteners or welds connecting them to the components of the member. In bolted and riveted construction, the spacing in the direction of stress in tie plates shall not be more than 6 diameters and the tie plates shall be connected to each component by at least 3 fasteners. In welded construction, the welding on each line connecting a tie plate shall aggregate not less than ⅓ the length of the plate.

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 ¾ times 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% 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. For lacing bars in compression the unsupported length of the lacing bar shall be taken as the distance between fasteners or welds connecting it to the components of the built-up member for single lacing, and 70% of that distance for double lacing. The inclination of lacing bars to the axis of the member shall preferably be not less than 60° for single lacing and 45° for double lacing. When the distance between the lines of fasteners or welds in the flanges is more than 15 in., the lacing preferably shall be double or be made of angles.

The function of tie plates and lacing may be performed by continuous cover plates perforated with access holes. The unsupported width of such plates at access holes, as defined in Sect. B5, is assumed available to resist axial stress, provided that: the width-to-thickness ratio conforms to the limitations of Sect. B5; the ratio of length (in direction of stress) to width of holes 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 fasteners or welds; and the periphery of the holes at all points shall have a minimum radius of 1½ in.

E5. PIN-CONNECTED COMPRESSION MEMBERS

Pin-connections of pin-connected compression members shall conform to the requirements of Sect. D3.

E6. COLUMN WEB SHEAR

Column connections must be investigated for concentrated force introduction in accordance with Sect. K1.

CHAPTER F

BEAMS AND OTHER FLEXURAL MEMBERS

Beams shall be distinguished from plate girders on the basis of the web slenderness ratio h/t_w. When this value is greater than $970/\sqrt{F_y}$ the allowable bending stress is given in Chapter G. The allowable shear stresses and stiffener requirements are given in Chapter F unless tension field action is used, then the allowable shear stresses are given in Chapter G.

This chapter applies to singly or doubly symmetric beams including hybrid beams and girders loaded in the plane of symmetry. It also applies to channels loaded in a plane passing through the shear center parallel to the web or restrained against twisting at load points and points of support. For members subject to combined flexural and axial force, see Sect. H1.

F1. ALLOWABLE STRESS: STRONG AXIS BENDING OF I-SHAPED MEMBERS AND CHANNELS

1. Members with Compact Sections

For members with compact sections as defined in Sect. B5.1 (excluding hybrid beams and members with yield points greater than 65 ksi) symmetrical about, and loaded in, the plane of their minor axis the allowable stress is

$$F_b = 0.66\ F_y \tag{F1-1}$$

provided the flanges are connected continuously to the web or webs and the laterally unsupported length of the compression flange L_b does not exceed the value of L_c, as given by the smaller of:

$$\frac{76b_f}{\sqrt{F_y}} \text{ or } \frac{20{,}000}{(d/A_f)\ F_y} \tag{F1-2}$$

Members (including composite members and excluding hybrid members and members with yield points greater than 65 ksi) which meet the requirements for compact sections and are continuous over supports or rigidly framed to columns, may be proportioned for 9/10 of the negative moments produced by gravity loading when such moments are maximum at points of support, provided that, for such members, the maximum positive moment is 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 is permitted in proportioning the column for the combined axial and bending loading, provided that the stress f_a due to any concurrent axial load on the member, does not exceed $0.15F_a$.

2. Members with Noncompact Sections

For members meeting the requirements of Sect. F1.1 except that their flanges are noncompact (excluding built-up members and members with yield points greater than 65 ksi), the allowable stress is

$$F_b = F_y \left[0.79 - 0.002 \frac{b_f}{2t_f} \sqrt{F_y} \right] \tag{F1-3}$$

For built-up members meeting the requirements of Sect. F1.1 except that their flanges are noncompact and their webs are compact or noncompact, (excluding hybrid girders and members with yield points greater than 65 ksi) the allowable stress is

$$F_b = F_y \left[0.79 - 0.002 \frac{b_f}{2t_f} \sqrt{\frac{F_y}{k_c}} \right] \tag{F1-4}$$

where

$$k_c = \frac{4.05}{(h/t_w)^{0.46}} \text{ if } h/t_w > 70, \text{ otherwise } k_c = 1.0.$$

For members with a noncompact section (Sect. B5), but not included above, and loaded through the shear center and braced laterally in the region of compression stress at intervals not exceeding $76b_f/\sqrt{F_y}$, the allowable stress is

$$F_b = 0.60\, F_y \tag{F1-5}$$

3. Members with Compact or Noncompact Sections with Unbraced Length Greater than L_c

For flexural members with compact or noncompact sections as defined in Sect. B5.1, and with unbraced lengths greater than L_c as defined in Sect. F1.1, the allowable bending stress in tension is determined from Equation (F1-5).

For such members with an axis of symmetry in, and loaded in the plane of their web, the allowable bending stress in compression is determined as the larger value from Equations (F1-6) or (F1-7) and (F1-8), except that Equation (F1-8) is applicable only to sections with a compression flange that is solid and approximately rectangular in cross section and that has an area not less than the tension flange. Higher values of the allowable compressive stress are permitted if justified by a more precise analysis. Stresses shall not exceed those permitted by Chapter G, if applicable.

For channels bent about their major axis, the allowable compressive stress is determined from Equation (F1-8).

When

$$\sqrt{\frac{102 \times 10^3 C_b}{F_y}} \le \frac{l}{r_T} \le \sqrt{\frac{510 \times 10^3 C_b}{F_y}}:$$

$$F_b = \left[\frac{2}{3} - \frac{F_y\,(l/r_T)^2}{1530 \times 10^3 C_b}\right] F_y \le 0.60\, F_y \qquad \text{(F1-6)}$$

When

$$\frac{l}{r_T} \ge \sqrt{\frac{510 \times 10^3 C_b}{F_y}}:$$

$$F_b = \frac{170 \times 10^3 C_b}{(l/r_T)^2} \le 0.60\, F_y \qquad \text{(F1-7)}$$

For any value of l/r_T:

$$F_b = \frac{12 \times 10^3 C_b}{ld/A_f} \le 0.60\, F_y \qquad \text{(F1-8)}$$

where

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

r_T = radius of gyration of a section comprising the compression flange plus ⅓ of the compression web area, taken about an axis in the plane of the web, in.

A_f = area of the compression flange, in.^{2}

C_b = $1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2$, but not more than 2.3;* where M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length, taken about the strong axis of the member, and where M_1/M_2, the ratio of end moments, is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite signs (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, the value of C_b shall be taken as unity. When computing F_{bx} to be used in Equation (H1-1), C_b may be computed by the equation given above for frames subject to joint translation, and it shall be taken as unity for frames braced against joint translation. C_b may conservatively be taken as unity for cantilever beams.**

*It is conservative to take C_b as unity. For values smaller than 2.3, see Table 6 in the Numerical Values Section.

**For the use of larger C_b values, see Galambos (1988).

For hybrid plate girders, F_y for Equations (F1-6) and (F1-7) is the yield stress of the compression flange. Equation (F1-8) shall not apply to hybrid girders.

Sect. F1.3 does not apply to tee sections if the stem is in compression anywhere along the unbraced length.

F2. ALLOWABLE STRESS: WEAK AXIS BENDING OF I-SHAPED MEMBERS, SOLID BARS AND RECTANGULAR PLATES

Lateral bracing is not required for members loaded through the shear center about their weak axis nor for members of equal strength about both axes.

1. Members With Compact Sections

For doubly symmetrical I- and H-shape members with compact flanges (Sect. B5) continuously connected to the web and bent about their weak axes (except members with yield points greater than 65 ksi); solid round and square bars; and solid rectangular sections bent about their weaker axes, the allowable stress is

$$F_b = 0.75\ F_y \tag{F2-1}$$

2. Members With Noncompact Sections

For members not meeting the requirements for compact sections of Sect. B5 and not covered in Sect. F3, bent about their minor axis, the allowable stress is

$$F_b = 0.60\ F_y \tag{F2-2}$$

Doubly symmetrical I- and H-shape members bent about their weak axes (except members with yield points greater than 65 ksi) with noncompact flanges (Sect. B5) continuously connected to the web may be designed on the basis of an allowable stress of

$$F_b = F_y \left[1.075 - 0.005 \left(\frac{b_f}{2t_f}\right) \sqrt{F_y}\right] \tag{F2-3}$$

F3. ALLOWABLE STRESS: BENDING OF BOX MEMBERS, RECTANGULAR TUBES AND CIRCULAR TUBES

1. Members With Compact Sections

For members bent about their strong or weak axes, members with compact sections as defined in Sect. B5 and flanges continuously connected to the webs, the allowable stress is

$$F_b = 0.66\ F_y \tag{F3-1}$$

To be classified as a compact section, a box-shaped member shall have, in addition to the requirements in Sect. B5, a depth not greater than 6 times the width, a flange thickness not greater than 2 times the web thickness and a laterally unsupported length L_b less than or equal to

$$L_c = \left(1{,}950 + 1{,}200\,\frac{M_1}{M_2}\right)\frac{b}{F_y} \tag{F3-2}$$

except that it need not be less than 1,200 (b/F_y), where M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length, taken about the strong axis of the member, and where M_1/M_2, the ratio of end moments, is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite signs (single curvature bending).

2. Members With Noncompact Sections

For box-type and tubular flexural members that meet the noncompact section requirements of Sect. B5, the allowable stress is

$$F_b = 0.60\ F_y \tag{F3-3}$$

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

F4. ALLOWABLE SHEAR STRESS

For $h/t_w \leq 380/\sqrt{F_y}$, on the overall depth times the web thickness, the allowable shear stress is

$$F_v = 0.40\ F_y \tag{F4-1}$$

For $h/t_w > 380/\sqrt{F_y}$, the allowable shear stress is on the clear distance between flanges times the web thickness is

$$F_v = \frac{F_y}{2.89}(C_v) \leq 0.40F_y \tag{F4-2}$$

where

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

$$= \frac{190}{h/t_w}\sqrt{\frac{k_v}{F_y}} \text{ when } C_v \text{ is more than 0.8}$$

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

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

t_w = thickness of web, in.
a = clear distance between transverse stiffeners, in.
h = clear distance between flanges at the section under investigation, in.

For shear rupture on coped beam end connections see Sect. J4.

Maximum h/t_w limits are given in Chapter G.

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

F5. TRANSVERSE STIFFENERS

Intermediate stiffeners are required when the ratio h/t_w is greater than 260 and the maximum web shear stress f_v is greater than that permitted by Equation (F4-2).

The spacing of intermediate stiffeners, when required, shall be such that the web shear stress will not exceed the value for F_v given by Equation (F4-2) or (G3-1), as applicable, and

$$\frac{a}{h} \leq \left[\frac{260}{(h/t_w)}\right]^2 \text{ and } 3.0 \qquad \text{(F5-1)}$$

F6. BUILT-UP MEMBERS

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 ft. Through-bolts and separators are permitted, provided that, in beams having a depth of 12 in. 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.

F7. WEB-TAPERED MEMBERS

See Appendix F7.

CHAPTER G

PLATE GIRDERS

Plate girders shall be distinguished from beams on the basis of the web slenderness ratio h/t_w. When this value is greater than $970/\sqrt{F_y}$, the provisions of this chapter shall apply for allowable bending stress, otherwise Chapter F is applicable.

For allowable shear stress and transverse stiffener design see appropriate sections in Chapter F or this chapter if tension field action is utilized.

G1. WEB SLENDERNESS LIMITATIONS

When no transverse stiffeners are provided or when transverse stiffeners are spaced more than 1½ times the distance between flanges

$$\frac{h}{t_w} \leq \frac{14{,}000}{\sqrt{F_{yf}(F_{yf} + 16.5)}} \tag{G1-1}$$

When transverse stiffeners are provided, spaced not more than 1½ times the distance between flanges

$$\frac{h}{t_w} \leq \frac{2{,}000}{\sqrt{F_{yf}}} \tag{G1-2}$$

G2. ALLOWABLE BENDING STRESS

When the web depth-to-thickness ratio exceeds $970/\sqrt{F_y}$, the maximum bending stress in the compression flange shall not exceed

$$F_b' \leq F_b \, R_{PG} \, R_e \tag{G2-1}$$

where

F_b = applicable bending stress given in Chapter F, ksi

$$R_{PG} = 1 - 0.0005 \frac{A_w}{A_f}\left(\frac{h}{t} - \frac{760}{\sqrt{F_b}}\right) \leq 1.0$$

$$R_e = \frac{12 + \left(\frac{A_w}{A_f}\right)(3\alpha - \alpha^3)}{12 + 2\left(\frac{A_w}{A_f}\right)} \leq 1.0$$

(non-hybrid girders, $R_e = 1.0$)

A_w = area of web at the section under investigation, in.^{2}

A_f = area of compression flange, in.^{2}

α = $0.6 \, F_{yw}/F_b \leq 1.0$

G3. ALLOWABLE SHEAR STRESS WITH TENSION FIELD ACTION

Except as herein provided, the largest average web shear, f_v, in kips per sq. in., computed for any condition of complete or partial loading, shall not exceed the value given by Equation (F4-2).

Alternatively, for girders other than hybrid girders, if intermediate stiffeners are provided and spaced to satisfy the provisions of Sect. G4 and if $C_v \leq 1$, the allowable shear including tension field action given by Equation (G3-1) is permitted in lieu of the value given by Equation (F4-2).

$$F_v = \frac{F_y}{2.89}\left[C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}}\right] \leq 0.40F_y \qquad \text{(G3-1)*}$$

G4. TRANSVERSE STIFFENERS

Transverse stiffeners shall meet the requirements of Sect. F5.

In girders designed on the basis of tension field action, the spacing between stiffeners at end panels, at panels containing large holes, and at panels adjacent to panels containing large holes shall be such that f_v does not exceed the value given by Equation (F4-2).

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

The moment of inertia, I_{st}, of a pair of intermediate stiffeners, or a single intermediate stiffener, with reference to an axis in the plane of the web, shall be limited as follows

$$I_{st} \geq \left(\frac{h}{50}\right)^4 \qquad \text{(G4-1)}$$

The gross area (*total* area, when stiffeners are furnished in pairs), in sq. in., of intermediate stiffeners spaced as required for Equation (G3-1) shall be not less than

$$A_{st} = \frac{1 - C_v}{2}\left[\frac{a}{h} - \frac{(a/h)^2}{\sqrt{1 + (a/h)^2}}\right] YDht \qquad \text{(G4-2)}$$

where

C_v, a, h, and t are as defined in Sect. F4
Y = ratio of yield stress of web steel to yield stress of stiffener steel
D = 1.0 for stiffeners furnished in pairs
= 1.8 for single angle stiffeners
= 2.4 for single plate stiffeners

*Equation (G3-1) recognizes the contribution of tension field action.

When the greatest shear stress f_v in a panel is less than that permitted by Equation (G3-1), the reduction of this gross area requirement is permitted in like proportion.

Intermediate stiffeners required by Equation (G3-1) shall be connected for a total shear transfer, in kips per linear inch of single stiffener or pair of stiffeners, not less than

$$f_{vs} = h \sqrt{\left(\frac{F_y}{340}\right)^3} \tag{G4-3}$$

where F_y = yield stress of web steel.

This shear transfer may be reduced in the same proportion that the largest computed shear stress f_v in the adjacent panels is less than that permitted by Equation (G3-1). However, rivets and welds in intermediate stiffeners which are required to transmit to the web an applied concentrated load or reaction shall be proportioned for not less than the applied load or reaction.

Intermediate stiffeners may be stopped short of the tension flange, provided bearing is not needed to transmit a concentrated load or reaction. The weld by which intermediate stiffeners are attached to the web shall be terminated not closer than 4 times nor more than 6 times the web thickness from the near toe of the web-to-flange weld. When single stiffeners are used, they shall be attached to the compression flange, if it consists of a rectangular plate, to resist any uplift tendency due to torsion in the plate. When lateral bracing is attached to a stiffener, or a pair of stiffeners, in turn, these shall be connected to the compression flange to transmit 1% of the total flange stress, unless the flange is composed only of angles.

G5. COMBINED SHEAR AND TENSION STRESS

Plate girder webs which depend upon tension field action, as provided in Equation (G3-1), shall be so proportioned that bending tensile stress, due to moment in the plane of the girder web, shall not exceed $0.60F_y$ nor

$$\left(0.825 - 0.375 \frac{f_v}{F_v}\right) F_y \tag{G5-1}$$

where

f_v = computed average web shear stress (total shear divided by web area), ksi

F_v = allowable web shear stress according to Equation (G3-1), ksi

The allowable shear stress in the webs of girders having flanges and webs with yield point greater than 65 ksi shall not exceed the values given by Equation (F4-2) if the flexural stress in the flange f_b exceeds $0.75F_b$.

CHAPTER H

COMBINED STRESSES

The strength of members subjected to combined stresses shall be determined according to the provisions of this chapter.

This chapter pertains to doubly and singly symmetrical members only. See Chapter E for determination of F_a and Chapter F for determination of F_{bx} and F_{by}.

H1. AXIAL COMPRESSION AND BENDING

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

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}} \leq 1.0 \qquad \text{(H1-1)}$$

$$\frac{f_a}{0.60 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \qquad \text{(H1-2)}$$

When $f_a/F_a \leq 0.15$, Equation (H1-3) is permitted in lieu of Equations (H1-1) and (H1-2):

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \qquad \text{(H1-3)}$$

In Equations (H1-1), (H1-2) and (H1-3), the subscripts x and y, combined with subscripts b, m and e, indicate the axis of bending about which a particular stress or design property applies, and

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

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

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

= Euler stress divided by a factor of safety, ksi (In the expression for F'_e, l_b is the actual unbraced length *in the plane of bending* and r_b is the corresponding radius of gyration. K is the effective length factor *in the plane of bending*.) As in the case of F_a, F_b and $0.60F_y$, F'_e may be increased ⅓ in accordance with Sect. A5.2.

f_a = computed axial stress, ksi

f_b = computed compressive bending stress at the point under consideration, ksi

C_m= Coefficient whose value shall be taken as follows:

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

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

$$C_m = 0.6 - 0.4\ (M_1/M_2)$$

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

c. For compression members in frames braced against joint translation in the plane of loading and subjected to transverse loading between their supports, the value of C_m may be determined by an analysis. However, in lieu of such analysis, the following values are permitted:

i. For members whose ends are restrained against rotation in the plane of bending .... $C_m = 0.85$

ii. For members whose ends are unrestrained against rotation in the plane of bending .... $C_m = 1.0$

H2. AXIAL TENSION AND BENDING

Members subject to both axial tension and bending stresses shall be proportioned at all points along their length to satisfy the following equation:

$$\frac{f_a}{F_t} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \qquad \text{(H2-1)}$$

where f_b is the computed bending tensile stress, f_a is the computed axial tensile stress, F_b is the allowable bending stress and F_t is the governing allowable tensile stress defined in Sect. D1.

However the computed bending compressive stress arising from an independent load source relative to the axial tension, taken above, shall not exceed the applicable value required in Chapter F.

CHAPTER I

COMPOSITE CONSTRUCTION

This chapter applies to steel beams supporting a reinforced concrete slab* so interconnected that the beams and the slab act together to resist bending. Simple and continuous composite beams with shear connectors and concrete-encased beams, constructed with or without temporary shores, are included.

I1. DEFINITION

Two cases of composite members are recognized: Totally encased members which depend upon natural bond for interaction with the concrete and those with shear connectors (mechanical anchorage to the slab) with the steel member not necessarily encased.

A beam totally encased in concrete cast integrally with the slab may be assumed to be connected to the concrete by natural bond, without additional anchorage, provided that:

1. Concrete cover over beam sides and soffit is at least 2 in.
2. The top of the beam is at least 1½ in. below the top and 2 in. above bottom of the slab.
3. Concrete encasement contains adequate mesh or other reinforcing steel throughout the whole depth and across the soffit of the beam to prevent spalling of the concrete.

Shear connectors must be provided for composite action if the steel member is not totally encased in concrete. The portion of the effective width of the concrete slab on each side of the beam centerline shall not exceed:

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

I2. DESIGN ASSUMPTIONS

1. Encased beams shall be proportioned to support, unassisted, all dead loads applied prior to the hardening of the concrete (unless these loads are supported temporarily on shoring) and, acting in conjunction with the slab, to support all dead and live loads applied after hardening of the concrete, without exceeding a computed bending stress of $0.66F_y$, where F_y is the yield stress of the steel beam. The bending stress produced by loads after the concrete has hardened shall be computed on the basis of the section properties of the composite section. Concrete tension stresses shall be ne-

*See Commentary Sect. I2.

glected. Alternatively, the steel beam alone may be proportioned to resist, unassisted, the positive moment produced by all loads, live and dead, using a bending stress equal to $0.76F_y$, in which case temporary shoring is not required.

2. When shear connectors are used in accordance with Sect. I4, the composite section shall be proportioned to support all of the loads without exceeding the allowable stress prescribed in Sect. F1.1, even when the steel section is not shored during construction. In positive moment areas, the steel section is exempt from compact flange criteria (Sect. B5) and there is no limit on the unsupported length of the compression flange.

   Reinforcement parallel to the beam within the effective width of the slab, when anchored in accordance with the provisions of the applicable building code, may be included in computing the properties of composite sections, provided shear connectors are furnished in accordance with the requirements of Sect. I4. The section properties of the composite section shall be computed in accordance with the elastic theory. Concrete tension stresses shall be neglected. For stress computations, the compression area of lightweight or normal weight concrete shall be treated as an equivalent area of steel by dividing it by the modular ratio n for normal weight concrete of the strength specified when determining the section properties. For deflection calculations, the transformed section properties shall be based on the appropriate modular ratio n for the strength and weight concrete specified, where $n = E/E_c$.

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

$$S_{eff} = S_s + \sqrt{\frac{V_h'}{V_h}}\,(S_{tr} - S_s) \tag{I2-1}$$

   where

   V_h and V_h' are as defined in Sect. I4
   S_s = section modulus of the steel beam referred to its bottom flange, in.^{3}
   S_{tr} = section modulus of the transformed composite section referred to its bottom flange, based upon maximum permitted effective width of concrete flange (Sect. I1), in.^{3}

   For composite beams constructed without temporary shoring, stresses in the steel section shall not exceed $0.90F_y$. Stresses shall be computed assuming the steel section alone resists all loads applied before the concrete has reached 75% of its required strength and the effective composite section resists all loads applied after that time.

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

I3. END SHEAR

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

I4. SHEAR CONNECTORS

Except in the case of encased beams, as defined in Sect. I2.1, the entire horizontal shear at the junction of the steel beam and the concrete slab shall be assumed to be transferred by shear connectors welded to the top flange of the beam and embedded in the concrete. For full composite action with concrete subject to flexural compression, the total horizontal shear to be resisted between the point of maximum positive moment and points of zero moment shall be taken as the smaller value using Equations (I4-1) and (I4-2):

$$V_h = 0.85 f'_c A_c / 2 \qquad \text{(I4-1)*}$$

and

$$V_h = F_y A_s / 2 \qquad \text{(I4-2)}$$

where

f'_c = specified compression strength of concrete, ksi
A_c = actual area of effective concrete flange defined in Sect. I1, in.^{2}
A_s = area of steel beam, in.^{2}

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

$$V_h = F_{yr} A_{sr} / 2 \qquad \text{(I4-3)}$$

where

A_{sr} = total area of longitudinal reinforcing steel at the interior support located within the effective flange width specified in Sect. I1, in.^{2}
F_{yr} = specified minimum yield stress of the longitudinal reinforcing steel, ksi

For full composite action, the number of connectors resisting the horizontal shear, V_h, each side of the point of maximum moment, shall not be less than that determined by the relationship V_h/q, where q, the allowable shear load for one connector, is given in Table I4.1 for flat soffit concrete slabs made with ASTM C33 aggregates. For flat soffit concrete slabs made with rotary kiln produced aggregates, conforming to ASTM C330 with concrete unit weight not less than 90 pcf, the allowable shear load for one connector is obtained by multiplying the values from Table I4.1 by the coefficient from Table I4.2.

For partial composite action with concrete subject to flexural compression, the horizontal shear V'_h to be used in computing S_{eff} shall be taken as the product

* The term $\frac{1}{2} F_{yr} A'_s$ shall be added to the right-hand side of Equation (I4-1) if longitudinal reinforcing steel with area A'_s located within the effective width of the concrete flange is included in the properties of the composite section.

of q times the number of connectors furnished between the point of maximum moment and the nearest point of zero moment.

The value of V_h' shall not be less than ¼ the smaller value of Equation (I4-1), using the maximum permitted effective width of the concrete flange, or Equation (I4-2). The effective moment of inertia for deflection computations shall be determined by:

$$I_{eff} = I_s + \sqrt{\frac{V_h'}{V_h}}\,(I_{tr} - I_s) \tag{I4-4}$$

where

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

The connectors required each side of the point of maximum moment in an area of positive bending may be uniformly distributed between that point and adjacent points of zero moment, except that N_2, the number of shear connectors

Table I4.1
Allowable Horizontal Shear Load for One Connector (q), kips[a]

Connector[b]	Specified Compressive Strength of Concrete (f_c'), ksi		
	3.0	3.5	≥4.0
½″ dia. × 2″ hooked or headed stud	5.1	5.5	5.9
⅝″ dia. × 2½″ hooked or headed stud	8.0	8.6	9.2
¾″ dia. × 3″ hooked or headed stud	11.5	12.5	13.3
⅞″ dia. × 3½″ hooked or headed stud	15.6	16.8	18.0
Channel C3 × 4.1	4.3w^c	4.7w^c	5.0w^c
Channel C4 × 5.4	4.6w^c	5.0w^c	5.3w^c
Channel C5 × 6.7	4.9w^c	5.3w^c	5.6w^c

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

Table I4.2
Coefficients for Use with Concrete Made with C330 Aggregates

Specified Compressive Strength of Concrete (f_c')	Air Dry Unit Weight of Concrete, pcf						
	90	95	100	105	110	115	120
≤4.0 ksi	0.73	0.76	0.78	0.81	0.83	0.86	0.88
≥5.0 ksi	0.82	0.85	0.87	0.91	0.93	0.96	0.99

required between any concentrated load in that area and the nearest point of zero moment, shall be not less than that determined by Equation (I4-5).

$$N_2 = \frac{N_1 \left[\dfrac{M\beta}{M_{max}} - 1 \right]}{\beta - 1} \tag{I4-5}$$

where

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

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

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

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

Shear connectors shall have at least 1 in. of lateral concrete cover, except for connectors installed in the ribs of formed steel decks. Unless located directly over the web, the diameter of studs shall not be greater than 2½ times the thickness of the flange to which they are welded. The minimum center-to-center spacing of stud connectors shall be 6 diameters along the longitudinal axis of the supporting composite beam and 4 diameters transverse to the longitudinal axis of the supporting composite beam. The maximum center-to-center spacing of stud connectors shall not exceed 8 times the total slab thickness.

I5. COMPOSITE BEAMS OR GIRDERS WITH FORMED STEEL DECK

Composite construction of concrete slabs on formed steel deck connected to steel beams or girders shall be designed by the applicable portions of Sects. I1 through I4, with the following modifications.

1. General

1. Section I5 is applicable to decks with nominal rib height not greater than 3 inches.
2. The average width of concrete rib or haunch w_r shall be not less than 2 in., but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck. See Sect. I5.3, subparagraphs 2 and 3, for additional provisions.
3. The concrete slab shall be connected to the steel beam or girder with welded stud shear connectors ¾ in. or less in diameter (AWS D1.1, Sect. 7, Part F). Studs may be welded through the deck or directly to the steel member.
4. Stud shear connectors shall extend not less than 1½ in. above the top of the steel deck after installation.
5. The slab thickness above the steel deck shall be not less than 2 in.

2. Deck Ribs Oriented Perpendicular to Steel Beam or Girder

1. Concrete below the top of the steel deck shall be neglected when determining section properties and in calculating A_c for Equation (I4-1).

2. The spacing of stud shear connectors along the length of a supporting beam or girder shall not exceed 36 in.
3. The allowable horizontal shear load per stud connector q shall be the value stipulated in Sect. I4 (Tables I4.1 and I4.2) multiplied by the following reduction factor:

$$\left(\frac{0.85}{\sqrt{N_r}}\right)\left(\frac{w_r}{h_r}\right)\left(\frac{H_s}{h_r} - 1.0\right) \le 1.0 \qquad \text{(I5-1)}$$

where

h_r = nominal rib height, in.
H_s = length of stud connector after welding, in., not to exceed the value (h_r + 3) in computations, although the actual length may be greater
N_r = number of stud connectors on a beam in one rib, not to exceed 3 in computations, although more than 3 studs may be installed.
w_r = average width of concrete rib, in. (see Sect. I5.1, subparagraph 2)

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

3. Deck Ribs Oriented Parallel to Steel Beam or Girder

1. Concrete below the top of the steel deck may be included when determining section properties and shall be included in calculating A_c for Equation (I4-1).
2. Steel deck ribs over supporting beams or girders may be split longitudinally and separated to form a concrete haunch.
3. When the nominal depth of steel deck is 1½ in. or greater, the average width w_r of the supported haunch or rib shall be not less than 2 in. for the first stud in the transverse row plus 4 stud diameters for each additional stud.
4. The allowable horizontal shear load per stud connector q shall be the value stipulated in Sect. I4 (Tables I4.1 and I4.2), except when the ratio w_r/h_r is less than 1.5, the allowable load shall be multiplied by the following reduction factor:

$$0.6\left(\frac{w_r}{h_r}\right)\left(\frac{H_s}{h_r} - 1.0\right) \le 1.0 \qquad \text{(I5-2)}$$

where h_r and H_s are as defined in Sect. I5.2 and w_r is the average width of concrete rib or haunch (see Sect. I5.1, subparagraph 2, and Sect. I5.3, subparagraph 3).

I6. SPECIAL CASES

When composite construction does not conform to the requirements of Sects. I1 through I5, the allowable load per shear connector must be established by a suitable test program.

CHAPTER J

CONNECTIONS, JOINTS AND FASTENERS

This chapter applies to connections consisting of connecting elements (plates, stiffeners, gussets, angles, brackets) and connectors (welds, bolts, rivets).

J1. GENERAL PROVISIONS

1. Design Basis

Connections shall be proportioned so that the calculated stress is less than the allowable stress determined (1) by structural analysis for loads acting on the structure or (2) as a specified proportion of the strength of the connected members, whichever is appropriate.

2. Simple Connections

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

3. Moment Connections

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

4. Compression Members with Bearing Joints

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

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

All compression joints shall be proportioned to resist any tension developed by the specified lateral loads acting in conjunction with 75% of the calculated dead-load stress and no live load.

5. Connections of Tension and Compression Members in Trusses

The connections at ends of tension or compression members in trusses shall develop the force due to the design load, but not less than 50% of the effective strength of the member, unless a smaller percentage is justified by engineering analysis that considers other factors including handling, shipping and erection.

6. Minimum Connections

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

7. Splices in Heavy Sections

This section applies to ASTM A6 Group 4 and 5 rolled shapes, or shapes built-up by welding plates more than 2 in. thick together to form the cross section*, and where the cross section is to be spliced and subject to primary tensile stresses due to tension or flexure.

When tensile forces in these sections are to be transmitted through splices by full-penetration groove welds, material notch-toughness requirements as given in Sect. A3.1c, weld access holes details as given in Sect. J1.8, compatible welding procedures as given in Sect. J2.6, welding preheat requirements as given in Sect. J2.7 and thermal cut surface preparation and inspection requirements as given in Sect. M2.2 apply.

At tension splices in these sections, weld tabs and backing shall be removed and the surfaces ground smooth.

When splicing these sections, and where the section is to be used as a primary compression member, all weld access holes required to facilitate groove welding operations shall satisfy the provisions of Sect. J1.8.

Alternatively, splicing of such members subject to compression, including members which are subject to tension due to wind or seismic loads, may be accomplished using splice details which do not induce large weld shrinkage strains such as partial-penetration flange groove welds with fillet-welded surface lap plate splices on the web, or with bolted or combination bolted/fillet-welded lap plate splices.

8. Beam Copes and Weld Access Holes

All weld access holes required to facilitate welding operations shall have a length from the toe of the weld preparation not less than 1½ times the thickness of the material in which the hole is made. The height of the access hole shall be adequate for deposition of sound weld metal in the adjacent plates and provide clearance for weld tabs. In hot rolled shapes and built-up shapes, all beam copes and weld access holes shall be shaped free of notches or sharp re-entrant corners except that, when fillet web-to-flange welds are used in built-up shapes, access holes are permitted to terminate perpendicular to the flange.

For Group 4 and 5 shapes and built-up shapes of material more than 2 in. thick, the thermally cut surfaces of beam copes and weld access holes shall be ground

*When the individual elements of the cross section are spliced prior to being joined to form the cross section in accordance with AWS D1.1, Article 3.4.6, the applicable provisions of AWS D1.1 apply in lieu of the requirements of this Section.

to bright metal and inspected by either magnetic particle or dye penetrant methods. If the curved transition portion of weld access holes and beam copes are formed by predrilled or sawed holes, that portion of the access hole or cope need not be ground. Weld access holes and beam copes in other shapes need not be ground nor inspected by dye penetrant or magnetic particle.

9. Placement of Welds, Bolts and Rivets

Groups of welds, bolts or rivets at the ends of any member which transmit axial stress into that member shall be sized so the center of gravity of the group coincides with the center of gravity of the member, unless provision is made for the eccentricity. The foregoing provision is not applicable to end connections of statically loaded single-angle, double-angle and similar members. Eccentricity between the gravity axes of such members and the gage lines for their riveted or bolted end connections may be neglected in statically loaded members, but shall be considered in members subject to fatigue loading.

See Sect. J3.10 for placement of fasteners in built-up members made of weathering steel.

10. Bolts in Combination with Welds

In new work, 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 proportioned for slip-critical connections may be considered as sharing the stress with the welds.

In making welded alterations to structures, existing rivets and high-strength bolts tightened to the requirements for slip-critical connections are permitted for carrying stresses resulting from loads present at the time of alteration, and the welding need be adequate to carry only the additional stress.

11. High-strength Bolts in Slip-Critical Connections in Combination with Rivets

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

12. Limitations on Bolted and Welded Connections

Fully-tensioned high-strength bolts (see Table J3.7) or welds shall be used for the following connections:

Column splices in all tier structures 200 ft or more in height
Column splices in tier structures 100 to 200 ft in height, if the least horizontal dimension is less than 40% of the height
Column splices in tier structures less than 100 ft in height, if the least horizontal dimension is less than 25% of the height
Connections of all beams and girders to columns and of any other beams and girders on which the bracing of columns is dependent, in structures over 125 ft in height

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

Connections for supports of running machinery or of other live loads which produce impact or reversal of stress

Any other connections stipulated on the design plans.

In all other cases, connections may be made with high-strength bolts tightened to the snug-tight condition or with A307 bolts.

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

J2. WELDS

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

1. Groove Welds

a. Effective Area

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

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

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

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

The effective throat thickness of a flare groove weld when flush to the surface of a bar or 90° bend in a formed section shall be as shown in Table J2.2. Random sections of production welds for each welding procedure, or such test sections as may be required by design documents, shall be used to verify that the effective throat is consistently obtained.

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

b. Limitations

The minimum effective throat thickness of a partial-penetration groove weld shall be as shown in Table J2.3. Minimum effective throat thickness is determined by the thicker of the two parts joined, except that the weld size need not exceed the thickness of the thinnest part joined. For this exception, particular care shall be taken to provide sufficient preheat for soundness of the weld.

TABLE J2.1
Effective Throat Thickness of Partial-penetration Groove Welds

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

TABLE J2.2
Effective Throat Thickness of Flare Groove Welds

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

[a]Use $\frac{3}{8}R$ for Gas Metal Arc Welding (except short circuiting transfer process) when $R \geq$ ½-in.

TABLE J2.3
Minimum Effective Throat Thickness of Partial-penetration Groove Welds

Material Thickness of Thicker Part Joined (in.)	Minimum Effective Throat Thickness[a] (in.)
To ¼ inclusive	⅛
Over ¼ to ½	3/16
Over ½ to ¾	¼
Over ¾ to 1½	5/16
Over 1½ to 2¼	⅜
Over 2¼ to 6	½
Over 6	⅝

[a]See Sect. J2.

2. Fillet Welds

a. Effective Area

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

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

The effective throat thickness of a fillet weld shall be the shortest distance from the root of the joint to the face of the diagrammatic weld, except that for fillet welds made by the submerged arc process, the effective throat thickness shall be taken equal to the leg size for ⅜-in. and smaller fillet welds, and equal to the theoretical throat plus 0.11-in. for fillet welds larger than ⅜-in.

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

b. Limitations

The *minimum size of fillet welds* shall be as shown in Table J2.4. Minimum weld size is dependent upon the thicker of the two parts joined, except that the weld size need not exceed the thickness of the thinner part. For this exception, particular care shall be taken to provide sufficient preheat for soundness of the weld. Weld sizes larger than the thinner part joined are permitted if required by calculated strength. In the as-welded condition, the distance between the edge of the base metal and the toe of the weld may be less than 1/16-in. provided the weld size is clearly verifiable.

The *maximum size of fillet welds* that is permitted along edges of connected parts shall be:

- Material less than ¼-in. thick, not greater than the thickness of the material.
- Material ¼-in. or more in thickness, not greater than the thickness of the material minus 1/16-in., unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness.

TABLE J2.4
Minimum Size of Fillet Welds

Material Thickness of Thicker Part Joined (in.)	Minimum Size of Fillet Weld[a] (in.)
To ¼ inclusive	⅛
Over ¼ to ½	3/16
Over ½ to ¾	¼
Over ¾	5/16

[a]Leg dimension of fillet welds. Single-pass welds must be used.

The *minimum effective length of fillet welds* designed on the basis of strength shall be not less than 4 times the nominal size, or else the size of the weld shall be considered not to exceed ¼ of its effective length. If longitudinal fillet welds are used alone in end connections of flat bar tension members, the length of each fillet weld shall be not less than the perpendicular distance between them. The transverse spacing of longitudinal fillet welds used in end connections of tension members shall not exceed 8 in., unless the member is designed on the basis of effective net area in accordance with Sect. B3.

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

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

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

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

3. Plug and Slot Welds

a. Effective Area

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

b. Limitations

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

The diameter of the hole for a plug weld shall not be less than the thickness of the part containing it plus 5/16-in., rounded to the next larger odd 1/16-in., nor greater than the minimum diameter plus 1/8-in. or 2¼ times the thickness of the weld.

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

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

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

The thickness of plug or slot welds in material 5/8-in. or less in thickness shall be equal to the thickness of the material. In material over 5/8-in. thick, the thickness of the weld shall be at least ½ the thickness of the material but not less than 5/8-in.

4. Allowable Stresses

Except as modified by the provisions of Sect. K4, welds shall be proportioned to meet the stress requirements given in Table J2.5.

5. Combination of Welds

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

6. Mixed Weld Metal

When notch-toughness is specified, the process consumables for all weld metal, tack welds, root pass and subsequent passes, deposited in a joint shall be compatible to assure notch-tough composite weld metal.

7. Preheat for Heavy Shapes

For ASTM A6 Group 4 and 5 shapes and welded built-up members made of plates more than 2 in. thick, a preheat equal to or greater than 350°F shall be used when making groove weld splices.

TABLE J2.5
Allowable Stress on Welds[f]

Type of Weld and Stress[a]	Allowable Stress	Required Weld Strength Level[b,c]
Complete-penetration Groove Welds		
Tension normal to effective area	Same as base metal	"Matching" weld metal shall be used.
Compression normal to effective area	Same as base metal	Weld metal with a strength level equal to or less than "matching" weld metal is permitted.
Tension or compression parallel to axis of weld	Same as base metal	
Shear on effective area	0.30 × nominal tensile strength of weld metal (ksi)	
Partial-penetration Groove Welds[d]		
Compression normal to effective area	Same as base metal	Weld metal with a strength level equal to or less than "matching" weld metal is permitted.
Tension or compression parallel to axis of weld[e]	Same as base metal	
Shear parallel to axis of weld	0.30 × nominal tensile strength of weld metal (ksi)	
Tension normal to effective area	0.30 × nominal tensile strength of weld metal (ksi), except tensile stress on base metal shall not exceed 0.60 × yield stress of base metal	
Fillet Welds		
Shear on effective area	0.30 × nominal tensile strength of weld metal (ksi)	Weld metal with a strength level equal to or less than "matching" weld metal is permitted.
Tension or compression Parallel to axis of weld[e]	Same as base metal	
Plug and Slot Welds		
Shear parallel to faying surfaces (on effective area)	0.30 × nominal tensile strength of weld metal (ksi)	Weld metal with a strength level equal to or less than "matching" weld metal is permitted.

[a]For definition of effective area, see Sect. J2.
[b]For "matching" weld metal, see Table 4.1.1, AWS D1.1.
[c]Weld metal one strength level stronger than "matching" weld metal will be permitted.
[d]See Sect. J2.1b for a limitation on use of partial-penetration groove welded joints.
[e]Fillet welds and partial-penetration groove welds joining the component elements of built-up members, such as flange-to-web connections, may be designed without regard to the tensile or compressive stress in these elements parallel to the axis of the welds.
[f]The design of connected material is governed by Chapters D through G. Also see Commentary Sect. J2.4.

J3. BOLTS, THREADED PARTS AND RIVETS

1. High-strength Bolts

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

If required to be tightened to more than 50% of their minimum specified tensile strength, ASTM A449 bolts in tension and bearing-type shear connections shall have an ASTM F436 hardened washer installed under the bolt head, and the nuts shall meet the requirements of ASTM A563.

2. Size and Use of Holes

a. The *maximum sizes* of holes for bolts are given in Table J3.1, except that larger holes, required for tolerance on location of anchor bolts in concrete foundations, are permitted in column base details.

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

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

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

e. *Long-slotted holes* are permitted in only one of the connected parts of either a slip-critical or bearing-type connection at an individual faying surface. Long-slotted holes are permitted without regard to direction of loading in slip-critical connections, but shall be normal to the direction of load in bearing-type connections. Where long-slotted holes are used in an outer ply, plate washers or a continuous bar with standard holes, having a size suf-

TABLE J3.1
Nominal Hole Dimensions

Bolt Dia.	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-slot (Width × length)	Long-slot (Width × length)
½	9/16	⅝	9/16 × 11/16	9/16 × 1¼
⅝	11/16	13/16	11/16 × ⅞	11/16 × 1 9/16
¾	13/16	15/16	13/16 × 1	13/16 × 1⅞
⅞	15/16	1 1/16	15/16 × 1⅛	15/16 × 2 3/16
1	1 1/16	1¼	1 1/16 × 1 5/16	1 1/16 × 2½
≥1⅛	d + 1/16	d + 5/16	(d + 1/16) × (d + ⅜)	(d + 1/16) × (2.5 × d)

ficient to completely cover the slot after installation, shall be provided. In high-strength bolted connections, such plate washers or continuous bars shall be not less than 5⁄16-in. thick and shall be of structural grade material, but need not be hardened. If hardened washers are required for use of high-strength bolts, the hardened washers shall be placed over the outer surface of the plate washer or bar.

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

3. Effective Bearing Area

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

4. Allowable Tension and Shear

Allowable tension and shear stresses on bolts, threaded parts and rivets shall be as given in Table J3.2, in ksi of the nominal body area of rivets (before driving) or the unthreaded nominal body area of bolts and threaded parts other than upset rods (see footnote c, Table J3.2). High-strength bolts supporting applied load by direct tension shall be so proportioned that their average tensile stress, computed on the basis of nominal bolt area and independent of any initial tightening force, will not exceed the appropriate stress given in Table J3.2. The applied load shall be the sum of the external load and any tension resulting from prying action produced by deformation of the connected parts.

When specified by the designer, the nominal slip resistance for connections having special faying surface conditions may be increased to the applicable values in the *RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts*.

Finger shims up to ¼-in. may be introduced into slip-critical connections designed on the basis of standard holes without reducing the allowable shear stress of the fastener to that specified for slotted holes.

Design for bolts, threaded parts and rivets subject to fatigue loading shall be in accordance with Appendix K4.3.

5. Combined Tension and Shear in Bearing-type Connections

Bolts and rivets subject to combined shear and tension shall be so proportioned that the tension stress F_t in ksi on the nominal body area A_b produced by forces applied to the connected parts, shall not exceed the values computed from the equations in Table J3.3, where f_v, the shear stress produced by the same forces, shall not exceed the value for shear given in Table J3.2. When allowable stresses are increased for wind or seismic loads in accordance with Sect. A5.2, the constants in the equations listed in Table J3.3 shall be increased by ⅓, but the coefficient applied to f_v shall not be increased.

TABLE J3.2
Allowable Stress on Fasteners, ksi

Description of Fasteners	Allowable Tension[g] (F_t)	Allowable Shear[g] (F_v)				
		Slip-critical Connections[e,i]				Bearing-type Connections[i]
		Standard size Holes	Oversized and Short-slotted Holes	Long-slotted holes		
				Transverse[j] Load	Parallel[j] Load	
A502, Gr. 1, hot-driven rivets	23.0[a]					17.5[f]
A502, Gr. 2 and 3, hot-driven rivets	29.0[a]					22.0[f]
A307 bolts	20.0[a]					10.0[b,f]
Threaded parts meeting the requirements of Sects. A3.1 and A3.4 and A449 bolts meeting the requirements of Sect. A3.4, when threads are not excluded from shear planes	$0.33F_u$[a,c,h]					$0.17F_u$[h]
Threaded parts meeting the requirements of Sects. A3.1 and A3.4, and A449 bolts meeting the requirements of Sect. A3.4, when threads are excluded from shear planes	$0.33F_u$[a,h]					$0.22F_u$[h]
A325 bolts, when threads are not excluded from shear planes	44.0[d]	17.0	15.0	12.0	10.0	21.0[f]
A325 bolts, when threads are excluded from shear planes	44.0[d]	17.0	15.0	12.0	10.0	30.0[f]
A490 bolts, when threads are not excluded from shear planes	54.0[d]	21.0	18.0	15.0	13.0	28.0[f]
A490 bolts, when threads are excluded from shear planes	54.0[d]	21.0	18.0	15.0	13.0	40.0[f]

[a]Static loading only.
[b]Threads permitted in shear planes.
[c]The tensile capacity of the threaded portion of an upset rod, based upon the cross-sectional area at its major thread diameter A_b shall be larger than the nominal body area of the rod before upsetting times $0.60F_y$.
[d]For A325 and A490 bolts subject to tensile fatigue loading, see Appendix K4.3.
[e]Class A (slip coefficient 0.33). Clean mill scale and blast-cleaned surfaces with Class A coatings. When specified by the designer, the allowable shear stress, F_v, for slip-critical connections having special faying surface conditions may be increased to the applicable value given in the RCSC Specification.
[f]When bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 50 in., tabulated values shall be reduced by 20%.
[g]See Sect. A5.2
[h]See Table 2, Numerical Values Section for values for specific ASTM steel specifications.
[i]For limitations on use of oversized and slotted holes, see Sect. J3.2.
[j]Direction of load application relative to long axis of slot.

6. Combined Tension and Shear in Slip-critical Joints

For A325 and A490 bolts used in slip-critical connections, the maximum shear stress allowed by Table J3.2 shall be multiplied by the reduction factor $(1 - f_t A_b / T_b)$, where f_t is the average tensile stress due to a direct load applied to all of the bolts in a connection and T_b is the pretension load of the bolt specified in Table J3.7. When allowable stresses are increased for wind or seismic loads in accordance with the provisions of Sect. A5.2, the reduced allowable shear stress shall be increased by ⅓.

7. Allowable Bearing at Bolt Holes

On the projected area of bolts and rivets in shear connections with the end distance in the line of force not less than $1\frac{1}{2}\,d$ and the distance c. to c. of bolts not less than $3d$:

1. In standard-or short-slotted holes with two or more bolts in the line of force,

$$F_p = 1.2\,F_u \qquad \text{(J3-1)}$$

where

F_p = allowable bearing stress, ksi

2. In long-slotted holes with the axis of the slot perpendicular to the direction of load and with two or more bolts in the line of force,

$$F_p = 1.0\,F_u \qquad \text{(J3-2)}$$

On the projected area of the bolt or rivet closest to the edge in standard or short-slotted holes with the edge distance less than $1\frac{1}{2}d$ and in all connections with a single bolt in the line of force:

$$F_p = L_e F_u / 2d \leq 1.2\,F_u \qquad \text{(J3-3)}$$

TABLE J3.3
Allowable Tension Stress F_t for Fasteners in Bearing-type Connections

Description of Fasteners	Threads Included in Shear Plane	Threads Excluded from Shear Plane
A307 bolts	$26 - 1.8f_v \leq 20$	
A325 bolts	$\sqrt{(44)^2 - 4.39f_v^2}$	$\sqrt{(44)^2 - 2.15f_v^2}$
A490 bolts	$\sqrt{(54)^2 - 3.75f_v^2}$	$\sqrt{(54)^2 - 1.82f_v^2}$
Threaded parts, A449 bolts over 1½-in. dia.	$0.43F_u - 1.8f_v \leq 0.33F_u$	$0.43F_u - 1.4f_v \leq 0.33F_u$
A502 Gr. 1 rivets	$30 - 1.3f_v \leq 23$	
A502 Gr. 2 rivets	$38 - 1.3f_v \leq 29$	

where,

L_e = distance from the free edge to center of the bolt, in.
d = bolt dia., in.

If deformation around the hole is not a design consideration and adequate spacing and edge distance is as required by Sects. J3.8 and J3.9, the following equation is permitted in lieu of Equation (J3-1):

$$F_p = 1.5\ F_u \tag{J3-4}$$

and the limit in Equation (J3-3) shall be increased to $1.5F_u$.

8. Minimum Spacing

The distance between centers of standard, oversized or slotted fastener holes shall not be less than 2⅔ times the nominal diameter of the fastener* nor less than that required by the following paragraph, if applicable.

Along a line of transmitted forces, the distance between centers of holes s shall be not less than $3d$ when F_p is determined by Equations (J3-1) and (J3-2). Otherwise, the distance between centers of holes shall not be less than the following:

a. For standard holes:

$$s \geq 2P/F_u t + d/2 \tag{J3-5}$$

where

P = force transmitted by one fastener to the critical connected part, kips
F_u = specified minimum tensile strength of the critical connected part, ksi
t = thickness of the critical connected part, in.

b. For oversized and slotted holes, the distance required for standard holes in subparagraph a, (above), plus the applicable increment C_1 from Table J3.4, but the clear distance between holes shall not be less than one bolt diameter.

9. Minimum Edge Distance

The distance from the center of a standard hole to an edge of a connected part shall be not less than the applicable value from Table J3.5 nor the value from Equation (J3-6), as applicable.

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

$$L_e \geq 2P/F_u t \tag{J3-6}$$

where P, F_u, t are defined in Sect. J3.8.

*A distance of $3d$ is preferred.

TABLE J3.4
Values of Spacing Increment C_1, in.

Nominal Dia. of Fastener	Oversize Holes	Slotted Holes		
		Perpendicular to Line of Force	Parallel to Line of Force	
			Short-slots	Long-slots[a]
≤ 7/8	1/8	0	3/16	1 1/2d − 1/16
1	3/16	0	1/4	1 7/16
≥1 1/8	1/4	0	5/16	1 1/2d − 1/16

[a]When length of slot is less than maximum allowed in Table J3.1, C_1 may be reduced by the difference between the maximum and actual slot lengths.

TABLE J3.5
Minimum Edge Distance, in.
(Center of Standard Hole[a] to Edge of Connected Part)

Nominal Bolt or Rivet Dia. (in.)	At Sheared Edges	At Rolled Edges of Plates, Shapes or Bars, Gas Cut or Saw-cut Edges[b]
1/2	7/8	3/4
5/8	1 1/8	7/8
3/4	1 1/4	1
7/8	1 1/2[c]	1 1/8
1	1 3/4[c]	1 1/4
1 1/8	2	1 1/2
1 1/4	2 1/4	1 5/8
Over 1 1/4	1 3/4 × Dia.	1 1/4 × Dia.

[a]For oversized or slotted holes, see Table J3.6.
[b]All edge distances in this column may be reduced 1/8-in. when the hole is at a point where stress does not exceed 25% of the maximum design strength in the element.
[c]These may be 1 1/4 in. at the ends of beam connection angles.

TABLE J3.6
Values of Edge Distance Increment C_2, in.

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

[a]When length of slot is less than maximum allowable (see Table J3.1), C_2 may be reduced by one-half the difference between the maximum and actual slot lengths.

The distance from the center of an oversized or slotted hole to an edge of a connected part shall be not less than required for a standard hole plus the applicable increment C_2 from Table J3.6.

10. Maximum Edge Distance and Spacing

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

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

11. Long Grips

A307 bolts which carry calculated stress, with a grip exceeding five diameters, shall have their number increased 1% for each additional 1/16 in. in the grip.

J4. ALLOWABLE SHEAR RUPTURE

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

$$F_v = 0.30F_u \qquad \text{(J4-1)}$$

TABLE J3.7
Minimum Pretension for Fully-tightened Bolts, kips[a]

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

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

acting on the net shear area A_v and,

$$F_t = 0.50F_u \tag{J4-2}$$

acting on the net tension area A_t.

The minimum net failure path on the periphery of welded connections shall be checked.*

J5. CONNECTING ELEMENTS

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

1. Eccentric Connections

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

2. Allowable Shear Rupture

For situations where failure might occur by shear along a plane through the fasteners, or by a combination of shear along a plane through the fasteners plus tension along a perpendicular plane, see Sect. J4.

J6. FILLERS

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

When bolts or rivets carrying computed stress pass through fillers thicker than ¼-in., except in slip-critical connections assembled with high-strength bolts, the fillers shall be extended beyond the splice material and the filler extension shall be secured by enough bolts or rivets to distribute the total stress in the member uniformly over the combined section of the member and the filler, or an equivalent number of fasteners shall be included in the connection. Fillers between ¼-in. and ¾-in. thick, inclusive, need not be extended and developed, provided the allowable shear stress in the bolts is reduced by the factor, $0.4\,(t-0.25)$, where t is the total thickness of the fillers, up to ¾ in.

*See Sects. B2 and Commentary Figs. C-J4.1, C-J4.2, C-J4.3 and C-J4.4.

J7. SPLICES

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

J8. ALLOWABLE BEARING STRESS

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

$$F_p = 0.90F_y^* \tag{J8-1}$$

Expansion rollers and rockers, kips per lin. in.:

$$F_p = \left(\frac{F_y - 13}{20}\right) 0.66d \tag{J8-2}$$

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

J9. COLUMN BASES AND BEARING ON MASONRY AND CONCRETE

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

In the absence of code regulations the following stresses apply:

On sandstone and limestone .... $F_p = 0.40$ ksi
On brick in cement mortar .... $F_p = 0.25$ ksi
On the full area of a concrete support .... $F_p = 0.35f'_c$
On less than the full area of a concrete support................ $F_p = 0.35f'_c\sqrt{A_2/A_1} \leq 0.70f'_c$

where

f'_c = specified compressive strength of concrete, ksi
A_1 = area of steel concentrically bearing on a concrete support, in.^{2}
A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2}

J10. ANCHOR BOLTS

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

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

CHAPTER K

SPECIAL DESIGN CONSIDERATIONS

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

K1. WEBS AND FLANGES UNDER CONCENTRATED FORCES

1. Design Basis

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

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

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

2. Local Flange Bending

A pair of stiffeners shall be provided opposite the tension flange or flange plate of the beam or girder framing into the member when the thickness of the member flange t_f is less than

$$0.4\sqrt{\frac{P_{bf}}{F_{yc}}} \tag{K1-1}$$

where

F_{yc} = column yield stress, ksi

P_{bf} = the computed force delivered by the flange or moment connection plate multiplied by 5/3, when the computed force is due to live and dead load only, or by 4/3,* when the computed force is due to live and dead load in conjunction with wind or earthquake forces, kips

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

*Except where other codes may govern. For example, see Section 4(D) "Recommended Lateral Force Requirements and Commentary," Structural Engineers Assoc. of California, 1975.

3. Local Web Yielding

Bearing stiffeners shall be provided if the compressive stress at the web toe of the fillets resulting from concentrated loads exceeds $0.66F_y$.

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

$$\frac{R}{t_w(N + 5k)} \leq 0.66F_y \tag{K1-2}$$

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

$$\frac{R}{t_w(N + 2.5k)} \leq 0.66F_y \tag{K1-3}$$

where

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

4. Web Crippling

Bearing stiffeners shall be provided in the webs of members under concentrated loads, when the compressive force exceeds the following limits:

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

$$R = 67.5t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{F_{yw}\, t_f/t_w} \tag{K1-4}$$

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

$$R = 34t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{F_{yw}\, t_f/t_w} \tag{K1-5}$$

where

F_{yw} = specified minimum yield stress of beam web, ksi
d = overall depth of the member, in.
t_f = flange thickness, in.

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

5. Sidesway Web Buckling

Bearing stiffeners shall be provided in the webs of members with flanges not restrained against relative movement by stiffeners or lateral bracing and subject to concentrated compressive loads, when the compressive force exceeds the following limits:

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

$$R = \frac{6{,}800 t_w^3}{h}\left[1 + 0.4\left(\frac{d_c/t_w}{l/b_f}\right)^3\right] \quad \text{(K1-6)}$$

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

$$R = \frac{6{,}800 t_w^3}{h}\left[0.4\left(\frac{d_c/t_w}{l/b_f}\right)^3\right] \quad \text{(K1-7)}$$

where

l = largest laterally unbraced length along either flange at the point of load, in.

b_f = flange width, in.

d_c = $d - 2k$ = web depth clear of fillets, in.

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

6. Compression Buckling of the Web

For unstiffened portions of webs of members under concentrated loads to both flanges, a stiffener or a pair of stiffeners shall be provided opposite the compression flange when the web depth clear of fillets d_c is greater than

$$\frac{4100\, t_{wc}{}^3\sqrt{F_{yc}}}{P_{bf}} \quad \text{(K1-8)}$$

where

t_{wc} = thickness of column web, in.

7. Compression Members with Web Panels Subject to High Shear

Members subject to high shear stress in the web should be checked for conformance with Sect. F4.*

8. Stiffener Requirements for Concentrated Loads

Stiffeners shall be placed in pairs at unframed ends or at points of concentrated loads on the interior of beams, girders or columns if required by Sect. K1.2 through K1.6, as applicable.

If required by Sects. K1.2, K1.3 or Equation (K1-9), stiffeners need not extend more than one-half the depth of the web, except as follows:

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

When the load normal to the flange is tensile, the stiffeners shall be welded to

*See Commentary Sect. E6.

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

When flanges or moment connection plates for end connections of beams and girders are welded to the flange of an I- or H-shape column, a pair of column-web stiffeners having a combined cross-sectional area A_{st} not less than that computed from Equation (K1-9) shall be provided whenever the calculated value of A_{st} is positive.

$$A_{st} = \frac{P_{bf} - F_{yc}t_{wc}\,(t_b + 5k)}{F_{yst}} \qquad \text{(K1-9)}$$

where

F_{yst} = stiffener yield stress, ksi
k = distance between outer face of column flange and web toe of its fillet, if column is a rolled shape, or equivalent distance if column is a welded shape, in.
t_b = thickness of flange or moment connection plate delivering concentrated force, in.

Stiffeners required by the provisions of Equation (K1-9) and Sects. K1.2 and K1.6 shall comply with the following criteria:

1. The width of each stiffener plus ½ the thickness of the column web shall be not less than ⅓ the width of the flange or moment connection plate delivering the concentrated force.
2. The thickness of stiffeners shall be not less than one-half the thickness of the flange or plate delivering the concentrated load.*
3. The weld joining stiffeners to the column web shall be sized to carry the force in the stiffener caused by unbalanced moments on opposite sides of the column.

K2. PONDING

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

The roof system shall be considered stable and not requiring further investigation if:

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

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

where

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

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

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

*See Commentary Sect. K1 for comment on width-thickness ratio for stiffeners.

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

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

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

K3. TORSION

The effects of torsion shall be considered in the design of members and the normal and shearing stresses due to torsion shall be added to those from all other loads, with the resultants not exceeding the allowable values.

K4. FATIGUE

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

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

CHAPTER L

SERVICEABILITY DESIGN CONSIDERATIONS

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

Limiting values of structural behavior to ensure serviceability (e.g., maximum deflections, accelerations, etc.) shall be chosen with due regard to the intended function of the structure.

L1. CAMBER

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

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

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

L2. EXPANSION AND CONTRACTION

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

L3. DEFLECTION, VIBRATION AND DRIFT

1. Deflection

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

2. Vibration

Beams and girders supporting large open floor areas free of partitions or other sources of damping shall be designed with due regard for vibration.

L4. CONNECTION SLIP

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

L5. CORROSION

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

Where beams are exposed they shall be sealed against corrosion of interior surfaces or spaced sufficiently far apart to permit cleaning and painting.

CHAPTER M

FABRICATION, ERECTION AND QUALITY CONTROL

M1. 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 welds, bolts and rivets, shall be prepared in advance of the actual fabrication. These drawings shall clearly distinguish between shop and field welds and bolts and shall clearly identify type of high-strength bolted connection (snug-tight or fully-tightened bearing, or slip-critical).

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

M2. FABRICATION

1. Cambering, Curving and Straightening

Local application of heat or mechanical means are permitted to introduce or correct camber, curvature and straightness. The temperature of heated areas, as measured by approved methods, shall not exceed 1050°F for A852 steel, 1100°F for A514 steel nor 1200°F for other steels. The same limits apply for equivalent grades of A709 steels.

2. Thermal Cutting

Thermally cut free edges which will be subject to substantial tensile stress shall be free of gouges greater than 3/16-in. Gouges greater than 3/16-in. deep and sharp notches shall be removed by grinding or repaired by welding. Thermally cut edges which are to have weld deposited upon them, shall be reasonably free of notches or gouges.

All reentrant corners shall be shaped to a smooth transition. If specific contour is required, it must be shown on the contract documents.

Beam copes and weld access holes shall meet the geometrical requirements of Sect. J1.8. Beam copes and weld access holes in ASTM A6 Group 4 and 5 shapes and welded built-up shapes with material thickness greater than 2 in. shall be preheated to a temperature of not less than 150°F prior to thermal cutting.

3. Planing of Edges

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

4. Welded Construction

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

5. High-strength Bolted Construction—Assembly

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

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

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.

The orientation of fully inserted finger shims, with a total thickness of not more than ¼-in. within a joint, is independent of the direction of application of the load.

When assembled, all joint surfaces, including surfaces adjacent to the bolt head and nut, shall be free of scale, except tight mill scale and shall be free of dirt or other foreign material. Burrs that would prevent solid seating of the connected parts in the snug-tight condition shall be removed. Contact surfaces within slip-critical connections shall be free of oil, paint, lacquer or other coatings, except as listed in Table 3 of the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts.*

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

6. Compression Joints

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

7. Dimensional Tolerances

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

8. Finishing of Column Bases

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

a. Rolled steel bearing plates 2 in. or less in thickness are permitted without milling,* provided a satisfactory contact bearing is obtained; rolled steel bearing plates over 2 in. but not over 4 in. in thickness may be straightened by pressing, or if presses are not available, by milling for all bearing surfaces (except as noted in subparagraphs c. and d. of this section), to obtain a satisfactory contact bearing; rolled steel bearing plates over 4 in. thick shall be milled for all bearing surfaces (except as noted in subparagraphs c. and d. of this section).
b. Column bases other than rolled steel bearing plates shall be milled for all bearing surfaces (except as noted in subparagraphs c. and d. of this section).
c. The bottom surfaces of bearing plates and column bases which are grouted to insure full bearing contact on foundations need not be milled.
d. The top surfaces of base plates with columns full-penetration welded need not be pressed or milled.

M3. SHOP PAINTING

1. General Requirements

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

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

2. Inaccessible Surfaces

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

3. Contact Surfaces

Paint is permitted unconditionally in bearing-type connections. For slip-critical connections, the faying surface requirements shall be in accordance with the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts,* paragraph 3.(b).

*See Commentary Sect. J8.

4. Finished Surfaces

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

5. Surfaces Adjacent to Field Welds

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

M4. ERECTION

1. Alignment of Column Bases

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

2. Bracing

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

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

3. Alignment

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

4. Fit of Column Compression Joints

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

5. Field Welding

Shop paint on surfaces adjacent to welds shall be wire-brushed to reduce paint film to a minimum.

6. Field Painting

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

7. Field Connections

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

M5. QUALITY CONTROL

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

1. Cooperation

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

2. Rejections

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

3. Inspection of Welding

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

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

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

The inspection of slip-critical, high-strength bolted connections shall be in accordance with the provisions of the RCSC *Allowable Stress Design Specification for Structural Joints Using ASTM A325 or A490 Bolts.*

5. Identification of Steel

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

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

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

CHAPTER N

PLASTIC DESIGN

N1. SCOPE

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

Rigid frames shall satisfy the requirements for Type 1 construction in the plane of the frame, as provided in Sect. A2.2. This does not preclude the use of some simple connections, provided provisions of Sect. N3 are satisfied. Type 2 construction is permitted for members between rigid frames. Connections joining a portion of a structure designed on the basis of plastic behavior with a portion not so designed need be no more rigid than ordinary seat-and-top-angle or ordinary web connections.

Where plastic design is used as the basis for proportioning continuous beams and structural frames, the provisions relating to allowable stress are waived. Except as modified by these rules, however, all other pertinent provisions of Chapters A through M 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.

N2. STRUCTURAL STEEL

Structural steel shall conform to one of the following specifications:

- Structural Steel, ASTM A36
- High-strength Low-alloy Structural Steel, ASTM A242
- High-strength Low-alloy Structural Manganese Vanadium Steel, ASTM A441
- Structural Steel with 42 ksi Minimum Yield Point, ASTM A529
- High-strength Low-alloy Columbium-Vanadium Steels of Structural Quality, ASTM A572
- High-strength Low-alloy Structural Steel with 50 ksi Minimum Yield Point to 4-in. Thick, ASTM A588

*As used here, "interior support" includes a rigid-frame knee formed by the junction of a column and a sloping or horizontal beam or girder.

N3. BASIS FOR MAXIMUM STRENGTH DETERMINATION

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

1. Stability of Braced Frames

The vertical bracing system for a plastically designed braced multi-story frame shall be adequate, as determined by an analysis, to:

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

It is permitted to consider that the vertical bracing system functions together with in-plane shear-resisting exterior and interior walls, floor slabs and roof decks, if these walls, slabs and decks are secured to the structural frames. The columns, girders, beams and diagonal members, when used as the vertical bracing system, could be considered to comprise a vertical-cantilever, simply connected truss in the analyses for frame buckling and lateral stability. Axial deformation of all members in the vertical bracing system shall be included in the lateral stability analysis. The axial force in these members caused by factored gravity plus factored horizontal loads shall not exceed $0.85P_y$, where P_y is the product of yield stress times the profile area of the member.

Girders and beams included in the vertical bracing system of a braced multi-story frame shall be proportioned for axial force and moment caused by the concurrent factored horizontal and gravity loads, in accordance with Equation (N4-2), with P_{cr} taken as the maximum axial strength of the beam, based on the actual slenderness ratio between braced points in the plane of bending.

2. Stability of Unbraced Frames

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

N4. COLUMNS

In the plane of bending of columns which would develop a plastic hinge at ultimate loading, the slenderness ratio l/r shall not exceed C_c, defined in Sect. E2.

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

$$P_{cr} = 1.7F_aA \tag{N4-1}$$

where A is the gross area of the member and F_a, as defined by Equation (E2-1), is based upon the applicable slenderness ratio.

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

$$\frac{P}{P_{cr}} + \frac{C_m M}{\left(1 - \frac{P}{P_e}\right) M_m} \leq 1.0 \tag{N4-2}$$

$$\frac{P}{P_y} + \frac{M}{1.18 M_p} \leq 1.0; \quad M \leq M_p \tag{N4-3}$$

in which

M = maximum factored moment, kip-ft
P = factored axial load, kips
P_e = Euler buckling load, kips
= $(23/12)F_e'A$, where F_e' is as defined in Sect. H1.
C_m = coefficient defined in Sect. H1.
M_m = maximum moment that can be resisted by the member in the absence of axial load, kip-ft
M_p = plastic moment, kip-ft
= F_yZ
Z = plastic section modulus, in.

For columns braced in the weak direction:

$$M_m = M_{px} \tag{N4-4}$$

For columns unbraced in the weak direction:

$$M_m = \left[1.07 - \frac{(l/r_y)\sqrt{F_y}}{3160}\right] M_{px} \leq M_{px} \tag{N4-5}$$

N5. SHEAR

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

$$V \leq 0.55 F_y t_w d \tag{N5-1}$$

where

V = shear that would be produced by the required factored loading, kips
d = depth of the member, in.
t_w = web thickness, in.

N6. WEB CRIPPLING

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

At points on a member where the concentrated load delivered by the flanges of a member framing into it would produce web crippling opposite the compression flange or high-tensile stress in the connection of the tension flange, web stiffeners are required in accordance with the provisions of Sect. K1.

N7. MINIMUM THICKNESS (WIDTH-THICKNESS RATIOS)

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

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

It is permitted to take the thickness of sloping flanges as their average thickness.

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

The depth-thickness ratio of webs of members subject to plastic bending shall not exceed the value given by Equation (N7-1) or (N7-2), as applicable.

$$\frac{d}{t} = \frac{412}{\sqrt{F_y}}\left(1 - 1.4\frac{P}{P_y}\right) \quad \text{when} \quad \frac{P}{P_y} \leq 0.27 \tag{N7-1}$$

$$\frac{d}{t} = \frac{257}{\sqrt{F_y}} \quad \text{when} \quad \frac{P}{P_y} > 0.27 \tag{N7-2}$$

N8. CONNECTIONS

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

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

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

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

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

N9. LATERAL BRACING

Members shall be braced adequately to resist lateral and torsional displacements at the plastic hinge locations associated with the failure mechanism. The laterally unsupported distance l_{cr} from such braced hinge locations to similarly braced adjacent points on the member or frame shall not exceed the value determined from Equation (N9-1) or (N9-2), as applicable.

$$\frac{l_{cr}}{r_y} = \frac{1375}{F_y} + 25 \quad \text{when} \quad +1.0 > \frac{M}{M_p} > -0.5 \tag{N9-1}$$

$$\frac{l_{cr}}{r_y} = \frac{1375}{F_y} \quad \text{when} \quad -0.5 \geq \frac{M}{M_p} > -1.0 \tag{N9-2}$$

where

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

The foregoing provisions need not apply in the region of the last hinge to form in the failure mechanism assumed as the basis for proportioning a given member, nor in members oriented with their weak axis normal to the plane of bending. However, in the region of the last hinge to form, and in regions not adjacent to a plastic hinge, the maximum distance between points of lateral support shall be such as to satisfy the requirements of Equations (F1-5), (F1-6), or (F1-7), as well as Equations (H1-1) and (H1-2). For this case, the values of f_a and f_b shall be computed from the moment and axial force at factored loading, divided by the applicable load factor.

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

N10. FABRICATION

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

1. The use of sheared edges shall be avoided in locations subject to plastic hinge rotation at factored loading. If used, they shall be finished smooth by grinding, chipping or planing.
2. In locations subject to plastic hinge rotation at factored loading, holes for rivets or bolts in the tension area shall be sub-punched and reamed or drilled full size.

APPENDIX B

DESIGN REQUIREMENTS

B5. LOCAL BUCKLING

2. Slender Compression Elements

Axially loaded members and flexural members containing elements subject to compression which have a width-thickness ratio in excess of the applicable noncompact value, as stipulated in Sect. B5.1 shall be proportioned according to this Appendix.

a. Unstiffened Compression Elements

The allowable stress of unstiffened compression elements whose width-thickness ratio exceeds the applicable noncompact value as stipulated in Sect. B5.1 shall be subject to a reduction factor Q_s. The value of Q_s shall be determined by Equations (A-B5-1) through (A-B5-6), as applicable, where b is the width of the unstiffened element as defined in Sect. B5.1. When such elements comprise the compression flange of a flexural member, the maximum allowable bending stress shall not exceed 0.60 F_yQ_s nor the applicable value as provided in Sect. F1.3. The allowable stress of axially loaded compression members shall be modified by the appropriate reduction factor Q, as provided in paragraph c.

For single angles:

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

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

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

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

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

When $95.0/\sqrt{F_y/k_c} < b/t < 195/\sqrt{F_y/k_c}$

$$Q_s = 1.293 - 0.00309(b/t)\sqrt{F_y/k_c} \tag{A-B5-3}$$

When $b/t > 195/\sqrt{F_y/k_c}$

$$Q_s = 26{,}200\ k_c/[F_y(b/t)^2] \tag{A-B5-4}$$

For stems of tees:

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

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

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

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

where

b = width of unstiffened compression element as defined in Sect. B5.1

t = thickness of unstiffened element, in.

F_y = specified minimum yield stress, ksi

$$k_c = \frac{4.05}{(h/t)^{.46}} \text{ if } h/t > 70, \text{ otherwise } k_c = 1.0.$$

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

b. Stiffened Compression Elements

When the width-thickness ratio of uniformly compressed stiffened elements (except perforated cover plates) exceeds the noncompact limit stipulated in Sect. B5.1, a reduced effective width b_e shall be used in computing the design properties of the section containing the element, except that the ratio b_e/t need not be taken as less than the applicable value permitted in Sect. B5.1.

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

$$b_e = \frac{253t}{\sqrt{f}}\left[1 - \frac{50.3}{(b/t)\sqrt{f}}\right] \leq b \tag{A-B5-7}$$

For other uniformly compressed elements:

$$b_e = \frac{253t}{\sqrt{f}}\left[1 - \frac{44.3}{(b/t)\sqrt{f}}\right] \leq b \tag{A-B5-8}$$

where

b = actual width of a stiffened compression element, as defined in Sect. B5.1, in.

b_e = reduced width, in.

t = element thickness, in.

f = computed compressive stress (axial plus bending stresses) in the stiffened elements, based on the design properties as specified in Appendix B5.2, ksi. If unstiffened elements are included in the

Table A-B5.1
Limiting Proportions for Channels and Tees

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

total cross section, f for the stiffened element must be such that the maximum compressive stress in the unstiffened element does not exceed $F_a Q_s$ or $F_b Q_s$, as applicable.

When the allowable stresses are increased due to wind or seismic loading in accordance with the provisions of Sect. A5.2, the effective width b_e shall be determined on the basis of 0.75 times the stress caused by wind or seismic loading acting alone or in combination with the design dead and live loading.

For axially loaded circular sections:

Members with diameter-to-thickness ratios D/t greater than $3{,}300/F_y$, but having a diameter-to-thickness ratio of less than $13{,}000/F_y$, shall not exceed the smaller value determined by Sect. E2 nor

$$F_a = \frac{662}{D/t} + 0.40F_y \tag{A-B5-9}$$

where

D = outside diameter, in.

t = wall thickness, in.

c. Design Properties

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

In computing the moment of inertia and section modulus of flexural members, the effective width of uniformly compressed stiffened elements, as determined in Appendix B5.2b, shall be used in determining effective cross-sectional properties.

For stiffened elements of the cross section

$$Q_a = \frac{\text{effective area}}{\text{actual area}} \tag{A-B5-10}$$

For unstiffened elements of the cross section, Q_s is determined from Appendix B5.2a.

For axially loaded compression members the gross cross-sectional area and the radius of gyration r shall be computed on the basis of the actual cross section.

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

$$F_a = \frac{Q\left[1 - \frac{(Kl/r)^2}{2C_c'^2}\right]F_y}{\frac{5}{3} + \frac{3(Kl/r)}{8C_c'} - \frac{(Kl/r)^3}{8C_c'^3}} \tag{A-B5-11}$$

when Kl/r is less than C_c', where

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

and

$$Q = Q_s Q_a$$

a. Cross sections composed entirely of unstiffened elements,

$$Q = Q_s \text{ i.e. } (Q_a = 1.0)$$

b. Cross sections composed entirely of stiffened elements,

$$Q = Q_a \text{ i.e. } (Q_s = 1.0)$$

c. Cross sections composed of both stiffened and unstiffened elements,

$$Q = Q_s Q_a$$

When Kl/r exceeds C_c':

$$F_a = \frac{12\pi^2 E}{23(Kl/r)^2} \qquad \text{(A-B5-12)}$$

d. Combined Axial and Flexural Stress

In applying the provisions of Chapter H to members subject to combined axial and flexural stress and containing stiffened elements whose width-thickness ratio exceeds the applicable noncompact limit given in Sect. B5.1, the stresses F_a, f_{bx} and f_{by} shall be calculated on the basis of the section properties as provided in Appendix B5.2c, as applicable. The allowable bending stress F_b for members containing unstiffened elements whose width-thickness ratio exceeds the noncompact limit given in Sect. B5.1 shall be the smaller value, $0.60F_yQ_s$ or that provided in Sect. F1.3. The term $f_a/0.60F_y$ in Equations (H1-2) and (A-F7-13) shall be replaced by $f_a/0.60F_yQ$.

APPENDIX F

BEAMS AND OTHER FLEXURAL MEMBERS

F7. WEB-TAPERED MEMBERS

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

1. General Requirements

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

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

$$d = d_o \left(1 + \gamma \frac{z}{L}\right) \qquad \text{(A-F7-1)}$$

where

d_o = depth at smaller end of member, in.
d_L = depth at larger end of member, in.
γ = $(d_L - d_o)/d_o \leq$ the smaller of $0.268(L/d_o)$ or 6.0
z = distance from the smaller end of member, in.
L = unbraced length of member measured between the center of gravity of the bracing members, in.

2. Allowable Tensile Stress

The allowable tensile stress of tapered tension members shall be determined in accordance with Sect. D1.

3. Allowable Compressive Stress

On the gross section of axially loaded tapered compression members, the allowable compressive stress, in kips per sq. in., shall not exceed the following:

When the effective slenderness ratio S is less than C_c:

$$F_{a\gamma} = \frac{\left(1.0 - \dfrac{S^2}{2C_c^2}\right) F_y}{\dfrac{5}{3} + \dfrac{3S}{8C_c} - \dfrac{S^3}{8C_c^3}} \qquad \text{(A-F7-2)}$$

When the effective slenderness ratio S exceeds C_c:

$$F_{a\gamma} = \frac{12\pi^2 E}{23S^2} \tag{A-F7-3}$$

where

S = Kl/r_{oy} for weak axis bending and $K_\gamma l/r_{ox}$ for strong axis bending
K = effective length factor for a prismatic member
K_γ = effective length factor for a tapered member as determined by an analysis*
l = actual unbraced length of member, in.
r_{ox} = strong axis radius of gyration at the smaller end of a tapered member, in.
r_{oy} = weak axis radius of gyration at the smaller end of a tapered member, in.

4. Allowable Flexural Stress**

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

$$F_{b\gamma} = \frac{2}{3}\left[1.0 - \frac{F_y}{6B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2}}\right] F_y \le 0.60F_y \tag{A-F7-4}$$

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

$$F_{b\gamma} = B\sqrt{F_{s\gamma}^2 + F_{w\gamma}^2} \tag{A-F7-5}$$

In the above equations,

$$F_{s\gamma} = \frac{12 \times 10^3}{h_s L d_o / A_f} \tag{A-F7-6}$$

$$F_{w\gamma} = \frac{170 \times 10^3}{(h_w L / r_{To})^2} \tag{A-F7-7}$$

where

h_s = factor equal to $1.0 + 0.0230\gamma\sqrt{Ld_o/A_f}$
h_w = factor equal to $1.0 + 0.00385\gamma\sqrt{L/r_{To}}$
r_{To} = radius of gyration of a section at the smaller end, considering only the compression flange plus ⅓ of the compression web area, taken about an axis in the plane of the web, in.
A_f = area of the compression flange, in.^{2}

and where B is determined as follows:

a. When the maximum moment M_2 in three adjacent segments of approximately equal unbraced length is located within the central segment and

---

*See Commentary Appendix F7.3.

**See Commentary Appendix F7.4.

M_1 is the larger moment at one end of the three-segment portion of a member:*

$$B = 1.0 + 0.37\left(1.0 + \frac{M_1}{M_2}\right) + 0.50\gamma\left(1.0 + \frac{M_1}{M_2}\right) \geq 1.0 \qquad \text{(A-F7-8)}$$

b. When the largest computed bending stress f_{b2} occurs at the larger end of two adjacent segments of approximately equal unbraced lengths and f_{b1} is the computed bending stress at the smaller end of the two-segment portion of a member:*

$$B = 1.0 + 0.58\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) - 0.70\gamma\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) \geq 1.0 \qquad \text{(A-F7-9)}$$

c. When the largest computed bending stress f_{b2} occurs at the smaller end of two adjacent segments of approximately equal unbraced length and f_{b1} is the computed bending stress at the larger end of the two-segment portion of a member:**

$$B = 1.0 + 0.55\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) + 2.20\gamma\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) \geq 1.0 \qquad \text{(A-F7-10)}$$

In the foregoing, $\gamma = (d_L - d_o)/d_o$ is calculated for the unbraced length containing the maximum computed bending stress.

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

$$B = \frac{1.75}{1.0 + 0.25\sqrt{\gamma}} \qquad \text{(A-F7-11)}$$

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

5. Allowable Shear

The allowable shear stress of tapered flexural members shall be in accordance with Sect. F4.

6. Combined Flexure and Axial Force

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

$$\left(\frac{f_{ao}}{F_{a\gamma}}\right) + \frac{C'_m}{\left(1 - \frac{f_{ao}}{F'_{e\gamma}}\right)}\left(\frac{f_{bl}}{F_{b\gamma}}\right) \leq 1.0 \qquad \text{(A-F7-12)}$$

* M_1/M_2 is considered as negative when producing single curvature. In the rare case where M_1/M_2 is positive, it is recommended it be taken as zero.

** f_{b1}/f_{b2} is considered as negative when producing single curvature. If a point of contraflexure occurs in one of two adjacent unbraced segments, f_{b1}/f_{b2} is considered as positive. The ratio $f_{b1}/f_{b2} \neq 0$.

and

$$\frac{f_a}{0.60F_y} + \frac{f_b}{F_{b\gamma}} \leq 1.0 \tag{A-F7-13}$$

When $f_{ao}/F_{a\gamma} \leq 0.15$, Equation (A-F7-14) is permitted in lieu of Equations (A-F7-12) and (A-F7-13).

$$\left(\frac{f_{ao}}{F_{a\gamma}}\right) + \left(\frac{f_{bl}}{F_{b\gamma}}\right) \leq 1.0 \tag{A-F7-14}$$

where

$F_{a\gamma}$ = axial compressive stress permitted in the absence of bending moment, ksi

$F_{b\gamma}$ = bending stress permitted in the absence of axial force, ksi

$F'_{e\gamma}$ = Euler stress divided by factor of safety, ksi, equal to

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

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

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

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

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

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

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

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

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

When $Kl/r \geq C_c$ and combined stresses are checked incrementally along the length, f_{ao} may be replaced by f_a, and f_{bl} may be replaced by f_b, in Equations (A-F7-12) and (A-F7-14).

APPENDIX K

STRENGTH DESIGN CONSIDERATIONS

K4. FATIGUE

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

Fatigue, as used in this Specification, is defined as the damage that may result in fracture after a sufficient number of fluctuations of stress. Stress range is defined as the magnitude of these fluctuations. In the case of a stress reversal, the stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the sum of maximum shearing stresses of opposite direction at a given point, resulting from differing arrangement of live load.

1. Loading Conditions; Type and Location of Material

In the design of members and connections subject to repeated variation of live load, consideration shall be given to the number of stress cycles, the expected range of stress and the type and location of member or detail.

Loading conditions shall be classified according to Table A-K4.1.

The type and location of material shall be categorized according to Table A-K4.2.

2. Allowable Stress Range

The maximum stress shall not exceed the basic allowable stress provided in Chapters A through M of this Specification and the maximum range of stress shall not exceed that given in Table A-K4. 3.

TABLE A-K4.1
Number of Loading Cycles

Loading Condition	From	To
1	20,000[a]	100,000[b]
2	100,000	500,000[c]
3	500,000	2,000,000[d]
4	Over 2,000,000	

[a]Approximately equivalent to two applications every day for 25 years.
[b]Approximately equivalent to 10 applications every day for 25 years.
[c]Approximately equivalent to 50 applications every day for 25 years.
[d]Approximately equivalent to 200 applications every day for 25 years

3. Tensile Fatigue

When subject to tensile fatigue loading, the tensile stress in A325 or A490 bolts due to the combined applied load and prying forces shall not exceed the following values, and the prying force shall not exceed 60% of the externally applied load.

Number of Cycles	A325	A490
Not more than 20,000	44	54
From 20,000 to 500,000	40	49
More than 500,000	31	38

Bolts must be tensioned to the requirements of Table J3.7.

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

TABLE A-K4.2
Stress Category Classifications

General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illustrative Example Nos. (see Fig. A-K4.1)[b]
Plain Material	Base metal with rolled or cleaned surface. Flame-cut edges with ANSI smoothness of 1,000 or less	T or Rev.	A	1,2
Built-up Members	Base metal in members without attachments, built-up plates or shapes connected by continuous full-penetration groove welds or by continuous fillet welds parallel to the direction of applied stress	T or Rev.	B	3,4,5,6
	Base metal in members without attachments, built-up plates, or shapes connected by full-penetration groove welds with backing bars not removed, or by partial-penetration groove welds parallel to the direction of applied stress	T or Rev.	B′	3,4,5,6
	Base metal at toe welds on girder webs or flanges adjacent to welded transverse stiffeners	T or Rev.	C	7
	Base metal at ends of partial length welded cover plates narrower than the flange having square or tapered ends, with or without welds across the ends or wider than flange with welds across the ends			
	Flange thickness ≤ 0.8 in.	T or Rev.	E	5
	Flange thickness > 0.8 in.	T or Rev.	E′	5
	Base metal at end of partial length welded cover plates wider than the flange without welds across the ends		E′	5

[a] "T" signifies range in tensile stress only; "Rev." signifies a range involving reversal of tensile or compressive stress; "S" signifies range in shear, including shear stress reversal.
[b] These examples are provided as guidelines and are not intended to exclude other reasonably similar situations.
[c] Allowable fatigue stress range for transverse partial-penetration and transverse fillet welds is a function of the effective throat, depth of penetration and plate thickness. See Frank and Fisher (1979).

TABLE A-K4.2 (cont'd)
Type and Location of Material

General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illustrative Example Nos. (see Fig. A-K4.1)[b]
Groove Welds	Base metal and weld metal at full-penetration groove welded splices of parts of similar cross section ground flush, with grinding in the direction of applied stress and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of 9.25.2 or 9.25.3 of AWS D1.1	T or Rev.	B	10,11
	Base metal and weld metal at full-penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2½ with grinding in the direction of applied stress, and with weld soundness established by radiographic or ultrasonic inspection in accordance with the requirements of 9.25.2 or 9.25.3 of AWS D1.1			
	A514 base metal	T or Rev.	B′	12,13
	Other base metals	T or Rev.	B	12,13
	Base metal and weld metal at full-penetration groove welded splices, with or without transitions having slopes no greater than 1 to 2½ when reinforcement is not removed but weld soundness is established by radiographic or ultrasonic inspection in accordance with requirements of 9.25.2 or 9.25.3 of AWS D1.1	T or Rev.	C	10,11,12, 13
Partial-Penetration Groove Welds	Weld metal of partial-penetration transverse groove welds, based on effective throat area of the weld or welds	T or Rev.	F[c]	16

TABLE A-K4.2 (cont'd)
Type and Location of Material

General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illustrative Example Nos. (see Fig. A-K4.1)[b]
Fillet-welded Connections	Base metal at intermittent fillet welds	T or Rev.	E	
	Base metal at junction of axially loaded members with fillet-welded end connections. Welds shall be disposed about the axis of the member so as to balance weld stresses $b \leq 1$ in. $b > 1$ in.	 T or Rev. T or Rev.	 E E′	 17,18 17,18
	Base metal at members connected with transverse fillet welds $b \leq$ ½ in. $b >$ ½ in.	T or Rev.	C See Note c	20,21
Fillet Welds	Weld metal of continuous or intermittent longitudinal or transverse fillet welds	S	F[c]	15,17,18 20,21
Plug or Slot Welds	Base metal at plug or slot welds	T or Rev.	E	27
	Shear on plug or slot welds	S	F	27
Mechanically Fastened Connections	Base metal at gross section of high-strength bolted slip-critical connections, except axially loaded joints which induce out-of-plane bending in connected material	T or Rev.	B	8
	Base metal at net section of other mechanically fastened joints	T or Rev.	D	8,9
	Base metal at net section of fully tensioned high-strength, bolted-bearing connections	T or Rev.	B	8,9

TABLE A-K4.2 (cont'd)
Type and Location of Material

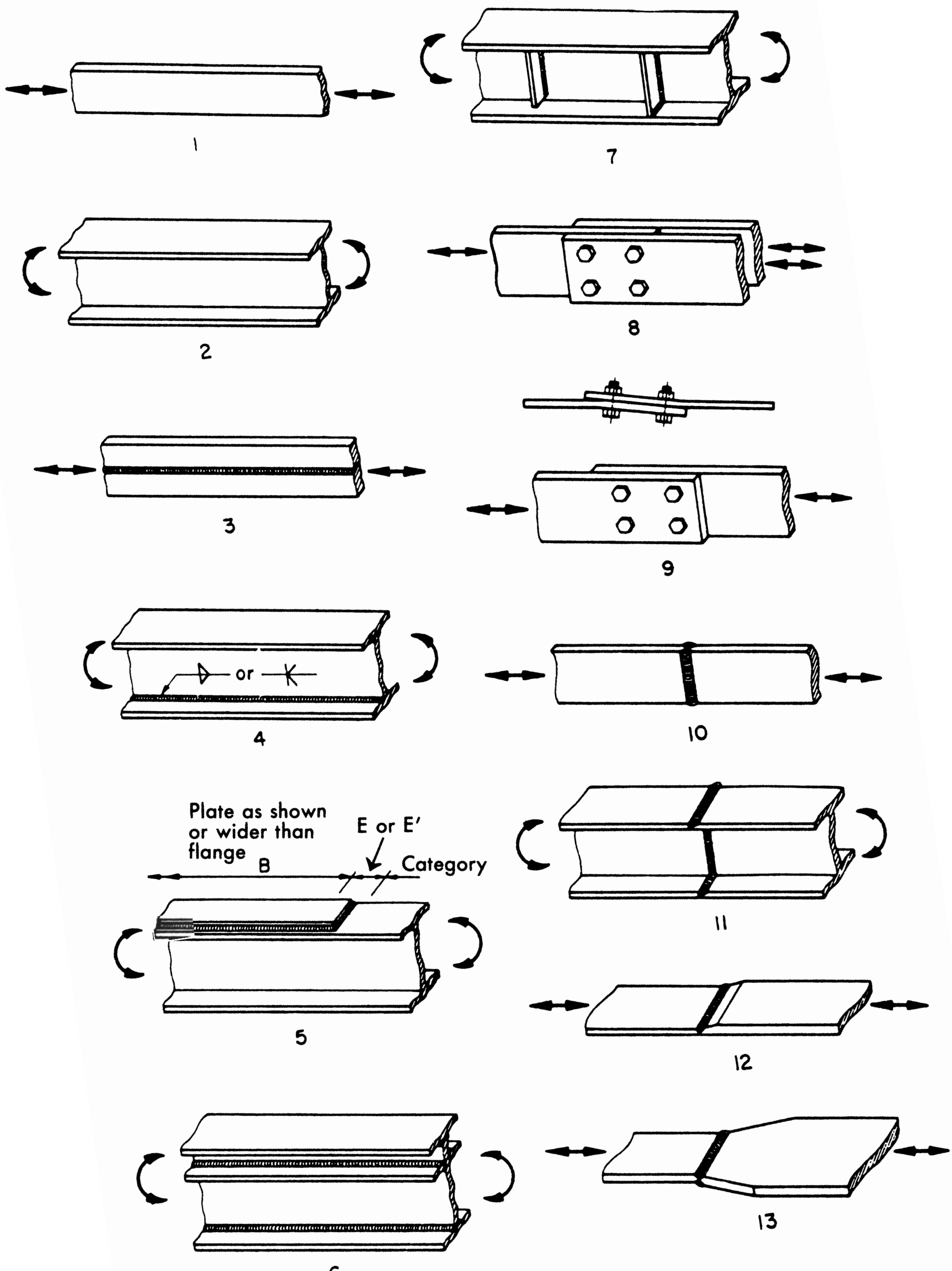
General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illustrative Example Nos. (see Fig. A-K4.1)[b]
Attachments	Base metal at details attached by full-penetration groove welds subject to longitudinal and/or transverse loading when the detail embodies a transition radius R with the weld termination ground smooth and for transverse loading, the weld soundness established by radiographic or ultrasonic inspection in accordance with 9.25.2 or 9.25.3 of AWS D1.1			
	Longitudinal loading			
	$R > 24$ in.	T or Rev.	B	14
	24 in. $> R >$ 6 in.	T or Rev.	C	14
	6 in. $> R >$ 2 in.	T or Rev.	D	14
	2 in. $> R$	T or Rev.	E	14
	Detail base metal for transverse loading: equal thickness and reinforcement removed			
	$R > 24$ in.	T or Rev.	B	14
	24 in. $> R >$ 6 in.	T or Rev.	C	14
	6 in. $> R >$ 2 in.	T or Rev.	D	14
	2 in. $> R$	T or Rev.	E	14,15
	Detail base metal for transverse loading: equal thickness and reinforcement not removed			
	$R > 24$ in.	T or Rev.	C	14
	24 in. $> R >$ 6 in.	T or Rev.	C	14
	6 in. $> R >$ 2 in.	T or Rev.	D	14
	2 in. $> R$	T or Rev.	E	14,15

TABLE A-K4.2 (cont'd)
Type and Location of Material

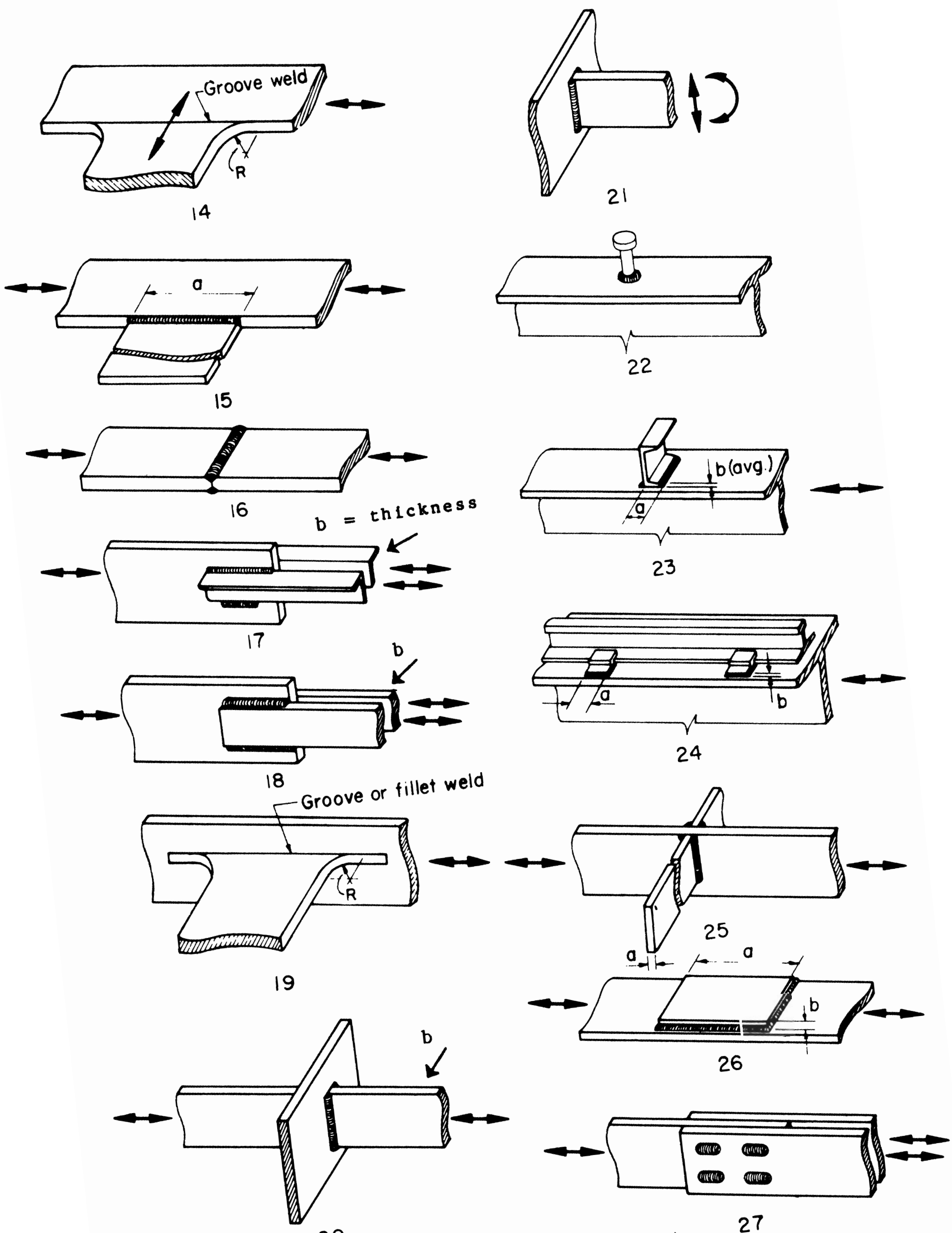
General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illustrative Example Nos. (see Fig. A-K4.1)[b]
Attachments (cont'd)	Detail base metal for transverse loading: unequal thickness and reinforcement removed			
	$R > 2$ in.	T or Rev.	D	14
	2 in. $> R$	T or Rev.	E	14,15
	Detail base metal for transverse loading: unequal thickness and reinforcement not removed			
	all R	T or Rev.	E	14,15
	Detail base metal for transverse loading			
	$R > 6$ in.	T or Rev.	C	19
	6 in. $> R >$ 2 in.	T or Rev.	D	19
	2 in. $> R$	T or Rev.	E	19
	Base metal at detail attached by full-penetration groove welds subject to longitudinal loading			
	$2 < a < 12b$ or 4 in.	T or Rev.	D	15
	$a > 12b$ or 4 in. when $b \leq 1$ in.	T or Rev.	E	15
	$a > 12b$ or 4 in. when $b > 1$ in.	T or Rev.	E′	15
	Base metal at detail attached by fillet welds or partial-penetration groove welds subject to longitudinal loading			
	$a < 2$ in.	T or Rev.	C	15,23,24, 25,26
	2 in. $< a < 12b$ or 4 in.	T or Rev.	D	15,23, 24,26
	$a > 12b$ or 4 in. when $b \leq 1$ in.	T or Rev.	E	15,23, 24,26
	$a > 12b$ or 4 in. when $b > 1$ in.	T or Rev.	E′	15,23, 24,26

TABLE A-K4.2 (cont'd)
Type and Location of Material

General Condition	Situation	Kind of Stress[a]	Stress Category (see Table A-K4.3)	Illustrative Example Nos. (see Fig. A-K4.1)[b]
Attachments (cont'd)	Base metal attached by fillet welds or partial-penetration groove welds subjected to longitudinal loading when the weld termination embodies a transition radius with the weld termination ground smooth: $R > 2$ in. $R \leq 2$ in.	 T or Rev. T or Rev.	 D E	 19 19
	Fillet-welded attachments where the weld termination embodies a transition radius, weld termination ground smooth, and main material subject to longitudinal loading: Detail base metal for transverse loading: $R > 2$ in. $R < 2$ in.	 T or Rev. T or Rev.	 D E	 19 19
	Base metal at stud-type shear connector attached by fillet weld or automatic end weld	T or Rev.	C	22
	Shear stress on nominal area of stud-type shear connectors	S	F	



Illustrative examples



Illustrative examples (cont.)

TABLE A-K4.3
Allowable Stress Range, Ksi

Category (from Table A-K4.2)	Loading Condition 1	Loading Condition 2	Loading Condition 3	Loading Condition 4
A	63	37	24	24
B	49	29	18	16
B′	39	23	15	12
C	35	21	13	10[a]
D	28	16	10	7
E	22	13	8	5
E′	16	9	6	3
F	15	12	9	8

[a] Flexural stress range of 12 ksi permitted at toe of stiffener welds on flanges

NUMERICAL VALUES

TABLE 1
Allowable Stress as a Function of F_y

F_y (ksi)	Allowable Stress (ksi)					
	$0.40F_y$ [b,e]	$0.45F_y$ [a]	$0.60F_y$ [a,c]	$0.66F_y$ [c]	$0.75F_y$ [c]	$0.90F_y$ [d]
33	13.2	14.9	19.8	21.8	24.8	29.7
35	14.0	15.8	21.0	23.1	26.3	31.5
36	14.5	16.2	21.6	23.8	27.0	32.4
40	16.0	18.0	24.0	26.4	30.0	36.0
42	16.8	18.9	25.2	27.7	31.5	37.8
45	18.0	20.3	27.0	29.7	33.8	40.5
46	18.4	20.7	27.6	30.4	34.5	41.4
50	20.0	22.5	30.0	33.0	37.5	45.0
55	22.0	24.8	33.0	36.3	41.3	49.5
60	24.0	27.0	36.0	39.6	45.0	54.0
65	26.0	29.3	39.0	42.9	48.8	58.5
70	28.0	31.5	42.0			63.0
90	36.0	40.5	54.0			81.0
100	40.0	45.0	60.0			90.0

[a]See Sect. D1, D3 Tension
[b]See Sect. D3, F4, K1 Shear
[c]See Sect. F1, F2 Bending
[d]See Sect. J8 Bearing
[e]See Sect. G3 Shear in Plate Girders

TABLE 2
Allowable Stresses as a Function of F_u

Item	ASTM Designation	F_y (ksi)	F_u (ksi)	Allowable Stress (ksi): Connected Part of Designated Steel: Tension $0.5F_u$[a]	Allowable Stress (ksi): Connected Part of Designated Steel: Bearing $1.2F_u$[b]	Allowable Stress (ksi): Bolt or Threaded Part of Designated Steel: Tension $0.33F_u$[c]	Allowable Stress (ksi): Bolt or Threaded Part of Designated Steel: Shear $0.17F_u$[d]	Allowable Stress (ksi): Bolt or Threaded Part of Designated Steel: Shear $0.22F_u$[e]
Shapes, Plates, Bars, Sheet and Tubing, or Threaded Parts	A36	36	58—80	29.0	69.6	19.1	9.9	12.8
	A53	35	60	30.0	72.0	—	—	—
	[A242, A441, A588]	50	70	35.0	84.0	23.1	11.9	15.4
		46	67	33.5	80.4	22.1	11.4	14.7
		42	63	31.5	75.6	20.8	10.7	13.9
		40[f]	60	30.0	72.0	19.8	10.2	13.2
	A500	33/39[g]	45	22.5	54.0	—	—	—
		42/46[g]	58	29.0	69.6	—	—	—
		46/50[g]	62	31.0	74.4	—	—	—
	A501	36	58	29.0	69.6	—	—	—
	A529	42	60—85	30.0	72.0	19.8	10.2	13.2
	A570	40	55	27.5	66.0	—	—	—
		42	58	29.0	69.6	—	-	-
	A572	42	60	30.0	72.0	19.8	10.2	13.2
		50	65	32.5	78.0	21.5	11.1	14.3
		60	75	37.5	90.0	24.8	12.8	16.5
		65	80	40.0	96.0	26.4	13.6	17.6
	A514	100	110—130	55.0	132	36.3	18.7	24.2
		90	100—130	50.0	120	33.0	17.0	22.0
	A606	45	65	32.5	78.0	—	—	—
		50	70	35.0	84.0	—	—	—
	A607	45	60	30.0	72.0	—	—	—
		50	65	32.5	78.0	—	—	—
		55	70	35.0	84.0	—	—	—
		60	75	37.5	90.0	—	—	—
		65	80	40.0	96.0	—	—	—
		70	85	42.5	102	—	—	—
	A618	50	70	35.0	84.0	—	—	—
		50	65	32.5	78.0	—	—	—
	A852	70	90—110	45.0	108	—	—	—
Bolts	A449	92	120	—	—	39.6	20.4	26.4
		81	105	—	—	34.7	17.9	23.1
		58	90	—	—	29.7	15.3	19.8

[a] On effective net area, see Sects. D1, J4.
[b] Produced by fastener in shear, see Sect. J3.7. Note that smaller maximum design bearing stresses, as a function of hole spacing, may be required by Sects. J3.8 and J3.9.
[c] On nominal body area, see Table J3.2.
[d] Threads not excluded from shear plane, see Table J3.2.
[e] Threads excluded from shear plane, see Table J3.2.
[f] For A441 material only.
[g] Smaller value for circular shapes, larger for square or rectangular shapes.
Note: For dimensional and size limitations, see the appropriate ASTM Specification.

TABLE 3
VALUES OF C_a

For Determining Allowable Stress When $Kl/r \le C_c$
for Steel of Any Yield Stress (by Eq. $F_a = C_aF_y$)[a]

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

[a]When ratios exceed the noncompact section limits of Sect. B5.1, use $\frac{Kl/r}{C_c'}$ in lieu of $\frac{Kl/r}{C_c}$ values and equation $F_a = C_aQ_aQ_sF_y$ (Appendix Sect. B5).

All grades of steel

TABLE 4
VALUES OF C_c

For Use with Equations (E2-1) and (E2-2) and in Table 3

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

TABLE 5
Slenderness Ratios of Elements as a Function of F_y

Specification Section and Ratios	F_y (ksi)					
	36	42	46	50	60	65
Table B5.1						
$65/\sqrt{F_y}$	10.8	10.0	9.6	9.2	8.4	8.1
$190/\sqrt{F_y}$	31.7	29.3	28.0	26.9	24.5	23.6
$640/\sqrt{F_y}$	106.7	98.8	94.4	90.5	82.6	79.4
$257/\sqrt{F_y}$	42.8	39.7	37.9	36.3	33.2	31.9
Sect. F1.2						
$\sqrt{\dfrac{102 \times 10^3 C_b}{F_y}}$	$53\sqrt{C_b}$	$49\sqrt{C_b}$	$47\sqrt{C_b}$	$45\sqrt{C_b}$	$41\sqrt{C_b}$	$40\sqrt{C_b}$
$\sqrt{\dfrac{510 \times 10^3 C_b}{F_y}}$	$119\sqrt{C_b}$	$110\sqrt{C_b}$	$105\sqrt{C_b}$	$101\sqrt{C_b}$	$92\sqrt{C_b}$	$89\sqrt{C_b}$
Table B5.1						
$76/\sqrt{F_y}$	12.7	11.7	11.2	10.7	9.8	9.4
$95/\sqrt{F_y}$	15.8	14.7	14.0	13.4	12.3	11.8
$127/\sqrt{F_y}$	21.2	19.6	18.7	18.0	16.4	15.8
Table B5.1						
$238/\sqrt{F_y}$	39.7	36.7	35.1	33.7	30.7	29.5
$317/\sqrt{F_y}$	52.8	48.9	46.7	44.8	40.9	39.3
$253/\sqrt{F_y}$	42.2	39.0	37.3	35.8	32.7	31.4
Table B5.1—Appendix B5.2b						
$3300/F_y$	91.7	78.6	71.7	66.0	55.0	50.8
$13000/F_y$	361	310	283	260	217	200
Sect. G1						
$\dfrac{14000}{\sqrt{F_y(F_y + 16.5)}}$	322	282	261	243	207	192
$2000/\sqrt{F_y}$	333	309	295	283	258	248

TABLE 6
Values of C_b

For Use in Equations (F1-6), (F1-7) and (F1-8)

$\frac{M_1}{M_2}$	C_b	$\frac{M_1}{M_2}$	C_b	$\frac{M_1}{M_2}$	C_b
−1.00	1.00	−0.45	1.34	0.10	1.86
−0.95	1.02	−0.40	1.38	0.15	1.91
−0.90	1.05	−0.35	1.42	0.20	1.97
−0.85	1.07	−0.30	1.46	0.25	2.03
−0.80	1.10	−0.25	1.51	0.30	2.09
−0.75	1.13	−0.20	1.55	0.35	2.15
−0.70	1.16	−0.15	1.60	0.40	2.22
−0.65	1.19	−0.10	1.65	0.45	2.28
−0.60	1.23	−0.05	1.70	≥0.47	2.30
−0.55	1.26	0	1.75		
−0.50	1.30	0.05	1.80		

Note 1: $C_b = 1.75 + 1.05(M_1/M_2) + 0.3\,(M_1/M_2)^2 \leq 2.3$
Note 2: M_1/M_2 positive for reverse curvature and negative for single curvature.

TABLE 7
Values of C_m

For Use in Equation (H1-1)

$\frac{M_1}{M_2}$	C_m	$\frac{M_1}{M_2}$	C_m	$\frac{M_1}{M_2}$	C_m
−1.00	1.00	−0.45	0.78	0.10	0.56
−0.95	0.98	−0.40	0.76	0.15	0.54
−0.90	0.96	−0.35	0.74	0.20	0.52
−0.85	0.94	−0.30	0.72	0.25	0.50
−0.80	0.92	−0.25	0.70	0.30	0.48
−0.75	0.90	−0.20	0.68	0.35	0.46
−0.70	0.88	−0.15	0.66	0.40	0.44
−0.65	0.86	−0.10	0.64	0.45	0.42
−0.60	0.84	−0.05	0.62	0.50	0.40
				0.60	0.36
−0.55	0.82	0	0.60	0.80	0.28
−0.50	0.80	0.05	0.58	1.00	0.20

Note 1: $C_m = 0.6 - 0.4(M_1/M_2)$
Note 2: M_1/M_2 is positive for reverse curvature and negative for single curvature.

TABLE 8
Values of F'_e
For Use in Equation (H1-1), for Steel of Any Yield Stress

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

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

Commentary

ON THE SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS— ALLOWABLE STRESS DESIGN AND PLASTIC DESIGN

(June 1, 1989)

INTRODUCTION

This Commentary provides information on the basis and limitations of various provisions of the Specification, so that designers, fabricators and erectors (users) can make more efficient use of the Specification. The Commentary and Specification, termed as documents, do not attempt to anticipate and/or set forth all the questions or possible problems that may be encountered, or situations in which special consideration and engineering judgment should be exercised in using and applying the documents. Such a recitation could not be made complete and would make the documents unduly lengthy and cumbersome.

Warning is given that AISC assumes the users of its documents are competent in their fields of endeavor and are informed on current developments and findings related to their fields.

CHAPTER A

GENERAL PROVISIONS

A2. LIMITS OF APPLICABILITY

2. Types of Construction

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

For better clarity, provisions covering tier buildings of Type 2 construction designed for wind loading were reworded in the 1969 Specification, but without change in intent. Justification for these provisions has been discussed by Disque (1964 and 1975) and Ackroyd (1987).

A3. MATERIAL

1. Structural Steel

a. ASTM Designations

The grades of structural steel approved for use under the Specification, covered by ASTM standard specifications, extend to a yield stress of 100 ksi. Some of these ASTM standards specify a minimum yield point, while others specify a minimum yield strength. The term "yield stress" is used in the Specification as a generic term to denote either the yield point or the yield strength. When requested, the fabricator must provide an affidavit that all steel specified has been provided in accordance with the plans and Specification.

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

Provisions of the Specification are based on providing a factor of safety against reaching yield stress in primary connected material at allowable loads. The direction parallel to the direction of rolling is the direction of principal interest in the design of steel structures. Hence, yield stress as determined by the standard tensile test is the principal mechanical property recognized in the selection of the steels approved for use under the Specification. It must be recognized that other mechanical and physical properties of rolled steel, such as anisotropy, ductility, notch toughness, formability, and corrosion resistance may also be important to the satisfactory performance of a structure. In such situations,the user of the Specification is advised to make use of reference material contained in the literature on the specific properties of concern and to specify supplementary material production or quality requirements as provided for in ASTM material specifications. One such situation, for example, is the design of highly restrained welded connections (AISC, 1973). Rolled steel is anisotropic, especially insofar as ductility is concerned; therefore weld contraction strains in the region of highly restrained welded connections may exceed the capabilities of the material if special attention is not given to material selection, details, workmanship and inspection. Another special situation is that of fracture control design for certain types of service conditions (Rolfe, 1977). The relatively warm temperatures of steel in buildings, the essentially static strain rates, the stress intensity and the number of cycles of full allowable stress make the probability of fracture in building structures extremely remote. Good details which incorporate joint geometry that avoids severe stress concentrations and good workmanship are generally the most effective means to provide fracture-resistant construction. However, for especially demanding service conditions, such as low temperatures with impact loading, the specification of steels with superior notch toughness should be specified.

c. Heavy Shapes

The web-to-flange intersection and the web center of heavy hot-rolled shapes, as well as the interior portions of heavy plates, may contain a coarser grain structure and/or lower toughness than other areas of these products. This is probably caused by ingot segregation, as well as somewhat less deformation during hot rolling, higher finishing temperature and a slower cooling rate after rolling. This characteristic is not detrimental to suitability for service as compression members or non-welded members. However when heavy sections are fabricated using full-penetration welds, tensile strains induced by weld shrinkage may result in cracking. For critical applications such as primary tension members, material should be produced to provide adequate toughness. Because of differences in the strain rate between the Charpy V-Notch (CVN) impact test and the strain rate experienced in actual structures, the CVN test is conducted at a temperature higher than the anticipated service temperature.

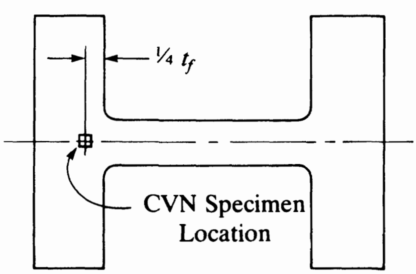


Fig. C-A3.1c Location from which charpy impact specimen shall be taken.

The toughness requirements of Sect. A3.1c are intended only to provide material of necessary toughness for ordinary service application. For unusual applications and/or low temperature service, more restrictive requirements and/or toughness requirements for other section sizes and thickness would be appropriate.

To minimize the potential for fracture, the notch toughness requirements of Sect. A3.1c must be used in conjunction with good design and fabrication procedures. Specific requirements are given in Sects. J1.7, J1.8, J2.6, J2.7 and M2.2.

4. Bolts, Washers and Nuts

The ASTM standard for A307 bolts covers two grades of fasteners. Either grade may be used under the Specification; however, Grade B is intended for pipe flange bolting. Grade A is used for structural applications.

6. Filler Metal and Flux for Welding

When specifying filler metal and/or flux by AWS designation, the applicable standard specifications should be carefully reviewed to assure a complete understanding of the electrode designation. This is necessary because the AWS designation systems are not consistent. For example, in the case of electrodes for shielded metal arc welding (AWS A5.1), the first two or three digits indicate the nominal tensile strength classification, (in ksi) of the weld material and the final two digits indicate the type coating. However, in the case of carbon steel electrodes for submerged arc welding (AWS A5.17), the first one or two digits times 10 indicates the nominal tensile strength classification, and the final digit or digits times − 10 indicates the testing temperature, in degrees F, for weld metal impact tests. In the case of low-alloy, steel-covered arc welding electrodes (AWS A5.5), certain portions of the designation indicate a requirement for stress relief, while others indicate no stress relief requirement.

A4. LOADS AND FORCES

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

2. Impact

A mass of the total moving load (wheel load) is used as the basis for impact loads on crane runway girders, because maximum impact load results when cranes travel while supporting lifted loads.

The increase in load, in recognition of random impacts, is not required to be applied to supporting columns, because the impact load effects (increase in eccentricities or increase in out-of-straightness) will not develop or will be negligible during the short duration of impact.

Association of Iron and Steel Engineers (AISE, 1979) gives more stringent requirements for crane girder and crane runway design.

3. Crane Runway Horizontal Forces

Minimum crane horizontal and longitudinal forces are provided in the Specification. Some cranes may require that the runway be designed for larger forces.

The magnitude and point of application of the crane stop forces should be provided by the owner. For additional information on runway forces, see AISE (1979).

A5. DESIGN BASIS

1. Allowable Stresses

The allowable stresses contained within the Specification are to be compared with stresses determined by analysis of the effects of design loads upon the structure. The factor of safety inherent in the allowable stresses provide for the uncertainties that are associated with typical simplifying assumptions and the use of nominal or average calculated stresses as the basis for manual methods of analysis. It is not intended that highly localized peak stresses that may be determined by sophisticated computer-aided methods of analysis, and which may be blunted by confined yielding, must be less than the stipulated allowable stresses. The exercise of engineering judgment is required.

In keeping with the inclusion of high strength low-alloy steels, the Specification recognizes high strength steel castings. Allowable stresses are expressed in terms of the specified minimum yield stress for castings.

CHAPTER B

DESIGN REQUIREMENTS

B3. EFFECTIVE NET AREA

Section B3 deals with the effect of shear lag. The inclusion of welded members acknowledges that shear lag is also a factor in determining the effective area of welded connections where the welds are so distributed as to directly connect some, but not all, of the elements of a tension member. However, since welds are applied to the unreduced cross-sectional area, the reduction coefficient U is applied to the gross area A_g. With this modification the values of U are the same as for similar shapes connected by bolts and rivets except that: (1) the provisions for members having only two fasteners per line in the direction of stress have no application to welded connections; and (2) tests (Kulak, Fisher and Struik, 1987) have shown that flat plates , or bars axially loaded in tension and connected only by longitudinal fillet welds, may fail prematurely by shear lag at their corners if the welds are separated by too great a distance. Therefore, the values of U are specified unless the member is designed on the basis of effective net area as discussed below.

As the length of a connection l is increased the intensity of shear lag is diminished. The concept can be expressed empirically as:

$$U = 1 - \bar{x}/l \tag{C-B1-1}$$

where:

$\bar{x}$ = the distance from the centroid of the shape profile to the shear plane of the connection, in.

l = length

Munse and Chesson have shown, using this expression to compute an effective net area, that with few exceptions, the estimated strength of some 1,000 test specimens correlated with observed test results with a scatterband of $\pm 10\%$ (Kulak, Fisher, and Struik, 1987; Munse and Chesson, 1963; Gaylord and Gaylord, 1972). For any given profile and connected elements, length l is dependent upon the number of fasteners or length of weld required to develop the given tensile force, and this in turn is dependent upon the mechanical properties of the member and the capacity of the fasteners or weld used. The values of U, given as the reduction coefficients in Sect. B3, are reasonable lower bounds for the profile types and connections described, based upon the use of the above expression.

The restriction that the net area shall in no case be considered as comprising more than 85% of the gross area is limited to relatively short fittings, such as splice plates, gusset plates or beam-to-column fittings.

B4. STABILITY

The stability of structures must be considered from the standpoint of the structures as a whole, including not only the compression members, but also the beams, bracing system and connections. The stability of individual elements must also be provided. Considerable attention has been given to this subject in the technical literature, and various methods of analysis are available to assure stability. The SSRC *Guide to Design Criteria for Metal Compression Members* (Galambos, 1988) devotes several chapters to the stability of different types of members considered as individual elements, and then considers the effects of individual elements on the stability of the structure as a whole.

B5. LOCAL BUCKLING

For the purposes of the ASD Specification, steel sections are divided into compact sections, noncompact sections and sections with slender compression elements.

When the width-thickness ratio of the compressed elements in a member does not exceed the noncompact section limit specified in Table B5.1, no reduction in allowable stress is necessary in order to prevent local buckling. Appendix B provides a design procedure for those infrequent situations where width-thickness ratios in excess of the limits given in Sect. B5.1 are involved.

Equations (A-B5-1), (A-B5-2), (A-B5-5) and (A-B5-6) are based upon the following expression for critical buckling stress σ_c for a plate supported against lateral deflection along one or both edges (Galambos, 1988), with or without torsional restraint along these edges and subject to in-plane compressive force:

$$\sigma_c = k_c \left[\frac{\pi^2 E \sqrt{\eta}}{12\,(1-\nu^2)(b/t)^2} \right] \qquad \text{(C-B5-1)}$$

where:

η = the ratio of the tangent modulus to the elastic modulus, E_t/E
ν = Poisson's ratio

The assumption of nothing more than knife-edge lateral support applied along one edge of the unstiffened element under a uniformly distributed stress (the most critical case) would give a value of $k_c = 0.425$. Some increase in this value is warranted because of the torsional restraint provided by the supporting element and because of the difference between b, as defined in Sect. B5.1, and the theoretical width b.

Equations (A-B5-3) and (A-B5-4) have been revised for this AISC ASD Specification. In the 1978 AISC Specification, these formulas assumed partial end restraint from the beam web in rolled shapes for compression flange stability. However, with more slender girder webs that may have already buckled, this beneficial effect is diminished and the previous Q factors have been reduced to account for this local buckling interaction. Research by Butler Manufacturing resulted in new provisions, given in Appendix B5.2, which are also reflected in Sects. B5.1 and G2 (Johnson, 1985).

In the interest of simplification, when $\sqrt{\eta} < 1.0$, a linear formula is substituted for the theoretical expression. Its agreement with the latter may be judged by the comparison shown in Fig. C-B5.1.

Equation (A-B5-5) recognizes that the torsional restraint characteristics of tees cut from rolled shapes might be of quite different proportions than those of tees formed by welding two plates together.

It has been shown that singly symmetrical members whose cross section consists of elements having large width-thickness ratios may fail by twisting under a smaller axial load than associated with general column failure (Chajes and Winter, 1965). Such is not generally the case with hot-rolled shapes. To guard against this type of failure, particularly when relatively thin-walled members are fabricated from plates, Table A-B5.1 in Appendix B places an upper limit on the proportions permissible for channels and tees.

With both edges parallel to the applied load supported against buckling, stiffened compression elements can support a load producing an average stress σ_c greater than that given in the above expression for critical plate buckling stress. This is true even when k_c is taken as 4.0, applicable to the case where both edges are simply supported, or a value between 4.0 and 6.97, applicable when some torsional restraint is also provided along these edges.

A better estimate of the compressive strength of stiffened elements, based upon an "effective width" concept was first proposed by von Karman, Sechler and Donnell (1932). This was later modified by Winter (1947) to provide a transition between very slender elements and stockier elements shown by tests to be fully effective.

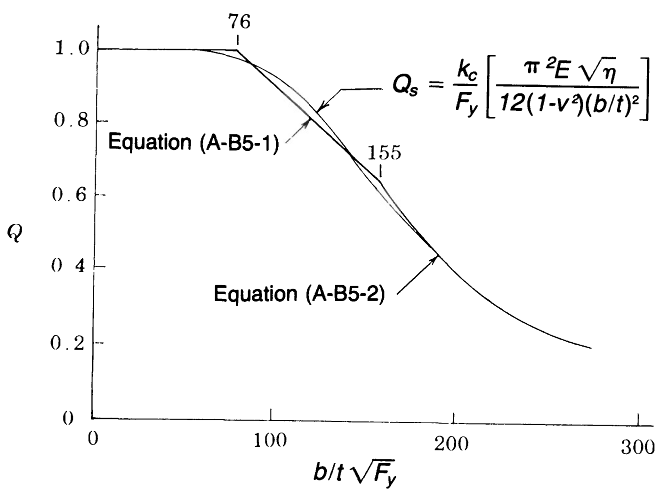


Fig. C-B5.1

As modified, the ratio of effective width to actual width increases as the level of compressive stress applied to a stiffened element in a member is decreased and takes the form

$$\frac{b_e}{t} = 1.9\sqrt{\frac{E}{f}}\left[1 - \frac{C}{(b/t)}\sqrt{\frac{E}{f}}\right] \qquad \text{(C-B5-2)}$$

where f is the level of uniformly distributed stress to which the element would be subjected based upon the design of the member, and C is an arbitrary constant based on test results (Winter, 1947).

Holding the effective width of a stiffened element to no greater value than that given by the limits provided in Sect. B5.1 is unnecessarily conservative when the maximum uniformly distributed design stress is substantially less than $0.60F_y$, or when the ratio b/t is considerably in excess of the limit given in Sect. B5.1.

For the case of square and rectangular box sections, the sides of which in their buckled condition afford negligible torsional restraint for one another along their corner edges, the value of C reflected in Equation (A-B5-7) is higher than for the other case, thereby providing a slightly more conservative evaluation of effective width. For cases where appreciable torsional restraint is provided, as for example the web of an I-shape column, the value of C implicit in Equation (A-B5-8) is decreased slightly. As in earlier editions of the AISC Specification, for such cases no reduction from actual width is required when the width-thickness ratio does not exceed $253/\sqrt{F_y}$, and for greater widths the effective width may be taken as equal to $253t/\sqrt{F_y}$. If the actual width-thickness ratio is substantially greater than $253/\sqrt{F_y}$, however, a larger effective width can be obtained using Equation (A-B5-8) rather than the earlier provisions.

In computing the section modulus of a member subject to bending, the area of stiffened elements parallel to the axis of bending and subject to compressive stress must be based upon their effective, rather than actual, width. In computing the effective area of a member subject to axial loading, the effective, rather than actual, area of all stiffened elements must be used. However, the radius of gyration of the *actual* cross section together with the form factor Q_a may be used to determine the allowable axial stress. If the cross section contains an unstiffened element, the allowable axial stress must be modified by the reduction factor Q_s.

The classical theory of longitudinally compressed cylinders overestimates the actual buckling strength, often by 200% or more. Inevitable imperfections of shape and the axiality of load are responsible for the reduction in actual strength below theoretical strength. The limits of B5.1 are based upon test evidence (Sherman, 1976), rather than theoretical calculations, that local buckling will not occur if the D/t ratio is equal to or less than $3300/F_y$ when the applied stress is equal to F_y. When D/t exceeds $3300/F_y$, but is less than $13,000/F_y$, Equation (A-B5-9) provides for a reduction in allowable stress with a factor of safety against local buckling of at least 1.67. The Specification contains no recommendations for allowable stresses when D/t exceeds $13,000/F_y$.

B6. ROTATIONAL RESTRAINT AT POINTS OF SUPPORT

Slender beams and girders resting on top of columns and stayed laterally only in the plane of their top flanges may become unstable due to the flexibility of the column. Unless lateral support is provided for the bottom flange, either by bracing or continuity at the beam-to-column connection, lateral displacement at the top of the column, accompanied by rotation of the beam about its longitudinal axis, may lead to collapse of the framing.

B7. LIMITING SLENDERNESS RATIOS

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.

See Commentary E4.

B10. PROPORTIONS OF BEAMS AND GIRDERS

As in earlier editions of the Specification, it is provided that flexural members be proportioned to resist bending on the basis of moment of inertia of their gross cross section. However, the 15% flange area allowance for holes in previous specifications (Lilly and Carpenter, 1940), has been replaced by an improved criterion based on a direct comparison of tensile fracture and yield. For the fracture calculation, no hole deduction need be made until $A_{net}/A_{gross} = 6/5$ (F_y/F_u). This is equivalent to a hole allowance of 25.5% for A36 and 7.7% for $F_y = 50$ ksi material. This provision includes the design of hybrid flexural members whose flanges are fabricated from a stronger grade of steel than that in their web. As in the case of flexural members having the same grade of steel throughout their cross section, their bending strength is defined by the product of the section modulus of the gross cross section multiplied by the allowable bending stress. On this basis, the stress in the web at its junction with the flanges may even exceed the yield stress of the web material, but under strains controlled by the elastic state of stress in the stronger flanges. Numerous tests have shown that, with only minor adjustment in the basic allowable bending stress as provided in Equation (G2-1), the bending strength of a hybrid member is predictable within the same degree of accuracy as is that of a homogeneous member (ASCE-AASHO, 1968).

If a partial length cover plate is to function as an integral part of a beam or girder at the theoretical cutoff point beyond which it is not needed, it must be developed in an extension beyond this point by high-strength bolts or welding to develop its portion of the flexural stresses (i.e., the stresses which the plate would have received had it been extended the full length of the member). The cover plate force to be developed by the fasteners in the extension is equal to

$$\frac{MQ}{I} \qquad \text{(C-B10-1)}$$

where

M = moment at theoretical cutoff
Q = statical moment of cover plate area about neutral axis of coverplated section
I = moment of inertia of coverplated section

When the nature of the loading is such as to produce fatigue, the fasteners must be proportioned in accordance with the provisions of Appendix K4.

In the case of welded cover plates, it is further provided that the amount of stress that may be carried by a partial length of cover plate, at a distance a' in from its actual end, may not exceed the capacity of the terminal welds deposited along its edges and optionally across its end within this distance a'. If the moment, computed by equating MQ/I to the capacity of the welds in this distance, is less than the value at the theoretical cutoff point, either the size of the welds must be increased or the end of the cover plate must be extended to a point such that the moment on the member at the distance a' from the end of the cover plate is equal to that which the terminal welds will support.

CHAPTER C

FRAMES AND OTHER STRUCTURES

C2. FRAME STABILITY

The stability of structures as a whole must be considered from the standpoint of the structure, including not only the columns, but also the beams, bracing system and connections. The stability of individual elements must also be provided. Considerable attention has been given in the technical literature to this subject, and various methods of analysis are available to assure stability. The SSRC *Guide to Design Criteria for Metal Compression Members* devotes several chapters to the stability of different types of members considered as individual elements, and then considers the effects of individual elements on the stability of the structure as a whole (Galambos, 1988).

The effective length concept is one method for estimating the interaction effects of the total frame on a column being considered. This concept uses K-factors to equate the strength of a framed compression element of length L to an equivalent pin-ended member of length KL subject to axial load only. Other methods are available for evaluating the stability of frames subject to gravity and lateral loading and individual compression members subject to axial load and moments. The effective length concept is one tool available for handling several cases which occur in practically all structures, and it is an essential part of many analysis procedures. Although the concept is completely valid for ideal structures, its practical implementation involves several assumptions of idealized conditions which will be mentioned later.

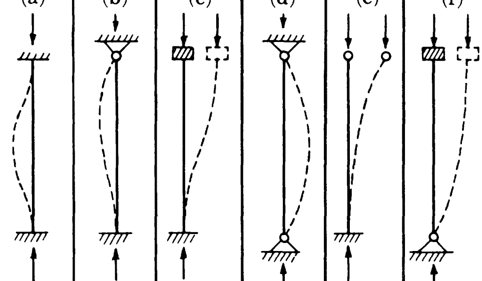
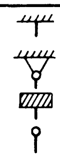
Two conditions, opposite in their effect upon column strength under axial loading, must be considered. If enough axial load is applied to the columns in an unbraced frame dependent entirely on its own bending stiffness for resistance to lateral deflection of the tops of the columns with respect to their bases (see Fig. C-C2.1), the effective length of these columns will exceed the actual length. On the other hand, if the same frame were braced to resist such lateral movement, the effective length would be less than the actual length, due to the restraint (resistance to joint rotation) provided by the bracing or other lateral support. The ratio K, effective column length to actual unbraced length, may be greater or less than 1.0.

The theoretical K-values for six idealized conditions in which joint rotation and translation are either fully realized or nonexistent are tabulated in Table C-C2.1. Also shown are suggested design values recommended by the Structural Stability Research Council for use when these conditions are approximated in actual design. In general, these suggested values are slightly higher than their theoretical equivalents, since joint fixity is seldom fully realized.

If the column base in Case f of Table C-C2.1 were truly pinned, K would actually exceed 2.0 for a frame such as that pictured in Fig. C-C2.1, because the flexibility of the horizontial member would prevent realization of full fixity at the top of the column. On the other hand, the restraining influence of foundations, even where these footings are designed only for vertical load, can be very substantial in the case of flat-ended column base details with ordinary anchorage (Stang and Jaffe, 1948). For this condition, a design K-value of 1.5 would generally be conservative in Case f.

While in some cases the existence of masonry walls provides enough lateral support for their building frames to control lateral deflection, the increasing use of light curtain wall construction and wide column spacing for high-rise structures not provided with a positive system of diagonal bracing can create a situation where only the bending stiffness of the frame itself provides this support.

Table C-C2.1

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code		Rotation fixed and translation fixed Rotation free and translation fixed Rotation fixed and translation free Rotation free and translation free				

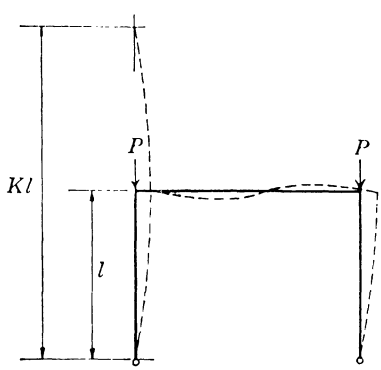


Figure C-C2.1

In this case, the effective length factor K for an unbraced length of column L is dependent on the amount of bending stiffness provided by the other in-plane members entering the joint at each end of the unbraced segment. If the combined stiffness provided by the beams is sufficiently small, relative to that of the unbraced column segments, KL could exceed two or more story heights (Bleich, 1952).

Several methods are available for estimating the effective length of columns in an unbraced frame. These range from simple interpolation between the idealized cases shown in Table C-C2.1 to very complex analytical procedures. Once a trial selection of framing members has been made, the use of the alignment chart in Fig. C-C2.2 affords a fairly rapid method for determining adequate K-values.

However, this alignment chart is based upon assumptions of idealized conditions which seldom exist in real structures (Galambos, 1988). These assumptions are as follows:

1. Behavior is purely elastic.
2. All members have constant cross section.
3. All joints are rigid.
4. For braced frames, rotations at opposite ends of beams are equal in magnitude, producing single curvature bending.
5. For unbraced frames, rotation at opposite ends of the restraining beams are equal in magnitude, producing reverse curvature bending.
6. The stiffness parameters $L\sqrt{P/EI}$ of all columns are equal.
7. Joint restraint is distributed to the column above and below the joint in proportion to I/L of the two columns.
8. All columns buckle simultaneously.

Where the actual conditions differ from these assumptions, unrealistic designs may result. There are design procedures available which may be used in the calculation of G for use in Fig. C-C2.2 to give results more truly representative of conditions in real structures (Yura, 1971 and Disque, 1973).

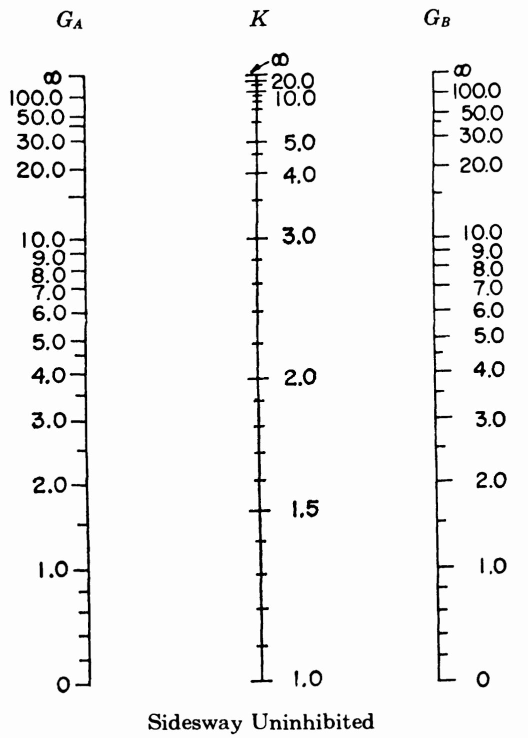
Research at Lehigh University on the load-carrying capacity of regular rectangular rigid frames has shown that it is not always necessary to directly account for the $P\Delta$ effect for a certain class of adequately stiff rigid frames (Ozer et al, 1974 and Cheong-Sait Moy, Ozer and Lu, 1977). In the research, second-order analyses using different load sequences to failure were used to confirm the adequacy of alternate allowable stress design procedures. The loading sequences used in the second order analysis were:

1. Constant gravity load at a load factor of 1.0 while the lateral load was progressively increased.
2. Constant gravity load at a load factor of 1.3 while the lateral load was progressively increased.
3. Both the lateral and gravity loads were progressively increased proportionately.

The seven frames included in the study were 10 to 40 stories high and in-plane column slenderness ratios h/r_x ranged from 18 to 42. The live load, including partitions, varied from 40 to 100 psf and the dead load from 50 to 75 psf. A uniform wind load of 20 psf was specified throughout. All beams and column sections were compact. The axial load ratios f_a/F_a and $f_a/0.60F_y$ were limited to not more than 0.75.

The results of the second order analyses showed that adequate strength and stability were assured under combined gravity and lateral loads or gravity load alone, when the rigid frames were designed by either a stress design procedure according to AISC ASD Specification requirements or by a modified stress design procedure. The modified allowable stress design procedure incorporated a stiffness parameter* which assured adequate frame stiffness, while the effective length factor K was assumed to be unity in calculations of f_a and F'_e, and the coefficient C_m was computed as for a braced frame.

Several other references** are available concerning alternatives to effective



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

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

in which Σ indicates a summation of all members rigidly connected to that joint and lying in the plane in which buckling of the column is being considered. I_c is the moment of inertia and L_c the unsupported length of a column section, and I_g is the moment of inertia and L_g the unsupported length of a girder or other restraining member. I_c and I_g are taken about axes perpendicular to the plane of buckling being considered.

For column ends supported by but not rigidly connected to a footing or foundation, G is theoretically infinity, but, unless actually designed as a true friction free pin, may be taken as "10" for practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as 1.0. Smaller values may be used if justified by analysis.

Alignment Chart for Effective Length of Columns in Continuous Frames

Fig. C-C2.2

*A design procedure based only upon a first order drift index may not assure frame stability.

**Yura, 1971; Springfield and Adams, 1972; Liapunov, 1974 (pp 1643-1655); Daniels and Lu, 1972; LeMessurier, 1976; and LeMessurier, 1977.

length factors for multistory frames under combined loads or gravity loads alone.

In frames which depend upon their own bending stiffness for stability, the amplified moments are accounted for in the design of columns by means of the interaction equations of Sect. H1. However, moments are also induced in the beams which restrain the columns; thus, consideration must be given to the amplification of those portions of the beam moments that are increased when the frame drifts. The effect may be particularly important in frames in which the contribution to individual beam moments from story shears becomes small as a result of distribution to many bays, but in which the $P\Delta$ moment in individual columns and beams is not diminished and becomes dominant.

If roof decks and 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 (Winter, 1958).

Although translation of the joints in the plane of a truss is inhibited and, due to end restraint, the effective length of compression members might therefore be assumed to be less than the distance between panel points, it is usual practice to take K as equal to 1.0, 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. For K less than unity in trusses, see Galambos (1988).

CHAPTER D

TENSION MEMBERS

D1. ALLOWABLE STRESS

Due to strain hardening, a ductile steel bar loaded in axial tension can resist, without fracture, a force greater than the product of its gross area and its coupon yield stress. However, excessive elongation of a tension member due to uncontrolled yielding of its gross area not only marks the limit of its usefulness, but can precipitate failure of the structural system of which it is a part. On the other hand, depending upon the scale of reduction of gross area and the mechanical properties of the steel, the member can fail by fracture of the net area at a load smaller than required to yield the gross area. Hence, general yielding of the gross area and fracture of the net area both constitute failure limit states.

To prevent failure of a member loaded in tension, Sect. D1 has imposed a factor of 1.67 against yielding of the entire member and of 2.0 against fracture of its weakest effective net area.

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

The mode of failure is dependent upon the ratio of effective net area to gross area and the mechanical properties of the steel. The boundary between these modes, according to the provisions of Sect. D1, is defined by the equation $A_e/A_g = 0.6F_y/0.5F_u$. When $A_e/A_g \geq F_y/0.833F_u$, general yielding of the member will be the failure mode. When $A_e/A_g < F_y/0.833F_u$, fracture at the weakest net area will be the failure mode.

In the case of short fittings used to transfer tensile force, an upper limit of 0.85 is placed on the ratio A_e/A_g. See B3.

D3. PIN-CONNECTED MEMBERS

Forged eyebars have generally been replaced by pin-connected plates or eyebars thermally cut from plates. Provisions for the proportioning of eyebars are based upon standards evolved from long experience with forged eyebars. Through extensive destructive testing, eyebars have been found to provide balanced designs when they are thermally cut instead of forged. The somewhat more conservative rules for pin-connected members of nonuniform cross section and those not having enlarged "circular" heads are likewise based on the results of experimental research (Johnston, 1939).

Somewhat stockier proportions are provided for eyebars and pin-connected members fabricated from steel having yield stress greater than 70 ksi to eliminate any possibility of their "dishing" under the higher working stress.

CHAPTER E

COLUMNS AND OTHER COMPRESSION MEMBERS

E1. EFFECTIVE LENGTH AND SLENDERNESS RATIO

The Commentary on Sect. C2 regarding frame stability and effective length factors applies here. Further analytical methods, formulas, charts and references for the determination of effective length are provided in the SSRC *Guide to Stability Design Criteria for Metal Structures* (Galambos, 1988).

E2. ALLOWABLE STRESS*

Equations (E2-1) and (E2-2) are founded upon the basic column strength estimate suggested by the Structural Stability Research Council. This estimate assumes that the upper limit of elastic buckling failure is defined by an average column stress equal to ½ of yield stress. The slenderness ratio C_c corresponding to this limit, can be expressed in terms of the yield stress of a given grade of structural steel as

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} \tag{C-E2-1}$$

A variable factor of safety has been applied to the column strength estimate to obtain allowable stresses. For very short columns, this factor has been taken as equal to, or only slightly greater than, that required for members axially loaded in tension, and can be justified by the insensitivity of such members to accidental eccentricities. For longer columns, entering the Euler slenderness range, the factor is increased 15% to approximately the value provided in the AISC Specification since it was first published.

To provide a smooth transition between these limits, the factor of safety has been defined arbitrarily by the algebraic equivalent of a quarter sine curve whose abscissas are the ratio of given Kl/r values to the limiting value C_c, and whose ordinates vary from 5/3 when Kl/r equals 0 to 23/12 when Kl/r equals C_c.

Equation (E2-2) covering slender columns (Kl/r greater than C_c) which fail by elastic buckling, is based upon a constant factor of safety of 23/12 with respect to the elastic (Euler) column strength.

E3. FLEXURAL-TORSIONAL BUCKLING

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

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*For tapered members, also see Commentary Appendix F7.

Appendix E3 of the LRFD Specification (AISC, 1986) may be used to establish the effect of flexural-torsional buckling. The critical elastic buckling stress F_e can be obtained directly from the equations in LRFD Appendix E3. The effective slenderness is then given by

$$\left(\frac{KL}{r}\right)_e = \pi \sqrt{\frac{E}{F_e}} \quad \text{(C-E2-2)}$$

The allowable stress is then obtained from Equations (E2-1) or (E2-2).

E4. BUILT-UP MEMBERS

Requirements for detailing of built-up members, which cannot be stated in terms of calculated stress, are based upon judgment, tempered by experience.

The longitudinal spacing of fasteners connecting components of built-up compression members must be such that the effective slenderness ratio K_a/r of the individual shape does not exceed 75% of the slenderness ratio Kl/r of the entire member. In addition, at least two intermediate connectors must be used along the length of the built-up member. To minimize the possibility of slip, the connectors must be welded or use high-strength bolts tightened to the requirements of Table J3.7. However, maximum fastener spacing less than that necessary to prevent local buckling may be needed to ensure a close fit-up over the entire faying surface of components designed to be in contact.

Provisions based on this latter consideration 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 on extensive experimental research (Stang and Jaffe, 1948).

E6. COLUMN WEB SHEAR

The column web shear stresses may be high within the boundaries of the rigid connection of two or more members whose webs lie in a common plane. Such webs should be reinforced when the calculated stress along plane *A-A* in Fig. C-E6.1 exceeds the allowable shear stress

$$\Sigma F = \frac{M_1}{0.95d_1} + \frac{M_2}{0.95d_2} - V_s \quad \text{(C-E6-1)}$$

$$\Sigma F/(d_c \times t_w) \leq F_v \quad \text{(C-E6-2)}$$

where:

$M_1 = M_{1L} + M_{1G}$ = sum of the moments due to the lateral load M_{1L} and the moments due to gravity load M_{1G} on the leeward side of the connection, kip-in.

$M_2 = M_{2L} - M_{2G}$ = difference between the moments due to lateral load M_{2L} and the moments due to gravity load M_{2G} on the windward side of the connection, kip-in.

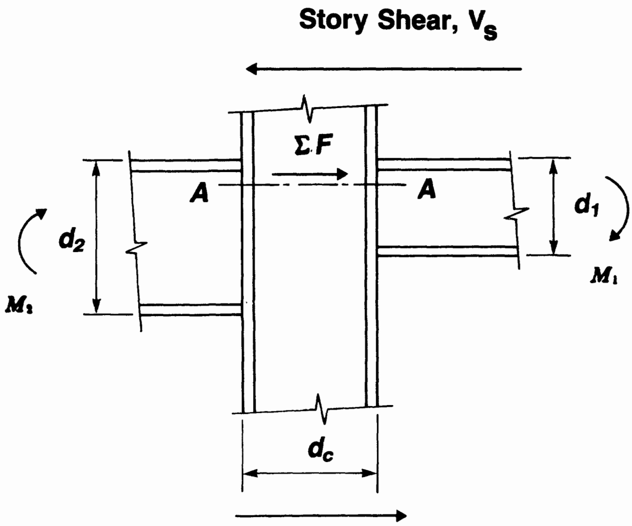


Fig. C-E6.1

CHAPTER F

BEAMS AND OTHER FLEXURAL MEMBERS

When flexural members, loaded to produce bending about their strong axis, are proportioned with width-thickness ratios not exceeding the noncompact section limits of Sect. B5, and are adequately braced to prevent the lateral displacement of the compression flange, they provide bending resistance equal at least to the product of their section modulus and yield stress, even when the width-thickness ratio of compressed elements of their profile is such that local buckling may be imminent. Lateral buckling of members bent about their strong axis may be prevented by bracing which either restrains the compression flange against lateral displacement or restrains the cross section against twisting which would induce bending about the weaker axis. Members bent solely about their minor axis, and members having approximately the same strength about both axes, do not buckle laterally and therefore may be stressed to the full allowable bending stress, consistent with the width-thickness proportions of their compression elements, without bracing.

F1. ALLOWABLE STRESS: STRONG AXIS BENDING OF I-SHAPED MEMBERS AND CHANNELS

1. Members with Compact Sections

Research in plastic design has demonstrated that local buckling will not occur in homogeneous sections meeting the requirements of Sect. F1.1 before the full plastic moment is reached. Practically all W and S shapes of A36 steel and a large proportion of these shapes having a yield stress of 50 ksi meet these provisions and are termed "compact" sections. It is obvious that the possibility of overload failure in bending of such rolled shapes must involve a higher level of stress (computed on the basis of M/S) than members having more slender compression elements. Since the shape factor of W and S beams is generally in excess of 1.12, the allowable bending stress for such members has been raised 10% from $0.60F_y$ to $0.66F_y$.

The further provisions permitting the arbitrary redistribution of 10% of the moment at points of support, due to gravity loading, gives partial recognition to the philosophy of plastic design. Subject to the restrictions provided in Sect. F1.1, continuous framing consisting of compact members may be proportioned on the basis of the allowable stress provisions of Chaps. D through K of the Specification when the moments, before redistribution, are determined on the basis of an elastic analysis. Fig. C-F1.1 illustrates the application of this provision by comparing calculated moment diagrams with the diagrams as altered by this provision.

2. Members with Noncompact Sections

Equation (F1-3) avoids an abrupt transition between an allowable bending stress of $0.66F_y$ when the half-flange width-to-thickness ratio of laterally supported compression flanges exceeds $65/\sqrt{F_y}$ and when this ratio is no more than $95/\sqrt{F_y}$. The assured hinge rotation capacity in this range is too small to permit redistribution of computed moment. Equation (F1-4) performs a similar function for homogeneous plate girders. See Commentary Sect. B5.

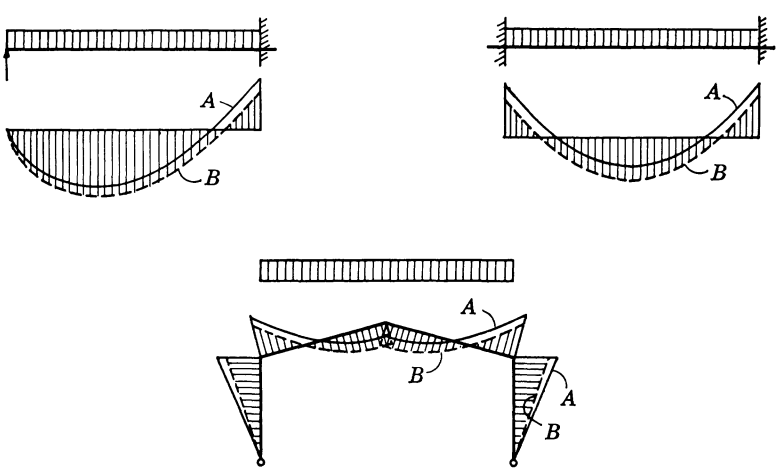
The allowable bending stress for all other flexural members is given as $0.60F_y$, provided the member is braced laterally at relatively close intervals ($l/b_f \leq 76/\sqrt{F_y}$).

3. Members with Compact or Noncompact Sections With Unbraced Length Greater than L_c

Members bent about their major axis and having an axis of symmetry in the plane of loading may be braced laterally at intervals greater than $76b_f/\sqrt{F_y}$ or $20{,}000/(d/A_f)F_y$ if the maximum bending stress is reduced sufficiently to prevent premature buckling of the compression flange.

The combination of Equations (F1-6) or (F1-7) and (F1-8) provides a reasonable design criterion in convenient form. Equations (F1-6) and (F1-7) are based on the assumption that only the bending stiffness of the compression flange will prevent the lateral displacement of that element between bracing points.

Equation (F1-8) is a convenient approximation which assumes the presence of both lateral bending resistance and St. Venant torsional resistance. Its agreement with more exact expressions for the buckling strength of intermittently braced flexural members (Galambos, 1988) is closest for homogeneous sections having substantial resistance to St. Venant torsion, identifiable in the case of



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

Fig. C-F1.1

doubly symmetrical sections by a relatively low d/A_f ratio. Due to the difference between flange and web yield strength of a hybrid girder, it is desirable to base the lateral buckling resistance solely on warping torsion of the flange. Hence, use of Equation (F1-8) is not permitted for such members.

For some sections having a compression flange area distinctly smaller than the tension flange area, Equation (F1-8) may be unconservative; for this reason, its use is limited to sections whose compression flange area is at least as great as the tension flange. In plate girders, which usually have a much higher d/A_f ratio than rolled W shapes, Equation (F1-8) may err grossly on the conservative side. For such members, the larger stress permitted by Equation (F1-6), and at times by Equation (F1-7), affords the better estimate of buckling strength. Although these latter equations underestimate the buckling strength somewhat because they ignore the St. Venant torsional rigidity profile, this rigidity for such sections is relatively small and the margin of overconservatism, therefore, is likewise small.

Equation (F1-8) is written for the case of elastic buckling. A transition is not provided for this formula in the inelastic stress range because, when actual conditions of load application and variation in bending moment are considered, any unconservative error without the transition will be small.

Singly symmetrical, built-up, I-shape members, such as some crane girders, often have an increased compression flange area in order to resist bending due to lateral loading acting in conjunction with the vertical loads. Such members usually can be proportioned for the full allowable bending stress when the stress is produced by the combined vertical and horizontal loading. Where the failure mode of a singly symmetrical I-shape member having a larger compression than tension flange would be by lateral buckling, the allowable bending stress can be obtained by using Equations (F1-6), (F1-7) or (F1-8).

Through the introduction of the modifier C_b, some liberalization in stress is permissible when there is moment gradient over the unbraced length, except that C_b must be taken as unity when computing F_{bx} for use in Equation (H1-1) for frames braced against joint translation (Galambos, 1988).

Equations (F1-6) and (F1-7) may be refined to include both St. Venant and warping torsion by substituting a derived value for r_T. The equivalent radius of gyration, r_{Tequiv}, can be obtained by equating the appropriate expression giving the critical elastic bending stress for the compression flange of a beam with that of an axially loaded column (Galambos, 1988).

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

$$(r_{Tequiv})^2 = \frac{I_y}{2S_x}\sqrt{d^2 + \frac{0.156l^2J}{I_y}} \qquad \text{(C-F1-1)}$$

where

I_y = minor axis moment of inertia of the member
S_x = major axis section modulus

$$J = \frac{2b_f t_f^3}{3} + \frac{dt^3}{3} \qquad \text{(C-F1-2)}$$

Closer approximations of Equations (F1-7) and (F1-8) are given in Galambos, 1988.

F2. ALLOWABLE STRESS: WEAK AXIS BENDING OF I-SHAPED MEMBERS, SOLID BARS AND RECTANGULAR PLATES

The 25% increase in allowable bending stress for compact sections and solid rectangular bars bent about their weak axis, as well as for square and rectangular bars, is based upon the favorable shape factor present when these sections are bent about their weaker axis, and the fact that, in this position, they are not subject to lateral-torsional buckling. While the plastic bending strength of these shapes, bent in this direction, is considerably more than 25% in excess of their elastic bending strength, full advantage is not taken of this fact in order to provide elastic behavior at service loading.

Equation (F2-3), like Equation (F1-3), is a transition between the allowable bending stress of $0.75F_y$ at $b_f/2t_f = 65/\sqrt{F_y}$ and the lower stress of $0.60F_y$ at $b_f/2t_f = 95/\sqrt{F_y}$.

F3. ALLOWABLE STRESS: BENDING OF BOX MEMBERS, RECTANGULAR TUBES AND CIRCULAR TUBES

The provision for compact circular members is given in Table B5.1 (Sherman, 1976).

Supplement No. 3 (1974) to the 1969 Specification added Equation (F3-2), an unsupported length criteria for compact tubular members with rectangular cross sections. The equation recognizes the effect of moment gradient, and tests have shown it to be conservative (Sherman, 1976).

Box-type members are torsionally very stiff (Galambos, 1988). The critical flexural stress due to lateral-torsional buckling, for the compression flange of a box-type beam loaded in the plane of its minor axis so as to bend about its major axis, can be obtained using Equation (E2-1) with an equivalent slenderness ratio, by the expression

$$\left(\frac{l}{r}\right)_{equiv} = \sqrt{\frac{5.1lS_x}{\sqrt{JI_y}}} \qquad \text{(C-F3-1)}$$

where:

l = distance between points of lateral support, in.
S_x = elastic section modulus about major axis, in.^{3}
I_y = moment of inertia about minor axis, in.^{4}
J = torsional constant for a section, in.^{4}

It can be shown that, when $d < 10b$ and $l/b > 2500/F_y$, the allowable compression flange stress indicated by the above equation will approximate $0.60F_y$. Beyond this limit, deflection rather than stress is likely to be the design criterion.

F4. ALLOWABLE SHEAR STRESS

Although the shear yield stress of structural steel has been variously estimated as between ½ and ⅝ of the tension and compression yield stress and is frequently taken as $F_y/\sqrt{3}$, it will be noted that the allowable value is given as ⅔ the recommended basic allowable tensile stress, substantially as it has been since the first edition of the AISC Specification published in 1923. This apparent reduction in factor of safety is justified by the minor consequences of shear yielding, as compared with those associated with tension and compression yielding, and by the effect of strain hardening.

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

When the computed average shear stress in the web is less than that permitted by Equation (F4-2), intermediate stiffeners are not required, provided the depth of the girders is limited to 260 times the web thickness. Such girders do not depend upon tension field action.

F5. TRANSVERSE STIFFENERS

In order to facilitate handling during fabrication and erection, when intermediate stiffeners are required the panel aspect ratio a/h is arbitrarily limited by Equation (F5-1) to $[260/(h/t_w)]^2$, with a maximum spacing of 3 times the girder depth.

CHAPTER G

PLATE GIRDERS

G1. WEB SLENDERNESS LIMITATIONS

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

The provision $h/t_w \leq 2000/\sqrt{F_y}$ is based upon tests on both homogeneous and hybrid girders with flanges having a specified yield stress of 100 ksi and a web of similar or weaker steel (ASCE-AASHO, 1968).

G2. ALLOWABLE BENDING STRESS

In regions of maximum bending moment, a portion of a thin web may deflect enough laterally on the compression side of the neutral axis that it does not provide the full bending resistance assumed in proportioning the girder on the basis of its moment of inertia. The compression stress which the web would have resisted is therefore shifted to the compression flange. But because the relative bending strength of this flange is so much greater than that of the laterally displaced portion of the web, the resulting increase in flange stress is at most only a few percent. The allowable design stress in the compression flange is reduced by the plate girder factor R_{PG} to ensure sufficient bending capacity is provided in the flange to compensate for any loss of bending strength in the web due to its lateral displacement.

To compensate for the slight loss of bending resistance when portions of the web or a hybrid flexural member are strained beyond their yield stress limit, the hybrid girder factor R_e reduces the allowable flange bending stress applicable to both flanges. The extent of the reduction is dependent upon the ratio of web area to flange area and of $.6F_{yw}$ to F_b. This is changed due to the reduction of F_{yf} based on local or lateral buckling. These reduction factors are multiplicable in the determination of the allowable bending stress for hybrid girders (Equation (G2-1)). This is to reflect the fact that the web continues to contribute some strength beyond the point of theoretical web buckling ($h/t_w = 760/\sqrt{F_b}$).

G3. ALLOWABLE SHEAR STRESS WITH TENSION FIELD ACTION

Unlike columns, which actually are on the verge of collapse as their buckling stage is approached, the panels of the 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 (Basler and Thurlimann, 1963 and Basler, 1961) and corroborated in an extensive program of tests (Basler et al, 1960). These methods form the basis for Equation (G3-1). Use of tension field action is not counted upon when $0.60F_y\sqrt{3} \leq F_u \leq 0.40F_y$, nor when $a/h > 3.0$. Until further research is completed, it is not recommended for hybrid girders.

To provide adequate lateral support for the web, all stiffeners are required to have a moment of inertia at least equal to $(h/50)^4$. In many cases, however, this provision will be overshadowed by the gross area requirement. The amount of stiffener area necessary to develop the tension field, which is dependent upon the ratios a/h and h/t_w, is given by Equation (G4-1). 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 sized fillet weld. The specified Equation (G4-3) affords a conservative estimate of required shear transfer under any condition of stress permitted by Equation (G3-1). 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 stiffeners are not additive. The stiffener need only be connected for the larger of the two shears.

G4. TRANSVERSE STIFFENERS

See Commentary G3.

G5. COMBINED SHEAR AND TENSION STRESS

Unless a flexural member is designed on the basis of tension field action, no stress reduction is required due to the interaction of concurrent bending and shear stress.

It has been shown that plate girder webs subject to tension field action can be proportioned on the basis of (Basler, 1979):

1. The allowable bending stress F_b, when the concurrent shear stress f_v is not greater than 0.60 of the allowable shear stress F_v or
2. The allowable shear stress F_v when the concurrent bending stress f_b is not greater than 0.75 of the allowable bending stress F_b.

Beyond these limits a linear interaction formula is provided in the AISC ASD Specification by Equation (G5-1).

However, because the webs of homogeneous girders of steel with yield points greater than 65 ksi loaded to their full capacity in bending develop more waviness than less-heavily-stressed girder webs of lower strength grades of steel, use of tension field action is limited in the case of webs with yield stress greater than 65 ksi to regions where the concurrent bending stress is no more than $0.75F_b$.

CHAPTER H

COMBINED STRESSES

H1. AXIAL COMPRESSION AND BENDING

The application of moment along the unbraced length of axially loaded members, with its attendant axial displacement in the plane of bending, generates a secondary moment equal to the product of resulting eccentricity and the applied axial load, which is not reflected in the computed stress f_b. To provide for this added moment in the design of members subject to combined axial and bending stress, Equation (H1-1) requires that f_b be amplified by the factor

$$\frac{1}{\left(1 - \frac{f_a}{F'_e}\right)} \qquad \text{(C-H1-1)}$$

Depending upon the shape of the applied moment diagram (and, hence, the critical location and magnitude of the induced eccentricity), this factor may overestimate the extent of the secondary moment. To take care of this condition the amplification factor is modified, as required, by a reduction factor C_m.

When bending occurs about both the x- and y-axes, the bending stress calculated about each axis is adjusted by the value of C_m and F'_e corresponding to the distribution of moment and the slenderness ratio in its plane of bending. It is then taken as a fraction of the stress permitted for bending about that axis, with due regard to the unbraced length of compression flange where this is a factor.

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

$$\frac{C_m}{\left(1 - \frac{f_a}{F'_e}\right)} \qquad \text{(C-H1-2)}$$

is generally small and may be neglected, as provided in Equation (H1-3). However, its use in Equation (H1-1) is not intended to permit a value of f_b greater than F_b when the value of C_m and f_a are both small.

Depending upon the slenderness ratio of the given unbraced length of a member in the plane of bending, the combined stress computed at one or both ends of this length may exceed the combined stress at all intermediate points where lateral displacement is created by the applied moments. The limiting value of the combined stress in this case is established by Equation (H1-2).

The classification of members subject to combined axial compression and bending stresses is dependent upon two conditions: the stability against sidesway of the frame of which they are an integral part, and the presence or absence of transverse loading between points of support in the plane of bending.

Note that f_b is defined as the computed bending stress *at the point under consideration*. In the absence of transverse loading between points of support, f_b is computed from the larger of the moments at these points of support. When intermediate transverse loading is present, the larger moment at one of the two supported points is used to compute f_b for use in the Equation (H1-2). However, to investigate the possibility of buckling failure, the maximum moment between points of support is used to compute f_b in Equation (H1-1).

In Equations (H1-1), (H1-2) and (H1-3), F_{bx} includes lateral-torsional buckling effects as provided in Equations (F1-6), (F1-7) and (F1-8).

Three categories are to be considered in computing values of C_m:

Category a covers columns in frames subject to sidesway, i.e., frames which depend upon the bending stiffness of their several members for overall lateral stability. For determining the value of F_a and F'_e, the effective length of such members, as discussed under C2, is never less than the actual unbraced length in the plane of bending, and may be greater than this length. The actual length is used in computing moments. For this case the value of C_m can be taken as

$$C_m = 1 - 0.18 f_a / F'_e \tag{C-H1-3}$$

However, under the combination of compression stress and bending stress most affected by the amplification factor, a value of 0.15 can be substituted for $0.18 f_a / F'_e$. Hence, a constant value of 0.85 is recommended for C_m here.*

Category b applies to columns not subject to transverse loading in frames where sidesway is prevented. For determining the value of F_a and F'_e, the effective length of such members is never greater than the actual unbraced length and may be somewhat less. The actual length is used in computing moments.

For this category, the greatest eccentricity, and hence the greatest amplification, occurs when the end moments, M_1 and $-M_2$** are numerically equal and cause single curvature. It is least when they are numerically equal and of a direction to cause reverse curvature.

To properly evaluate the relationship between end moment and amplified moment, the concept of an equivalent moment M_e to be used in lieu of the numerically smaller end moment, has been suggested. M_e can be defined as the value of equal end moments of opposite signs which would cause failure at the same concurrent axial load as would the given unequal end

*See Commentary Sect. C2 for cases where C_m for unbraced frames 10 to 40 stories high may be computed as for braced frames.

**The sign convention for moments here and in Chap. H is that generally used in frame analysis. It should not be confused with the beam sign convention used in many textbooks. Moments are considered positive when acting clockwise about a fixed point, negative when acting counterclockwise.

moments. Then, M_e/M_2 can be written in terms of $\pm M_1/M_2$ as (Galambos, 1988):

$$\frac{M_e}{M_2} = C_m = \sqrt{0.3\left(\frac{M_1}{M_2}\right)^2 - 0.4\left(\pm\frac{M_1}{M_2}\right) + 0.3} \qquad \text{(C-H1-4)}$$

It has been noted that the simpler formulation (Austin, 1961):

$$C_m = 0.6 - 0.4\left(\pm\frac{M_1}{M_2}\right) \geq 0.4 \qquad \text{(C-H1-5)}$$

affords a good approximation to this expression. The 0.4 limit on C_m corresponding to a M_1/M_2 ratio of 0.5, was included in the 1978 AISC Specification. The limit was intended to apply to lateral-torsional buckling and not to second-order, in-plane bending strength. As in the 1978 AISC Specification and the 1986 AISC LRFD Specification, this AISC ASD Specification uses a modification factor C_b as given in Sect. F1.3 for lateral-torsional buckling. C_b which is limited to 2.3, is approximately the inverse of C_m as presented in Austin (1961) with a 0.4 limit. In Zandonini (1985) it was pointed out this C_m equation could be used for in-plane second order moments if the 0.4 limit was eliminated. This adjustment has been made here, as it is in the 1986 AISC LRFD Specification.

Category c is exemplified by the compression chord of a truss subject to transverse loading between panel points, or by a simply supported column subjected to transverse loads between supports. For such cases, the value of C_m can be approximated using the equation:

$$C_m = 1 + \psi\frac{f_a}{F'_e} \qquad \text{(C-H1-6)}$$

Values of ψ for several conditions of transverse loading and end restraint (simulating continuity at panel points) are given in Table C-H1.1, together with two cases of simply supported beam-columns. In the case of continuity at panel points, f_b is maximum at the restrained ends or end, and the value of C_m for usual f_a/F'_e ratios is only slightly less than unity (a value of 0.85 is suggested in the Specification in the final paragraph of H1). For determinate (simply supported) beam-columns, f_b is maximum at or near midspan, depending upon the pattern of transverse loading. For this case

$$\psi = \frac{\pi^2\delta_o EI}{M_o L^2} - 1 \qquad \text{(C-H1-7)}$$

where

δ_o = maximum deflection due to tranverse loading
M_o = maximum moment between supports due to transverse loading

If, as in the case of a derrick boom, such a beam-column is subject to transverse (gravity) load and a calculable amount of end moment, δ_o should include the deflection between supports produced by this moment.

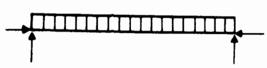
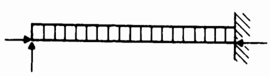
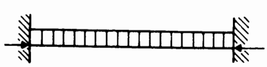
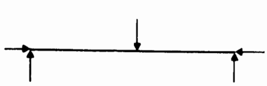
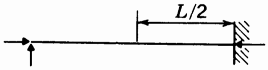
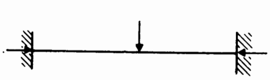
It should be noted that, for amplified end moments in indeterminate members, stress alone is critical and is controlled by Equation (H1-2). For determinate members, where the amplified bending stress is maximum between supports, buckling-type failure is also of concern.

Note that F_a is governed by the maximum slenderness ratio, regardless of the plane of bending. F'_e, on the other hand, is always governed by the slenderness ratio in the plane of bending. Thus, when flexure is about the strong axis only, two different values of slenderness ratio may be involved in solving a given problem.

H2. Axial Tension and Bending

Contrary to the behavior in compression members, axial tension tends to *reduce* the bending stress because the secondary moment, which is the product of the deflection and the axial tension, is opposite in sense to the applied moment; thus, the secondary moment diminishes, rather than amplifies, the primary moment.

TABLE C-H1.1
Amplification Factors ψ and C_m

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

CHAPTER I

COMPOSITE CONSTRUCTION

I1. DEFINITION

When the dimensions of a concrete slab supported by steel beams are such that the slab can effectively serve as the flange of a composite T-beam, and the concrete and steel are adequately tied together so as to act as a unit, the beam can be proportioned on the assumption of composite action.

Two cases are recognized: fully encased steel beams, which depend upon natural bond for interaction with the concrete, and beams with mechanical anchorage to the slab (shear connectors), which do not have to be encased.

For composite beams with formed steel deck, studies have demonstrated that total slab thickness, including ribs, can be used in determining effective slab width (Grant, Fisher and Slutter, 1977 and Fisher, 1970).

I2. DESIGN ASSUMPTIONS

Unless temporary shores are used, beams encased in concrete and interconnected only by a natural bond must be proportioned to support all of the dead load, unassisted by the concrete, plus the superimposed live load in composite action, without exceeding the allowable bending stress for steel provided in Chap. F.

Because the completely encased steel section is restrained from both local and lateral buckling, an allowable stress of $0.66F_y$, rather than $0.60F_y$, can be applied when the analysis is based on the properties of the transformed section. The alternate provision to be used in designs where a fully encased beam is proportioned, on the basis of the steel beam alone, to resist all loads at a stress not greater than $0.76F_y$, reflects a common engineering practice where it is desired to eliminate the calculation of composite section properties.

When shear connectors are used to obtain composite action, this action may be assumed, within certain limits, in proportioning the beam for the moments created by the sum of live and dead loads, even for unshored construction (Fisher, 1970). This liberalization is based upon an ultimate strength concept, although the provisions for proportioning of the member are based upon the elastic section modulus of the transformed cross section.

The flexural capacity of composite steel-concrete beams designed for complete composite action is the same for either lightweight or normal weight concrete, given the same area of concrete slab and concrete strength, but with the number of shear connectors appropriate to the type of concrete. The same concrete design stress level can be used for both types of concrete.

For unshored construction, so the steel beam under service loading will remain elastic, the superposition of precomposite and composite stresses is limited to $0.9F_y$. This direct stress check replaces the derived equivalent maximum transformed section modulus used in the past. The $0.9F_y$ stress limit only prevents permanent deformation under service loads and has no effect on the ultimate moment capacity of the composite beam.

On the other hand, to avoid excessively conservative slab-to-beam proportions, it is required that the flexural stress in the concrete slab, due to composite action, be computed on the basis of the transformed section modulus, referred to the top of concrete, and limited to the generally accepted working stress limit.

For a given beam and concrete slab, the increase in bending strength intermediate between no composite action and full composite action is dependent upon the shear resistance developed between the steel and concrete, i.e., the number of shear connectors provided between these limits (Slutter and Driscoll, 1965). Usually, it is not necessary, and occasionally it may not be feasible, to provide full composite action. Therefore, the AISC ASD Specification recognizes two conditions: full and partial composite action.

For the case where total shear V'_h developed between steel and concrete on each side of the point of maximum moment is less than V_h, Equation (I2-1) can be used to derive an effective section modulus S_{eff} having a value less than the section modulus for fully effective composite action S_{tr}, but more than that of the steel beam alone.

I4. SHEAR CONNECTORS

Composite beams in which the longitudinal spacing of shear connectors has been varied according to the intensity of 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 the other less heavily stressed connectors. The important consideration is that the total number of connectors be sufficient to develop the shear V_h either side of the point of maximum moment. The provisions of the AISC ASD Specification are based upon this concept of composite action.

In computing the section modulus at points of maximum negative bending, reinforcement parallel to the steel beam and lying within the effective width of slab may be included, provided such reinforcement is properly anchored beyond the region of negative moment. However, enough shear connectors are required to transfer, from slab to the steel beam, one-half of the ultimate tensile strength of the reinforcement.

Studies have defined stud shear connection strength Q_u in terms of normal weight and lightweight aggregate concretes, as a function of both concrete modulus of elasticity and concrete strength (McGarraugh and Baldwin, 1971 and Ollgaard, Slutter and Fisher, 1971):

$$Q_u = 0.5A_s\sqrt{f'_c E_c} \qquad \text{(C-I4-1)}$$

where

A_s =cross-sectional area of stud, in.^{2}
f'_c =concrete compressive strength, ksi
E_c =concrete modulus of elasticity, ksi

Tests have shown that fully composite beams designed using the values in Tables I4.1 and/or I4.2 as appropriate, and concrete meeting the requirements of Part 3, Chap. 4, "Concrete Quality", of ACI Standard 318-83 made with ASTM C33 or C330 aggregates, develop their full flexural capacity (Ollgaard, Slutter and Fisher, 1971). For normal weight concrete, compressive strengths greater than 4.0 ksi do not increase the shear capacity of the connectors, as is reflected in Table I4.1. For lightweight concrete, compressive strengths greater than 5 ksi do not increase the shear capacity of the connectors. The reduction coefficients in Table I4.2 are applicable to both stud and channel shear connectors and provide comparable margins of safety.

When partial composite action is counted upon to provide flexural capacity, the restriction on the minimum value of V'_h is to prevent excessive slip as well as substantial loss in beam stiffness. Studies indicate that Equations (I2-1) and (I4-4) adequately reflect the reduction in strength and beam stiffness, respectively, when fewer connectors than required for full composite action are used.

Where adequate flexural capacity is provided by the steel beam alone, that is, composite action to any degree is not required for flexural strength, but where it is desired to provide interconnection between the steel frame and the concrete slab for other reasons, such as to increase frame stiffness or to take advantage of diaphragm action, the minimum requirement that V_h be not less than $V_h/4$ does not apply.

The required shear connectors can generally be spaced uniformly between the points of maximum and zero moment (Slutter and Driscoll, 1965). However, certain loading patterns can produce a condition where closer connector spacing is required over part of this distance.

For example, consider the case of a uniformly loaded simple beam also required to support two equal concentrated loads, symmetrically disposed about midspan, of such magnitude that the moment at the concentrated loads is only slightly less than the maximum moment at midspan. The number of shear connectors N_2 required between each end of the beam and the adjacent concentrated load would be only slightly less than the number N_1 required between each end and midspan.

Equation (I4-5) is provided to determine the number of connectors, N_2, required between one of the concentrated loads and the nearest point of zero moment. It is based upon the following requirement:

$$\frac{N_2}{N_1} = \frac{S - S_s}{S_{eff} - S_s} = \frac{\left[\frac{S}{S_{eff}} \times \frac{S_{eff}}{S_s}\right] - 1}{\frac{S_{eff}}{S_s} - 1} \qquad \text{(C-I4-2)}$$

where

S = section modulus required at the concentrated load at which location moment equals M, in.^{3}

S_{eff} = section modulus required at M_{max} (equal to S_{tr} for fully composite case), in.^{3}

S_s = section modulus of steel beam, in.^{3}

N_1 = number of studs required from M_{max} to zero moment

N_2 = number of studs required from M to zero moment

M = moment at the concentrated load point

M_{max} = maximum moment in the beam

Noting that $S/S_{eff} = M/M_{max}$, and defining β as S_{eff}/S_s, the above equation is equivalent to Equation (I4-5).

With the issuance of Supplement No. 3 to the 1969 AISC Specification, the requirement for 1-in. cover over the tops of studs was eliminated. Only the concrete surrounding the stud below the head contributes to the strength of the stud in resistance to shear. When stud shear connectors are installed on beams with formed steel deck, concrete cover at the sides of studs adjacent to sides of steel ribs is not critical. Tests have shown that studs installed as close as is permitted to accomplish welding of studs does not reduce the composite beam capacity.

Stud welds not located directly over the web of a beam tend to tear out of a thin flange before attaining their full shear-resisting capacity. To guard against this contingency, the size of a stud not located over the beam web is limited to 2½ times the flange thickness.

I5. COMPOSITE BEAMS OR GIRDERS WITH FORMED STEEL DECK

The 6-diameter minimum center-to-center spacing of studs in the longitudinal direction is based upon observation of concrete shear failure surfaces in sectioned flat soffit concrete slab composite beams which had been tested to full ultimate strength. The reduction in connection capacity of more closely spaced shear studs within the ribs of formed steel decks oriented perpendicular to beam or girder, is accounted for by the parameter $0.85/\sqrt{N_r}$ in Equation (I5-1).

When studs are used on beams with formed steel deck, they may be welded directly through the deck or through prepunched or cut-in-place holes in the deck. The usual procedure is to install studs by welding directly through the deck; however, when the deck thickness is greater than 16 ga. for single thickness, or 18 ga. for each sheet of double thickness, or when the total thickness of galvanized coating is greater than 1.25 ounces per sq. ft., special precautions and procedures recommended by the stud manufacturer should be followed.

Fig. C-I5.1 is a graphic presentation of the terminology used in Sect. I5.1.

The design rules which have been added for composite construction with formed steel deck are based upon a study of all available test results (Grant, Fisher and Slutter, 1977). The limiting parameters listed in Sect. I5.1 were established to keep composite construction with formed steel deck within the available research data.

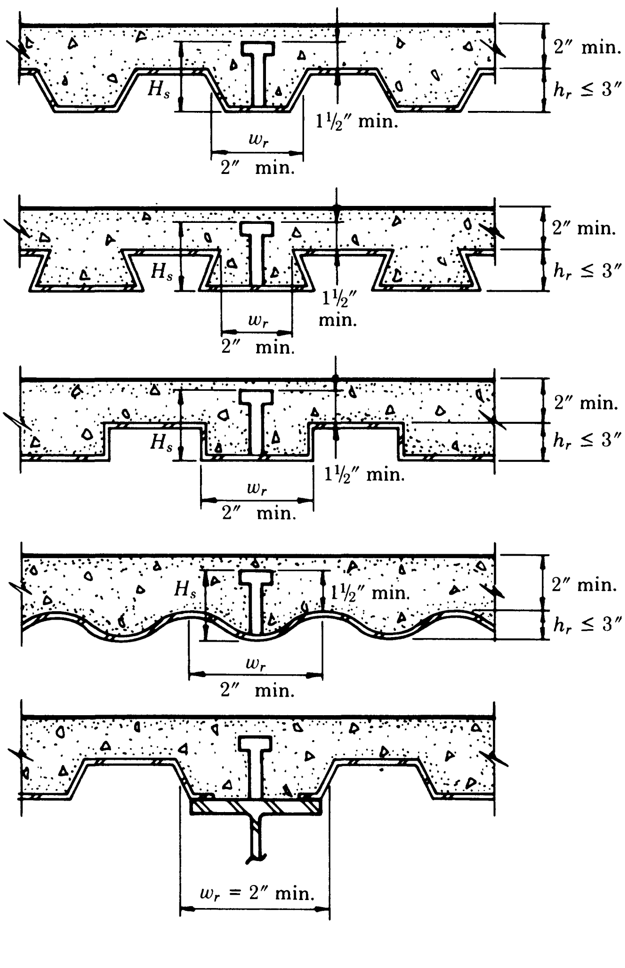


Figure C-I5.1

Seventeen full-sized composite beams with concrete slab on formed steel deck were tested at Lehigh University and the results supplemented by the results of 58 tests performed elsewhere. The range of stud and steel deck dimensions encompassed by the 75 tests were limited to:

1. Stud dimensions: ¾-in dia. × 3.00 to 7.00 in.
2. Rib width: 1.94 in. to 7.25 in.
3. Rib height: 0.88 in. to 3.00 in.
4. Ratio w_r/h_r: 1.30 to 3.33
5. Ratio H_s/h_r: 1.50 to 3.41
6. Number of studs in any one rib: 1, 2, or 3

Based upon all tests, the strength of stud connectors in flat soffit composite slab beams, determined in previous test programs, when multiplied by values computed from Equation (I5-1), reasonably approximates the strength of stud connectors installed in the ribs of concrete slabs on formed steel deck with the ribs oriented perpendicular to the steel beam (Ollgaard, Slutter and Fisher, 1971). Hence, Equation (I5-1) provides a reasonable reduction factor to be applied to the allowable design stresses in Tables I4.1 and I4.2.

Testing has shown that the maximum spacing of shear connectors can be increased to 36 in. instead of the previous value of 32 in. (Klyce, 1988).

For the case where ribs run parallel to the beam, limited testing has shown that shear connection is not significantly affected by the ribs (Grant, Fisher and Slutter, 1977). However, for narrow ribs, where the ratio w_r/h_r is less than 1.5, a shear stud reduction factor, Equation (I5-2), has been suggested in view of lack of test data.

The Lehigh study also indicated that Equation (I2-1) for effective section modulus and Equation (I4-4) for effective moment of inertia were valid for composite construction with formed steel deck (Grant, Fisher and Slutter, 1977).

When metal deck includes units for carrying electrical wiring, crossover headers are commonly installed over the cellular deck, perpendicular to the ribs, in effect creating trenches which completely or partially replace sections of the concrete slab above the deck. These trenches, running parallel to or transverse to a composite beam, may reduce the effectiveness of the concrete flange. Without special provisions to replace the concrete displaced by the trench, the trench should be considered as a complete structural discontinuity in the concrete flange. When trenches are parallel to the composite beam, the effective flange width should be determined from the known position of the trench.

Trenches oriented transverse to the composite beam should, if possible, be located in areas of low bending moment and the full required number of studs should be placed between the trench and the point of maximum positive moment. Where the trench cannot be located in an area of low moment, the beam should be designed as non-composite.

CHAPTER J

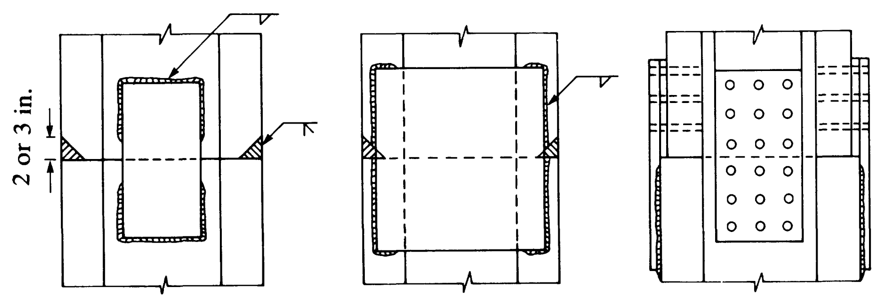
CONNECTIONS, JOINTS AND FASTENERS

J1. GENERAL PROVISIONS

7. Splices in Heavy Sections

Solidified but still hot weld metal contracts significantly as it cools to ambient temperature. Shrinkage of large welds between elements which are not free to move to accommodate the shrinkage causes strains in the material adjacent to the weld that can exceed the yield point strain. In thick material, the weld shrinkage is restrained in the thickness direction as well as in the width and length directions causing triaxial stresses to develop that may inhibit the ability of ductile steel to deform in a ductile manner. Under these conditions, the possibility of brittle fracture increases.

When splicing ASTM Group 4 and 5 rolled sections or heavy welded built-up members, the potentially harmful weld shrinkage strains can be avoided by use of bolted splices or fillet welded lap splices or a splice using a combination welded and bolted detail (Fig. C-J1.1). Details and techniques, that perform well for materials of modest thickness usually must be changed or supplemented by more demanding requirements when welding thick material. Also, the provisions of Structural Welding Code AWS D1.1 are minimum requirements that apply to most structural welding situations; however, when designing and fabricating welded splices of Group 4 and 5 shapes and similar built-up

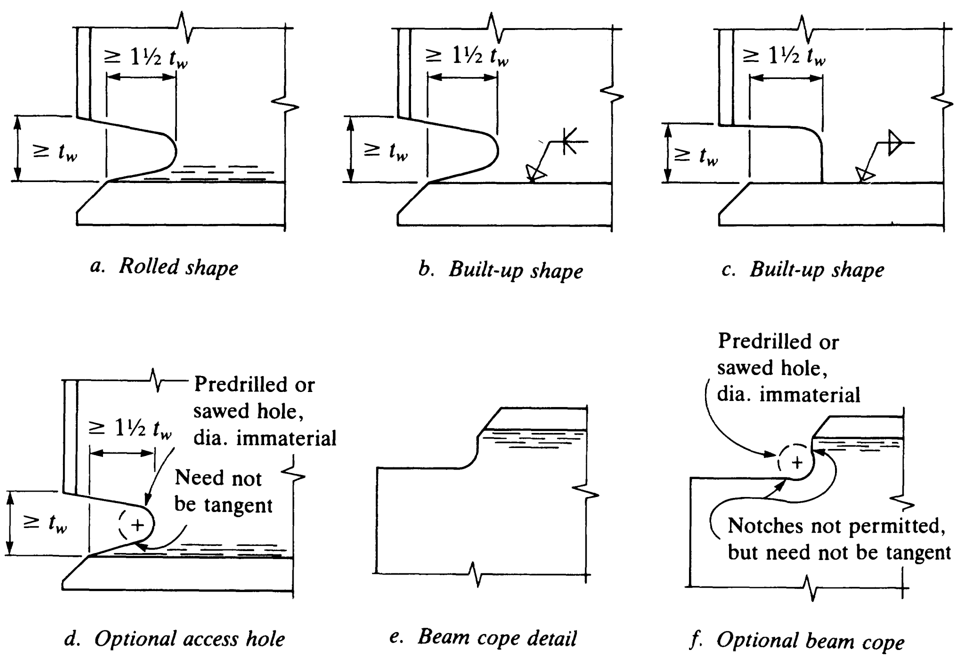


a. Shear plate welded to web *b. Shear plate welded to flange tips* *c. Bolted splice plates*

Fig. C-J1.1. Alternative splices that minimize weld resistant tensile stresses cross sections special consideration must be given to all aspects of the welded splice detail. These are as follows:

- Notch-tough requirements should be specified for tension members. See Commentary A3.1c.
- Generously sized weld access holes (Fig. C-J1.2) are required to provide increased relief from concentrated weld shrinkage strains, to avoid close juncture of welds in orthogonal directions, and to provide adequate clearance for the exercise of high quality workmanship in hole preparation, welding and ease of inspection.
- Preheating for thermal cutting is required to minimize the formation of a hard surface layer.
- Grinding to bright metal and inspection using magnetic particle or dye penetrant methods is required to remove the hard surface layer and to assure smooth transitions free of notches or cracks.

In addition to tension splices of truss chord members and tension flanges of flexural members, other joints fabricated of heavy sections subject to tension should be given special consideration during design and fabrication.



Note: For Group 4 and 5 shapes and welded built-up members made of material more than 2-in. thick, preheat prior to thermal cutting, grind and inspect thermally cut edges using magnetic particle or dye penetrant methods.

Fig. C-J1.2. Weld access hole and beam cope geometry

9. Placement of Welds, Bolts and Rivets

Slight eccentricities between the gravity axis of single- and double-angle members and the center of gravity of their connecting bolts or rivets have long been ignored as having negligible effect upon the static strength of such members. Tests have shown that similar practice is warranted in the case of welded members in statically loaded structures (Gibson and Wake, 1942). However, the fatigue life of single angles, loaded in tension or compression, has been shown to be very short (Koppel and Seeger, 1964).

10. Bolts in Combination with Welds

High-strength bolts used in bearing-type connections should not be required to share load with welds. High-strength bolts used in slip-critical connections, however, because of the rigidity of the connection, may be proportioned to function in conjunction with welds in resisting the transfer of stress across faying surfaces. Because the welds, if installed prior to final tightening of the bolts, might interfere with the development of the high contact pressure between faying surfaces that is counted upon in slip-critical connections, it is advisable that the welds be made after the bolts are tightened. At the location of the fasteners, the heat of welding the connected parts will not alter the mechanical properties of the fasteners.

In making alterations to existing structures, it is assumed whatever slip is likely to occur in high-strength bolted, bearing-type connections will have already taken place. Hence, in such cases the use of welding to resist all stresses other than those produced by existing dead load present at the time of making the alteration is permitted.

J2. WELDS

The requirements of the AWS Code have been adopted by reference, with four exceptions and most requirements governing welding workmanship have been deleted. For convenience of the designer, provisions for allowable design stresses and proportioning of welds have been retained, even though the AISC and AWS provisions are consistent.

The provisions of the AWS *Structural Welding Code* to which exception is taken in the AISC ASD Specification are as follows:

1. Section 2.3.2.4 of the AWS Code and Sect. J2.2a of the AISC ASD Specification both define the effective throat of fillet welds as the shortest distance from the root to the face of the diagrammatic weld. However, for fillet welds made by the submerged arc process, Sect. J2.2a additionally recognizes the deep penetration that is provided by this automatic process at the root of the weld beyond the limits of the diagrammatic weld.
2. Section 2.5 of the AWS Code prohibits the use of partial-penetration welds subject to cyclic tension normal to the longitudinal axis of the weld, whereas the AISC ASD Specification Appendix K4 recognizes partial-penetration welds subject to fatigue loading, but only at the same severely limited stress ranges of Category F that are appropriate to fillet welds.

3. Section 8.13 of the AWS Code provides criteria for the flatness of girder webs, which are arbitrary and based upon a concern for possible cyclic secondary stresses resulting from breathing action of thin girder webs subject to fatigue loading. The AISC ASD Specification does not include such criteria, because lateral deflection or out-of-flatness of webs of girders subject to static loading is of no structural significance. If architectural appearance of exposed girders is of importance, then tolerances based upon specific consideration of architectural requirements, rather than tolerances based upon unrelated consideration of fatigue effects should be provided in the project specification.
4. Section 9 of the AWS Welding Code is applicable to bridges, which are outside the scope of the AISC ASD Specification. Therefore, no comparable provisions are included in the AISC ASD Specification.
5. Section 10 of the AWS Welding Code is applicable to offshore construction, which is outside the scope of the AISC ASD Specification. Therefore, no comparable provisions are included in the AISC ASD Specification.

As in earlier editions, the Specification accepts, without further procedure qualification, numerous weld and joint details executed in accordance with the provisions of the AWS Code. Other welding procedures may be used, provided they are qualified to the satisfaction of the designer and the building code authority and are executed in accordance with the provisions of the AWS Code.

4. Allowable Stresses

The strength of welds is governed by the strength of either the base material or the deposited weld metal.

It should be noted that in Table J2.5 the allowable stress of fillet welds is determined from the effective throat area, whereas the design of the connected parts is governed by their respective thicknesses. Fig. C-J2.1 illustrates the shear planes for fillet welds and base material:

a. Plane 1-1, in which the design is governed by the shear strength for material A
b. Plane 2-2, in which the design is governed by the shear strength of the weld metal
c. Plane 3-3, in which the design is governed by the shear strength of material B

The design of the welded joint is governed by the weakest plane of shear transfer. Note that planes 1-1 and 3-3 are positioned away from the fusion areas between the weld and the base material. Tests have demonstrated that the stress on this fusion area is normally not critical in determining the shear strength of fillet welds. (Preece, 1968) However, if the weld metal is overstrength as might occur when materials with two different strength levels are connected, then the shear plane of the lower strength material at the fusion area may govern. The allowable shear stress on the leg of the weld at the lower strength base metal will be 0.3 times the tensile strength of the base metal.

As in the past, the allowable stresses for statically loaded full-penetration welds are the same as those permitted for the base metal, provided the me-

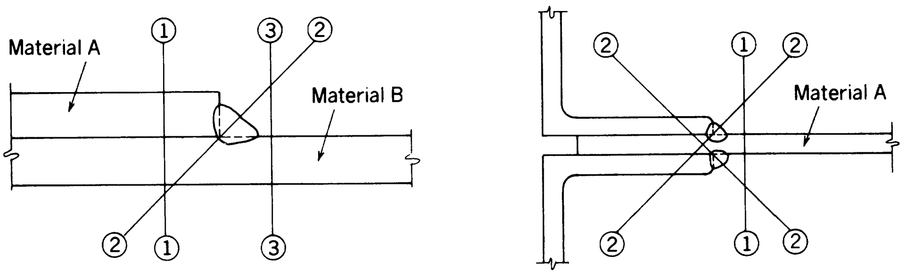


Fig. C-J2.1 Alternative column splices that minimize weld restraint tensile stresses (From: Fisher, J.W. and Pense, A.W. 1987)

chanical properties of the electrodes used are such as to match or exceed those of the weakest grade of base metal being joined.

On the basis of physical tests, the allowable stress on fillet welds deposited on "matching" base metal, or on steel having mechanical properties higher than those specified for such base metal, has been given in terms of the nominal tensile strength* of the weld metal since the 1969 edition of the Specification (Higgins and Preece, 1968).

As in the past, the same allowable value is given to a transverse as to a longitudinal weld, even though the force the former can resist is substantially greater than that of the latter. In the case of tension on the throat of partial-penetration groove welds normal to their axis (more nearly analogous to that of transverse than longitudinal fillets), the allowable stress is conservatively taken the same as for fillet welds.

When partial-penetration groove welds are so disposed that they are stressed in tension parallel to the longitudinal axis of the groove, or primarily in compression or bearing, they may be proportioned to resist such stress at the same unit value permitted in the base metal.

6. Mixed Weld Metal

Instances have been reported in which tack welds deposited using a self-shielded process with aluminum deoxidizers (which by itself provided notch-tough weld metal) were subsequently covered by weld passes using a submerged arc process (which by itself provided notch-tough weld metal) resulted in composite weld metal with low notch-toughness (Terashima and Hart, 1984; Kotecki and Moll, 1970; and Kotecki and Moll, 1972).

J3. BOLTS, THREADED PARTS AND RIVETS

The provisions for mechanical fasteners are based on an extended review and reexamination of the large body of data growing out of voluminous research, which has been completed in the past two decades. In order to consolidate and

*See Commentary Sect. A3.

organize this material and, for the convenience of the engineering profession, to present concise, rational and well balanced conclusions within the covers of a single volume, the Research Council on Structural Connections sponsored the preparation of the 2nd Edition of *Guide to Design Criteria for Bolted and Riveted Joints*, (Kulak, Fisher and Struik, 1987) (in subsequent references this publication will be noted as the "Guide").

The first edition of the Guide was published in 1974 and has provided the background for two revisions of the *Specification for Structural Joints Using ASTM A325 or A490 Bolts* of the Research Council on Structural Connections. The most recent version was approved Sept. 1, 1986. Likewise, it has been the basis for the revision of AISC ASD Specification provisions concerning mechanically fastened structural connections.

At the outset, the Guide notes a distinction between a factor of safety adequate to prevent loss of usefulness of a structure, member, or connection, and one needed to insure against complete failure of these entities. In the latter case, it notes that, under the long-standing misconception of "balanced design," when the weakest element of a joint has a factor of safety of 2, other elements may be grossly overdesigned, with attendant loss in economy (Kulak, Fisher and Struik, 1987).

The balanced design concept may have been valid when there was but one grade of structural steel and but one grade of fastener. However, it has lost its meaning with today's multiplicity of both fastener and connected material strengths.

Based on the earlier criteria, the weakest component in some of the largest and most important joints of existing structures have a factor of safety no greater than 2, yet they have proven with time to be entirely satisfactory. The Guide has adopted this value as basic with respect to failure, increasing it somewhat in rounding off to even working stress values or, as in the case of slip-resistance, reducing it somewhat when impairment of usefulness alone is at stake. With considerable accumulation of data now available as to the effectiveness of joint components under various loading conditions, probabilistic methods of statistical analysis have been used in determining the critical stress to which the factor of safety should be applied (Kulak, Fisher and Struik, 1987).

Provision for the limited use of A449 bolts, in lieu of A325 bolts, is predicated on the fact that the provisions of ASTM A449 concerning quality control are less stringent that those contained in ASTM A325. These bolts differ from A325 bolts only as to reduced size of head and increased length of threading.

4. Allowable Tension and Shear

Allowable stresses for rivets are given in terms applicable to the nominal cross-sectional area of the rivet before driving. For convenience in the proportioning of high-strength bolted connections, allowable stresses for bolts and threaded items are given in terms applicable to their nominal body area, i.e., the area of the threaded part based on its major diameter.

Except as provided in Appendix K4.3, 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 allowable 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 (Kulak, Fisher and Struik, 1987).

Mechanically fastened connections which transmit load by means of shear in their fasteners are categorized as either slip-critical or bearing type. The former depend upon sufficiently high clamping force to prevent slip of the connected parts under anticipated service conditions. The latter depend upon contact of the fasteners against the sides of their holes to transfer the load from one connected part to another.

The amount of clamping force developed by shrinkage of a rivet after cooling and by A307 bolts is unpredictable and generally insufficient to prevent complete slippage at the allowable stress. Riveted connections and connections made with A307 and A449 bolts for shear are treated as bearing-type. The high clamping force produced by properly tightened A325 and A490 bolts is sufficient to assure that slip will not occur at full allowable stress in slip-critical connections and probably will not occur at service loads in bearing-type connections.

The working values given in Table J3.2 for slip-critical and bearing-type shear connections are, with only minor modifications based on reliability analysis of existing data, equivalent to those in previous editions of the AISC ASD Specification for use with A325 and A490 bolts in standard or slotted holes with tight mill scale surfaces.

The requirement of footnote f in Table J3.2, which calls for a 20% reduction in allowable fastener shear stress, as noted in the Guide, is based upon tests on butt-type splice specimens where all connected parts were loaded in tension. This footnote provision would not apply to connection angles at the ends of plate girders which transmit the girder reaction to the supporting member by means of shear in the connection angles. Nor would the distance between extreme fasteners in tension members connected at opposite edges of a gusset plate govern; instead, the length of the connection for each tension member would control the design.

Bearing-type connections are intended for use where service conditions are such that cyclic loading approaching complete stress reversal will not occur, and deformation of the structural frame or a component thereof, due to slip of the connection into bearing, can be tolerated. The allowable stresses in this case are based upon a factor of safety of 2 or more, which over a long period has been found to be adequate. This is substantially higher than that which is basic to the design of the connected members.

The efficiency of threaded fasteners in resisting shear in bearing-type connections is reduced when the threading extends into the shear plane between the connected parts. Except in the case of A307 bolts, two allowable shear stress values are given: one when threading is excluded from the shear plane and one when it is not. In selecting appropriate allowable shear stresses, it was deemed an unwarranted refinement to make a distinction between threads in a single shear plane and threads in two planes (double shear of an enclosed part). Therefore, the allowable stresses were established on the conservative assumption of threads in two planes. Because it is not customary to control this feature in the case of A307 bolts, and because the length of threading on A307 and A449 bolts is greater than on A325 and A490 bolts, it is assumed threading may extend into the shear plane and the allowable shear value, applicable to the gross area, is reduced accordingly.

5. Combined Tension and Shear in Bearing-type Connections

The strength of fasteners subject to combined tension and shear is provided by elliptical interaction curves in Table J3.3 for A325 and A490 bolts, which account for the connection length effect on bolts loaded in shear, the ratio of shear strength to tension strength of threaded fasteners and the ratios of root area to nominal body area and tensile stress area to nominal body area (Yura, 1987). The elliptical interaction curve provides the best estimate of the strength of bolts subject to combined shear and tension and thus is used in this Specification.

6. Combined Tension and Shear in Slip-critical Joints

In the case of slip-critical connections subject to combined tension and shear at the contact surface common to a beam connection and the supporting member, where the fastener tension f_t is produced by moment in the plane of the beam web, the shear component may be neglected in proportioning the fasteners for tension. This is because the shear component assigned to the fasteners subject to direct tensile stress is picked up by the increase in compressive force on the compression side of the beam axis, resulting in no actual shear force on the fasteners in tension.

However, when a slip-critical connection must resist an axially applied tensile force, the clamping force is reduced and F_v must be reduced in proportion to the loss of pretension.

7. Allowable Bearing at Bolt Holes

Bearing values are provided, not as a protection to the fastener (because it needs no such protection) but for the protection of the connected parts. Therefore, the same bearing value applies to joints assembled by bolts, regardless of fastener shear strength or the presence or absence of threads in the bearing area.

It should be noted that the value for bearing stress $1.5F_u$ is the *maximum* allowable value provided deformation around the bolt hole is not a design consideration. As explained under Sects. J3.8 and J3.9 of this Commentary, this maximum value is permitted only if the end distance and intermediate spacing of fasteners, measured in the direction of applied force, are adequate to prevent failure by splitting of a connected part parallel to the line of force at a load less than required to cause transverse fracture through the net area of the part.

Tests have demonstrated that hole elongation greater than 0.25 in. will begin to develop as the bearing stress is increased beyond the values given in Equations (J3-1) and (J3-2), especially if it is combined with high-tensile stress on the net section, even though rupture does not occur. Equation (J3-4) considers the effect of hole ovalization.

Although the possibility of a slip-critical connection slipping into bearing under anticipated service conditions is extremely remote, such connections should comply with the provisions of Sects. J3.4 and J3.8 to insure the usual minimum factor of safety of 2 against complete connection failure.

8. Minimum Spacing

Critical bearing stress is a function of the material tensile strength, the spacing of fasteners, and the distance from the edge of the part to the centerline of the nearest fastener. Tests have shown that a linear relationship exists between the ratio of critical bearing stress to tensile strength of the connected material and the ratio of fastener spacing (in the line of force) to fastener diameter (Kulak, Fisher and Struik, 1987). The following equation affords a good lower bound to published test data for single-fastener connections with standard holes, and is conservative for adequately spaced multi-fastener connections:

$$\frac{F_{pcr}}{F_u} = \frac{l_e}{d} \tag{C-J3-1}$$

where

F_{pcr} = critical bearing stress
F_u = tensile strength of the connected material
l_e = distance, along a line of transmitted force, from the center of a fastener to the nearest edge of an adjacent fastener or to the free edge of a connected part (in the direction of stress), in.
d = diameter of a fastener, in.

This equation, modified by a safety factor of 2, is the basis for Equations (J3-5) and (J3-6).

Along a line of transmitted force, the required spacing center-to-center of standard holes is found from Equation (J3-5). For oversized and slotted holes, this spacing is increased by an increment C_1, given in Table J3.4, providing the same clear distance between holes as for standard holes.

The required edge distance in the direction of stress is found from Equation (J3-6) as the distance from the center of a standard hole to the edge of a connected part. For oversized and slotted holes, this distance is increased by an increment C_2, given in Table J3.6, providing the same clear distance from the edge of the hole as for a standard hole.

The provisions of Sect. J3.8 are concerned with l_e as hole spacings, whereas Sect. J3.9 is concerned with l_e as edge distance L_e in the direction of stress, and Sect. J3.7 establishes a maximum allowable bearing stress. Spacing and/or edge distance may be increased to provide for a required bearing stress, or bearing force may be reduced to satisfy a spacing and/or edge distance limitation.

The critical bearing stress of a single fastener connection is more dependent upon a given edge distance than multi-fastener connections (Jones, 1940). For this reason, longer edge distances (in the direction of force) are required for connections with one fastener in the line of transmitted force than required for those having two or more.

9. Minimum Edge Distance

See Commentary Sect. J3.8.

10. Maximum Edge Distance and Spacing

See Brockenbrough (1983).

11. Long Grips

Provisions requiring a decrease in calculated stress for A307 bolts 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 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 (Bendigo, Hansen and Rumpf, 1963).

J4. ALLOWABLE SHEAR RUPTURE

Tests have shown high-strength-bolted beam end connections which subject a coped web to high bearing stresses may cause a tearing failure mode where a portion of the beam web tears out along the perimeter of the holes (Birkemoe and Gilmor, 1978). The tests demonstrated the failure load can be predicted using an analytical model which combines ultimate shear strength of the net section subject to shear stress with the ultimate tensile strength of the net section subject to tensile stress. More recent research has suggested an alternative approach (Ricles and Yura, 1983 and Hardash and Bjorhovde, 1985).

The block shear failure mode is not limited to coped ends of beams (Fig. C-J4.1). Other examples are shown in Figs. C-J4.2, C-J4.3 and C-J4.4.

There may be similar connections, such as thin bolted gusset plates in double shear, where this type of failure could occur. Such situations should be investigated.

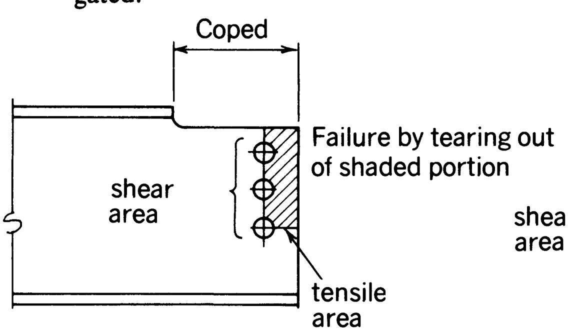


Fig. C-J4.1

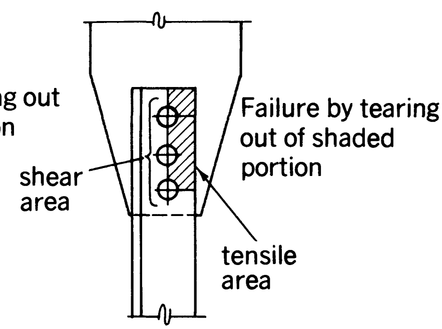


Fig. C-J4.2

J6. FILLERS

The practice of securing fillers by means of additional fasteners, so that they are in effect an integral part of a shear-connected component, is not required where a connection is designed as a slip-critical connection using high-strength bolts. In such connections the resistance to slip between filler and either connected part is comparable to that which would exist between the connected parts if no fill were required.

J8. ALLOWABLE BEARING STRESS

As used throughout the AISC ASD Specification, the terms milled surface, milled, or milling are intended to include surfaces which have been accurately sawed or finished to a true plane by any suitable means. The recommended bearing stress on pins is not the same as for bolts and rivets. The lower value, 9/10 of the yield stress of the part containing the pin hole, provides a safeguard against instability of the plate beyond the hole and high bearing stress concentration which might interfere with operation of the pin, but which is of no concern with bolts and rivets (Johnston, 1939).

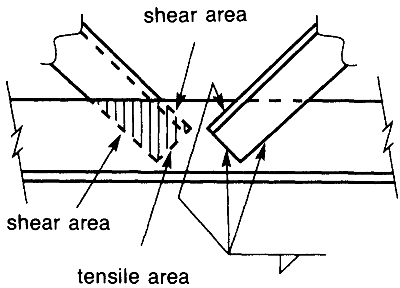


Fig. C-J4.3

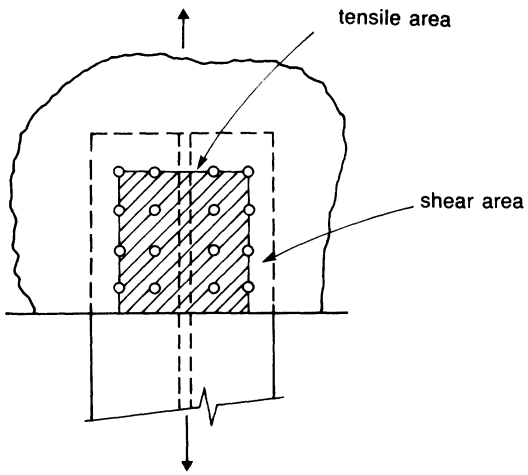


Fig. C-J4.4

J9. COLUMN BASES AND BEARING ON MASONRY AND CONCRETE

It is not the intent of this Specification to prescribe bearing values for masonry materials. The values specified are included to permit a complete design within the scope of this specification, if desired.

The provisions given were derived from ACI Standard 318-83 ultimate strength criteria, using a load factor of 1.7 applied to both live and dead load. These provisions are more conservative than the ACI ultimate strength provisions, wherein a load factor of 1.4 is permitted for dead load.

J10. ANCHOR BOLTS

Shear at the base of a column resisted by bearing of the column base details against the anchor bolts is seldom, if ever, critical. Even considering the lowest conceivable slip coefficient, the vertical load on a column is generally more than sufficient to result in the transfer of any likely amount of shear from column base to foundation by frictional resistance, so that the anchor bolts usually experience only tensile stress. Generally, the largest tensile force for which anchor bolts should be designed is that produced by bending moment at the column base, at times augmented by uplift caused by the overturning tendency of a building under lateral load.

Hence, the use of oversized holes required to accommodate the tolerance in setting anchor bolts cast in concrete, permitted in Sect. J3.2, is not detrimental to the integrity of the supported structure.

CHAPTER K

SPECIAL DESIGN CONSIDERATIONS

K1. WEBS AND FLANGES UNDER CONCENTRATED FORCES

1. Design Basis

Whether or not transverse stiffeners are required on the web of a member opposite the flanges of members rigidly connected to its flanges, as in Fig. C-K1.1, depends on the proportions of these members.

Equation (K1-1) limits the bending stress in the flange of the supporting member. Equation (K1-8) limits the slenderness ratio of an unstiffened web of the supporting member, in order to avoid possibility of its buckling.

When Equation (K1-1) and/or Equation (K1-8) indicate the need for stiffeners; the required area of stiffeners is not given. However, minimum stiffener dimensions are given in Sect. K1.8 and their width-to-thickness ratio must satisfy Sect. B5.

Equation (K1-9), giving the required area of stiffeners when stiffeners are needed, is based on tests supporting the concept that, in the absence of transverse stiffeners, the web and flange thickness of member A should be such that these elements will not yield inelastically under concentrated forces delivered by member B (Graham et al, 1959).

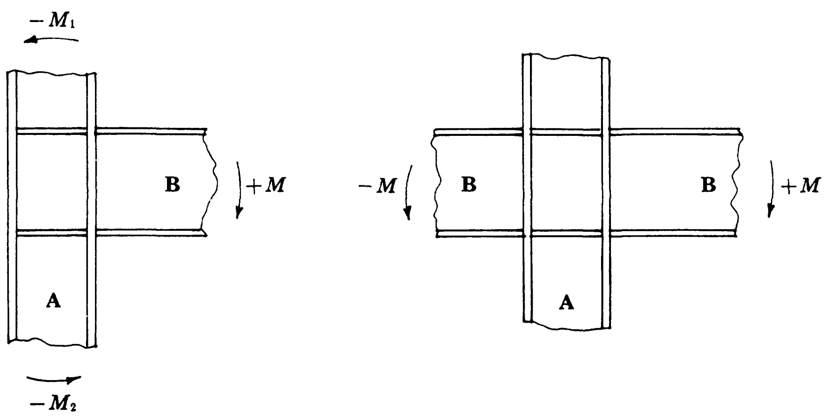


Fig. C-K1.1

3. Local Web Yielding

This web strength criteria has been established to limit the stress in the web member into which a force is being transmitted. 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 2.5 or 5 times the k-distance of the flange, depending upon the location of the load, is limited by Equation (K1-2) or (K1-3) to $0.66F_y$. This represents a change from the past web yield criteria that is consistent with AISC (1986).

4. Web Crippling

The expression for resistance to web crippling at a concentrated load is a departure from previous specifications (IABSE, 1968; Bergfelt, 1971; Hoglund, 1971; and Elgaaly, 1983). Equations (K1-4) and (K1-5) are based on research by Roberts (1981).

5. Sidesway Web Buckling

The sidesway web buckling criteria were developed after observing several unexpected failures in beams (Yura, 1982). In these tests, the compression flanges were braced at the concentrated load, the web was squeezed into compression and the tension flange buckled. (see Fig. C-K1.2).

Sidesway web buckling will not occur in the following cases:

For flanges restrained against rotation:

$$\frac{d_c/t_w}{\ell/b_f} > 2.3 \qquad \text{(C-K1-1)}$$

For flange rotation not restrained:

$$\frac{d_c/t_w}{\ell/b_f} > 1.7 \qquad \text{(C-K1-2)}$$

Sidesway web buckling can also be prevented by the proper design of lateral bracing or stiffeners at the load point. It is suggested that local bracing at both flanges be designed for 1% of the concentrated load applied to that point. Stiffeners must extend from the load point through at least one-half the girder depth. In addition, the pair of stiffeners should be designed to carry the full load. If flange rotation is permitted at the loaded flange, stiffeners will not be effective.

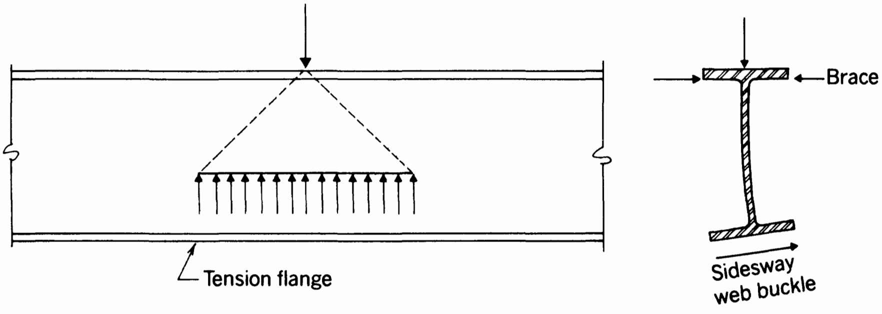


Fig. C-K1.2 Sidesway web buckling

K2. PONDING

As used in the Specification, ponding refers to the retention of water due solely to the deflection of flat roof framing. The amount of this water is dependent upon the flexibility of the framing. If the roof framing members have insufficient stiffness, the water can accumulate and collapse the roof.

Representing the deflected shape of the primary and critical secondary member as a half-sine wave, the weight and distribution of the ponded water can be estimated and, from this, the contribution that the deflection each of these members makes to the total ponding can be expressed (Marino, 1966) as

$$\Delta_w = \frac{\alpha_p \Delta_o \left[1 + \frac{\pi}{4}\alpha_s + \frac{\pi}{4}\rho\,(1 + \alpha_s)\right]}{1 - \frac{\pi}{4}\alpha_p\alpha_s} \tag{C-K2-1}$$

for the primary member, and

$$\delta_w = \frac{\alpha_s \delta_o \left[1 + \frac{\pi^3}{32}\alpha_p + \frac{\pi^2}{8\rho}(1 + \alpha_p) + 0.185\alpha_s\alpha_p\right]}{1 - \frac{\pi}{4}\alpha_p\alpha_s} \tag{C-K2-2}$$

for the secondary member. In these expressions Δ_o and δ_o are, respectively, the primary and secondary beam deflections due to loading present at the initiation of ponding, $\alpha_p = C_p/(1-C_p)$, $\alpha_s = C_s/(1-C_s)$, and $\rho = \delta_o/\Delta_o = C_s/C_p$.

Using the above expression for Δ_w and δ_w, the ratios Δ_w/Δ_o and δ_w/δ_o can be computed for any given combination of primary and secondary beam framing using, respectively, the computed value of parameters C_p and C_s defined in the AISC ASD Specification.

Even on the basis of unlimited elastic behavior, it is seen that the ponding deflections would become infinitely large unless

$$\left(\frac{C_p}{1 - C_p}\right)\left(\frac{C_s}{1 - C_s}\right) < \frac{4}{\pi} \tag{C-K2-3}$$

Because elastic behavior is limited, the effective bending strength available in each member to resist the stress caused by ponding action is restricted to the difference between the yield stress of the member and the stress, f_o, produced by the total load supported by it before consideration of ponding is included.

Noting that elastic deflection is directly proportional to stress, and providing a factor of safety of 1.25 with respect to stress due to ponding, the admissible amount of ponding deflection in either the primary or critical (midspan) secondary member, in terms of the applicable ratio Δ_w/Δ_o or δ_w/δ_o, can be represented as $(0.8F_y - f_o)/f_o$. Substituting this expression for Δ_w/Δ_o and δ_w/δ_o and combining with the foregoing expressions for Δ_w and δ_w, the relationship between critical values for C_p and C_s and the available elastic bending strength to resist ponding is obtained. The curves presented in Figs. C-K2.1 and C-K2.2

are based upon this relationship. They constitute a design aid for use when a more exact determination of required flat roof framing stiffness is needed than given by the Specification provision that $C_p + 0.9C_s \leq 0.25$.

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

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_p \quad \text{for the primary member} \qquad \text{(C-K2-4)}$$

$$U_s = \left(\frac{0.8F_y - f_o}{f_o}\right)_s \quad \text{for the secondary member} \qquad \text{(C-K2-5)}$$

where f_o, in each case, is the computed bending stress in the member due to the supported loading, neglecting ponding effect. Depending upon geographic location, this loading should include such amount of snow as might also be present, although ponding failures have occurred more frequently during torrential summer rains, when the rate of precipitation exceeded the rate of drainage runoff and the resulting hydraulic gradient over large roof areas caused substantial accumulation of water some distance from the eaves.

Given the size, spacing, and span of a tentatively selected combination of primary and secondary beams, for example, one may enter Fig. C-K2.1 at the level of the computed stress index U_p determined for the primary beam; move horizontally to the computed C_s-value of the secondary beams; and, thence, downward to the abscissa scale. The combination stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility constant read from this latter scale is more than the value of C_p computed for the given primary member; if not, a stiffer primary or secondary beam, or combination of both, is required.

If the roof framing consists of a series of equally spaced wall-bearing beams, they would be considered as secondary members, supported on an infinitely stiff primary member. For this case, one would enter Fig. C-K2.2. The limiting value of C_s would be determined by the intercept of a horizontal line representing the U_s-value and the curve for $C_p = 0$.

The ponding deflection contributed by a metal deck is usually such a small part of the total ponding deflection of a roof panel, that it is sufficient merely to limit its moment of inertia (per foot of width normal to its span) to 0.000025 times the fourth power of its span length, as provided in the Specification. However, the stability against ponding of a roof consisting of a metal roof deck of relatively slender depth-span ratio, spanning between beams supported directly on columns, may need to be checked. This can be done using Fig. C-K2.1 or C-K2.2 with the following computed values:

U_p, the stress index for the supporting beam
U_s, the stress index for the roof deck
C_p, the flexibility constant for the supporting beams
C_s, the flexibility constant for one foot width of the roof deck ($S = 1.0$)

Since the shear rigidity of their web system is less than that of a solid plate, the moment of inertia of steel joists and trusses should be taken as somewhat *less* than that of their chords.

K3. TORSION

See AISC (1983).

K4. FATIGUE

Because most members in building frames are not subject to a large enough number of cycles of full allowable stress application to require design for fatigue, the provisions covering such designs have been placed in Appendix K4.

When fatigue is a design consideration, its severity is most significantly affected by the number of load applications, the magnitude of the stress range, and the severity of the stress concentrations associated with the particular details. These factors are not encountered in normal building designs; however, when encountered and when fatigue is of concern, all provisions of Appendix K4 must be satisfied.

Members or connections subject to less than 20,000 cycles of loading will not involve a fatigue condition, except in the case of repeated loading involving large ranges of stress. For such conditions, the admissible range of stress can conservatively be taken as 1½ times the applicable value given in Table A-K4.3 for Loading Condition 1.

Fluctuation in stress which does not involve tensile stress does not cause crack propagation and is not considered to be a fatigue situation. On the other hand, in elements of members subject solely to calculated compression stress, fatigue cracks may initiate in regions of high tensile residual stress. In such situations, the cracks generally do not propagate beyond the region of the residual tensile stress, because the residual stress is relieved by the crack. For this reason, stress ranges that are completely in compression are not included in the column headed by "Kind of Stress" in Table A-K4.2 of Appendix K4. This is also true of comparable tables of the current AASHTO and AREA Specifications.

When fabrication details involving more than one category occur at the same location in a member, the stress range at that location must be limited to that of the most restrictive category. By locating notch-producing fabrication details in regions subject to a small range of stress, the need for a member larger than required by static loading will often be eliminated.

Extensive test programs using full size specimens, substantiated by theoretical stress analysis, have confirmed the following general conclusions (Fisher et al, 1970; Fisher et al, 1974):

1. Stress range and notch severity are the dominant stress variables for welded details and beams.
2. Other variables such as minimum stress, mean stress and maximum stress are not significant for design purposes.

3. Structural steels with yield points of 36 to 100 ksi do not exhibit significantly different fatigue strength for given welded details fabricated in the same manner.

Allowable stress ranges can be read directly from Table A-K4.3 for a particular category and loading condition. The values are based on recent research (Keating and Fisher, 1985). Provisions for A325 and A490 bolts subjected to tension are given in Appendix K4.3.

Tests have uncovered dramatic differences in fatigue life, not completely pre-

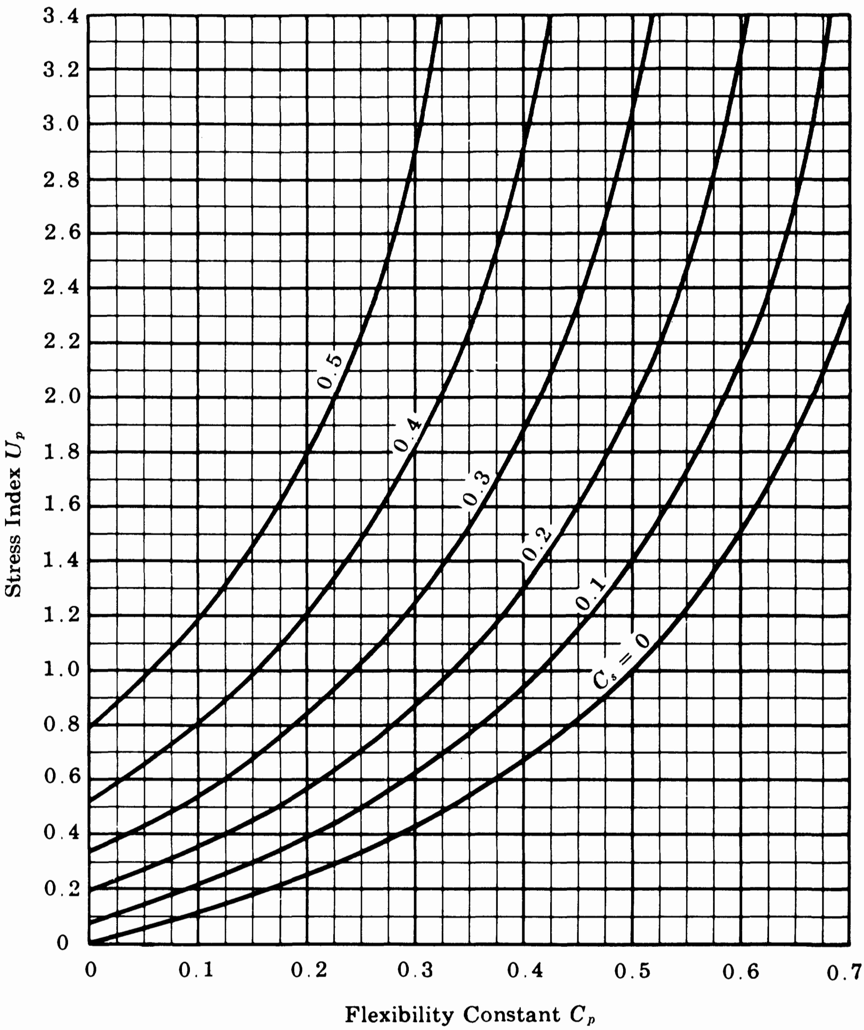


Fig. C-K2.1

dictable from the various published formulas for estimating the actual magnitude of prying force (Kulak, Fisher and Struik, 1987).

The use of other types of mechanical fasteners to resist applied cyclic loading in tension is not permitted. Lacking a high degree of assured pretension, the range of stress is generally too great to resist such loading for long. However, all types of mechanical fasteners survive unharmed when subject to cyclic stresses sufficient to fracture the connected parts, which is provided for elsewhere in Appendix K4.

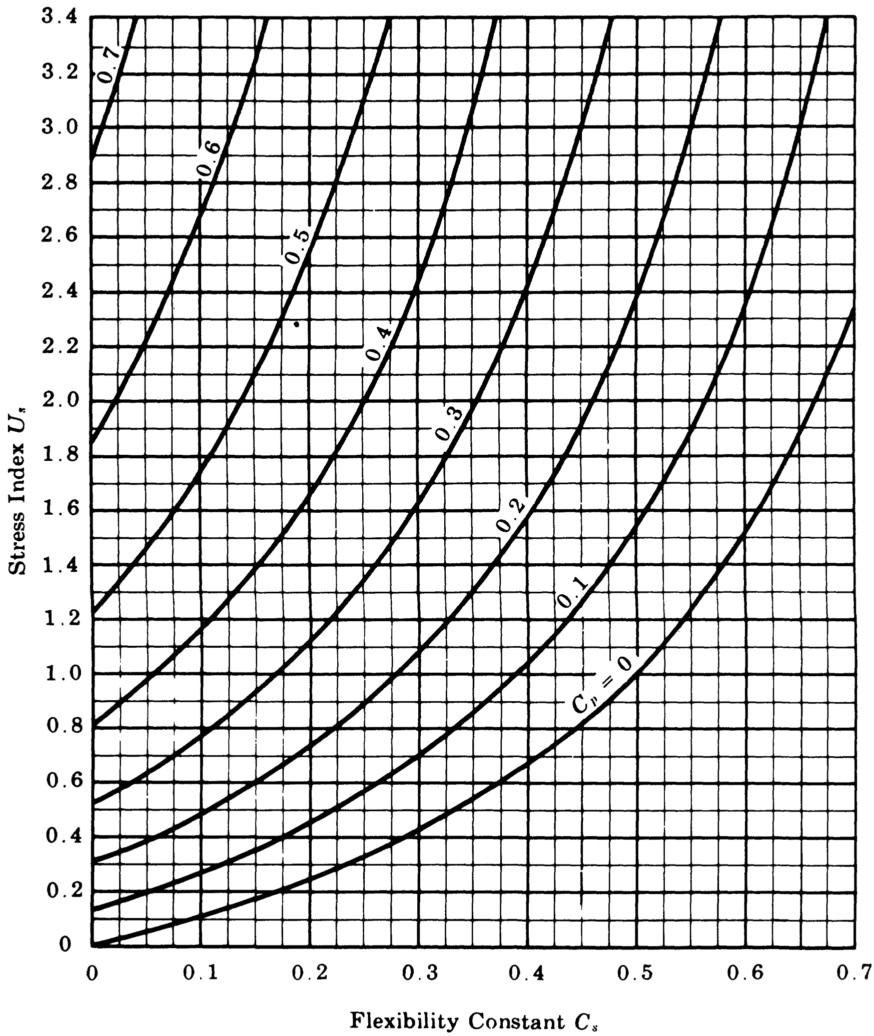


Fig. C-K2.2

CHAPTER L

SERVICEABILITY DESIGN CONSIDERATIONS

L1. CAMBER

The cambering of flexural members, to eliminate the appearance of sagging or to match the elevation of adjacent building components when the member is loaded, is accomplished in various ways. In the case of trusses and girders, the desired curvature can be built in during assembly of the component parts. Within limits, rolled beams can be cold-cambered at the producing mill.

The local application of heat has come into common use as a means of straightening or cambering beams and girders. The method depends upon an ultimate shortening of the heated 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.

Although the final curvature of camber produced by any of these methods can be controlled to a degree, it must be realized that some tolerance, to cover workmanship error and permanent change due to handling, is inevitable.

L2. EXPANSION AND CONTRACTION

As in the case of deflections, the satisfactory control of expansion cannot be reduced to a few simple rules, but must depend largely upon the good judgment of qualified engineers.

The problem is more serious in buildings 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.

L3. DEFLECTION, VIBRATION AND DRIFT

1. Deflection

Although deflection, rather than stress, is sometimes the criterion of satisfactory design, there is no single scale by which the limit of tolerable deflection can be defined. Where limitations on flexibility are desirable, they are often dictated by the nature of collateral building components, such as plastered walls and ceilings, rather than by considerations of human comfort and safety. The admissible amount of movement varies with the type of component.

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

1. The depth of fully stressed beams and girders in floors should, if practicable , be not less than (F_y/800) times the span. If members of less depth are used, the unit stress in bending should be decreased in the same ratio as the depth is decreased from that recommended above.
2. The depth of fully stressed roof purlins should, if practicable, be not less than (F_y/1000) times the span, except in the case of flat roofs.

2. Vibration

Where human comfort is the criterion for limiting motion, as in the case of perceptible vibrations, the limit of tolerable amplitude is dependent on both the frequency of the vibration and the damping effect provided by components of the construction. At best, the evaluation of these criteria is highly subjective, although mathematical models do exist which may be useful (Murray, 1975). When such vibrations are caused by running machinery, they should be isolated by damping devices or by the use of independent foundations.

The depth of a steel beam supporting large open floor areas free of partitions or other sources of damping should not be less than 1/20 of the span to minimize perceptible transient vibration due to pedestrian traffic.

L5. CORROSION

Steel members may deteriorate in particular service environments. This deterioration may appear either in external corrosion, which would be visible upon inspection, or in undetected changes in the material that would reduce its load-carrying capacity. The designer should recognize these problems by either factoring a specific amount of damage tolerance into his design or providing adequate protection systems (e.g., coating, cathodic protection) and/or planned maintenance programs so that such problems do not occur.

CHAPTER M

FABRICATION, ERECTION AND QUALITY CONTROL

M2. FABRICATION

2. Thermal Cutting

Thermal cutting should preferably be done by machine. The requirements for a positive preheat of 150°F minimum when thermal cutting beam copes and weld access holes in ASTM A6 Group 4 and 5 shapes and in built-up shapes made of material more than 2-in. thick tends to minimize the hard surface layer and the possible initiation of cracks.

5. High-strength Bolted Construction - Assembly

In the past, all ASTM A325 and A490 bolts in both slip-critical and bearing-type connections were required to be tightened to a specified tension. The requirement was changed in 1985 to permit some bearing-type connections to be tightened to only a snug-tight condition.

To qualify as a snug-tight bearing connection, the bolts are not subject to tension loads, slip is permitted and loosening or fatigue due to vibration or load fluctuations are not design considerations.

It is suggested that snug-tight bearing-type connections be used in applications when A307 bolts would be permitted. Sect. J1.12 serves as a guide to these applications.

In other cases, A325 and A490 bolts are required to be tightened to 0.7 of their tensile strength. This may be done either by the turn-of-nut method, by a calibrated wrench or by using direct tension indicators (RCSC, 1985). Since fewer fasteners and stiffer connected parts are involved than is generally the case with A307 bolts, the greater clamping force is recommended to ensure solid seating of the connected parts. However, because the performance of bolts in bearing is not dependent on an assured minimum level of pretension, thorough inspection requirements to assure full compliance with pretightening criteria are not warranted. This is especially true regarding the arbitration inspection requirements of Sect 9b of the RCSC Specification (1985). Visual evidence of solid seating of the connected parts, and of wrench impacting to assure that the nut has been tightened sufficiently to prevent it from loosening, is adequate.

M3. SHOP PAINTING

The surface condition of steel framing disclosed by the demolition of long-standing buildings has been found to be unchanged from the time of its erection except at isolated spots where leakage may have occurred. Where such leakage is not eliminated, the presence or absence of a shop coat is of minor influence (Bigos et al, 1954).

The Specification does not define the type of paint to be used when a shop coat is required. Conditions of exposure and individual preferences with regard to finish paint are factors which have a bearing on the selection of the proper primer. Hence, a single formulation would not suffice.*

M4. ERECTION

4. Fit of Column Compression Joints

Tests on spliced, full-sized columns with joints that had been intentionally milled out-of-square, relative to either strong or weak axis, demonstrated their load-carrying capacity was the same as for a similar unspliced column (Popov and Stephen, 1977). In the tests, gaps of 1/16-in. were not shimmed; gaps of 1/4-in. were shimmed with non-tapered mild steel shims. Minimum size partial-penetration welds were used in all tests. No tests were performed on specimens with gaps greater than 1/4-in.

The criteria for fit of column compression joints are equally applicable to joints at column splices and joints between columns and base plates.

*For a comprehensive treatment of the subject, see Ref. 54.

CHAPTER N

PLASTIC DESIGN

N1. SCOPE

The Specification recognizes three categories of profiles, classified according to the ability to resist local buckling of elements of the cross section when subject to compressive stress. The categories are : (1) noncompact, (2) compact, and (3) plastic design. The elements of *noncompact* sections (B5) will not buckle locally when subject to elastic limit strains. Elements of *compact* sections (B5) are proportioned so that the cross section may be strained in bending to the degree necessary to achieve full plastification of the cross section; however, the reserve for inelastic strains is adequate only to achieve modest redistribution of moments. The elements of *plastic design* sections (N7) are proportioned so they will not only achieve full plastification of the cross section, but also will remain stable while being bent through an appreciable angle at a constant plastic moment up to the point where strain hardening is initiated. Thus, plastic design cross sections are capable of providing the hinge rotations that are counted upon in the plastic method of analysis.

Superior bending strength of compact sections is recognized in F1.1 of the Specification by increasing the allowable bending stress to $0.66F_y$ and by permitting 10% redistribution of moment. By the same token, the logical load factor for plastically designed beams is given by the equation

$$F = \frac{F_y}{0.66F_y} \times \text{(shape factor)} \qquad \text{(C-N1-1)}$$

For such shapes listed in the AISC *Manual of Steel Construction*, the variation of shape factor is from 1.10 to 1.23, with a mode of 1.12 for the most commonly used shapes. Then, the corresponding load factor must vary from 1.67 to 1.86, with a mode of 1.70. Such a load factor is consistent and in better balance with that inherent in the allowable working stresses for tension members and deep plate girders.

Research on the ultimate strength of heavily loaded columns subjected to concurrent bending moments has provided data which justifies a load factor, for such members, that is the same as that provided for members subject to bending only, namely 1.7. Consistent with the 1/3 increase in allowable stress permitted in Sect. A5.2 of the Specification, the load factor to be used in designing for gravity loading combined with wind or seismic loading is 1.3 (Van Kuren and Galambos, 1964).

Based on research on multi-story framing, application of the Specification provisions includes the complete design of braced and unbraced planar frames in high-rise buildings (Driscoll et al, 1965; Driscoll, 1966). Systematic procedures for application of plastic design in proportioning the members of such frames have been developed (AISI, 1968; Lu, 1967).

N2. STRUCTURAL STEEL

Research testing has demonstrated the suitability of all of the steels listed in this section for use in plastic design(Adams, Lay and Galambos, 1965; ASCE, 1971).

N3. BASIS FOR MAXIMUM STRENGTH DETERMINATION

Although resistance to wind and seismic loading can be provided in moderate height buildings by means of concrete and masonry shear walls, which also provide for overall frame stability at factored gravity loading, taller building frames must provide this resistance acting alone. This can be achieved in one of two ways: either by a system of bracing or by a moment-resisting frame.

For one- and two-story unbraced frames with Type 1 construction throughout, where the column axial loads are generally modest, the frame instability effect is small and $P\Delta$ effects* may be safely ignored. However, where such frames are designed with a mixture of rigid connections and simple or semi-rigid connections (Type 2 and Type 3 construction), it may be necessary to consider the frame instability effect $P\Delta$. In this case, stability is dependent upon a reduced number of rigid connections and the effect of frame drift may be a significant consideration in the design.

1. Stability of Braced Frames

The limitation on axial force $0.85P_y$ was inserted as a simple means to compensate for three possible effects (Douty and McGuire, 1965):

1. Loss of stiffness due to residual stress
2. Effect of secondary $P\Delta$ moments on the vertical bracing system
3. Lateral-torsional buckling effect

N4. COLUMNS

Equations (N4-2) and (N4-3) will be recognized as similar in type to Equations (H1-1) and (H1-2), except they are written in terms of factored loads and moments, instead of allowable stresses at service loading. As in the case of Equations (H1-1) and (H1-2), P_{cr} is computed on the basis of l/r_x or l/r_y, whichever is larger, for any given unbraced length (Driscoll et al, 1965).

A column is considered to be fully braced if the slenderness ratio l/r_y between the braced points is less than or equal to that specified in Sect. N9. When the unbraced length ratio of a member bent about its strong axis exceeds the limit specified in Sect. N9, the rotation capacity of the member may be impaired, due to the combined influence of lateral and torsional deformation, to such an extent that plastic hinge action within the member cannot be counted upon. However, if the computed value of M is small enough so limitations of Equations (N4-2) and (N4-3) are met, the member will be strong enough to function at a joint where the required hinge action is provided in another member entering the joint. An assumed reduction in moment-resisting capacity is provided by using the value M_m, computed from Equation (N4-4), in Equation (N4-2).

*See Commentary C2 for discussion of $P\Delta$ effects.

Equation (N4-4) was developed empirically on the basis of test observations and provides an estimate of the critical lateral buckling moment, in the absence of axial load, for the case where $M_1/M_2 = -1.0$ (single curvature bending) (Driscoll et al, 1965). For other values of M_1/M_2, adjustment is provided by using the appropriate C_m value as defined in Sect. H1.

Equation (N4-4) is to be used only in connection with Equation (N4-2).

Space frames containing plastically designed planar rigid frames are assumed to be supported against sidesway normal to these frames. Depending upon other conditions of restraint, the basis for determination of proper values for P_{cr} and P_e and M_m, for a plastically designed column oriented to resist bending about its strong axis, is outlined in Table C-N4.1. In each case l is the distance between points of lateral support corresponding to r_x or r_y, as applicable. When K is indicated, its value is governed by the provisions of Sect. C2.2.

N5. SHEAR

Using the von Mises criterion, the average stress at which an unreinforced web would be fully yielded in pure shear can be expressed as $F_y/\sqrt{3}$. It has been observed that the plastic bending strength of an I-shaped beam is not reduced appreciably until shear yielding occurs over the full effective depth, which may be taken as the distance between the centroids of its flanges (approximately 0.95 times its actual depth) (ASCE, 1971). Thus,

$$V_u = \frac{F_y}{\sqrt{3}} \times 0.95dt = 0.55F_y dt_w \quad \text{(C-N5-1)}$$

Shear stresses are generally high within the boundaries of a rigid connection of two or more members whose webs lie in a common plane. Assuming the moment $+M$, in Fig. C-N5.1, expressed in kip-ft, to be resisted by a force couple acting at the centroid of the beam flanges, the shear, in kips, produced in beam-to-column connections web *abcd* can be computed as

$$V = \frac{12M}{0.95d_b} - V_s \quad \text{(C-N5-2)}$$

when $V = 0.55F_y d_c t_w$

$$\text{req'd } t_w = \frac{1}{0.55F_y d_c}\left[\frac{12M}{0.95d_b} - V_s\right] \quad \text{(C-N5-3)}$$

TABLE C-N4.1

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

[1]Webs of columns assumed to be in plane of frame.

where A_{bc} is the planar area *abcd* and F_y is expressed in ksi. If the thickness of the web panel is less than that given by this formula, the deficiency may be compensated for by a pair of diagonal stiffeners or by a reinforcing plate in contact with the web panel and welded around its boundary to the column flanges and horizontal stiffeners.

N6. WEB CRIPPLING

Usually stiffeners are needed, as *ab* and *dc* in Fig. C-N5.1, in line with the flanges of a beam rigidly connected to the flange of a second member so located that their webs lie in the same plane 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 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, because of the lack of bending stiffness in the flange to which the beam is connected. Since their design is based on equating the plastic resisting capacity of the supporting member to the plastic moment delivered by the supported member, Equations (K1-1), (K1-8) and (K1-11) are equally applicable to allowable stress design and plastic design.

When stiffeners are required, as an alternative to the usual pair of horizontal plates, vertical plates parallel to but separated from the web, as shown in Fig. C-N6.1, may prove advantageous. See Graham et al., 1959.

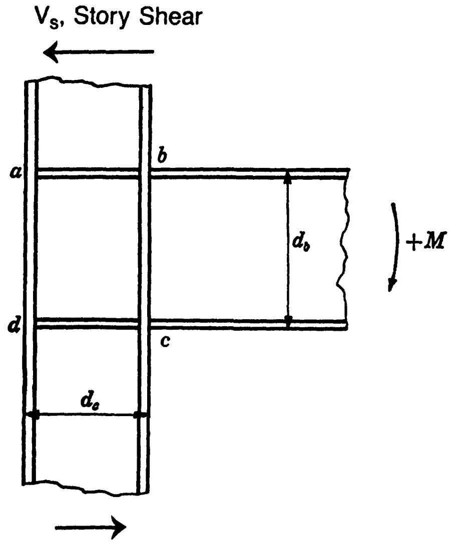


Fig. C-N5.1

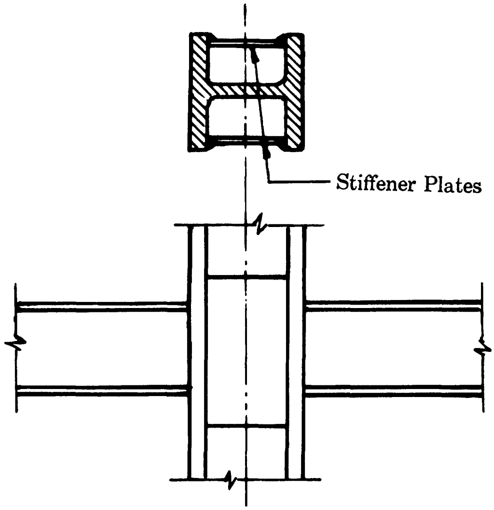


Fig. C-N6.1

N7. MINIMUM THICKNESS (WIDTH-THICKNESS RATIOS)

Research has shown the limiting flange and web width-thickness ratios, below which ample plastic hinge rotations could be relied upon without reduction in the M_p value due to local buckling, are not exactly proportional to $1/\sqrt{F_y}$, although the discrepancy using such a relationship, within the range of yield stress presently permitted by the Specification, is not large (ASCE, 1971). Expressions including other pertinent factors are complex and involve use of mechanical properties that have not been defined clearly. Tabular values for limiting flange width-thickness ratios are given in the Specification for the approved grades of steel.

No change in basic philosophy was involved in extending the earlier expression for limiting web depth-thickness ratio to stronger steels. Equations (N7-1) and (N7-2) were derived, with minor adjustments for better correlation with observed test results, by multiplying Equation (25) of the 1963 Specification by the factor $\sqrt{36/F_y}$ to cover the accepted range in yield point stress. Equation (N7-1) is identical to Equation (1.5-4) in Part 1 of the 1969 Specification, except that it is written in terms of factored loads instead of allowable stresses at service loading. Equation (1.5-4) in the 1969 Specification was liberalized in 1974 and redesignated as Equation (1.5-4a) in the 1978 Specification; in this Specification it is included in Table B5.1. However, this liberalization was not extended to plastic design sections, which require greater rotational capacity than compact sections.

N8. CONNECTIONS

Connections located outside of regions where hinges would have formed at ultimate load can be treated in the same manner as similar connections in frames designed in accordance with the provisions of Chaps. A through M. Since the moments and forces to be resisted will be those corresponding to the factored loading, the allowable stresses to be used in proportioning parts of the connections can be taken as 1.7 times those given in J.

The same procedure is valid in proportioning connections located in the region of a plastic hinge. Connections required to resist moments and forces due to wind and earthquake loads combined with gravity loading factored to 1.3, and proportioned on the basis of limiting stresses equal to 1.7 times those given in J, provide a balance between frame strength and connection strength, provided they are adequate to resist gravity loading alone, factored to 1.7.

The width-thickness ratio and unbraced length of all parts of the connection that would be subject to compression stresses in the region of a hinge must meet the requirements given in N, and sheared edges and punched holes must not be used in portions of the connection subject to tension.

When a haunched connection is proportioned elastically for the moments that would exist within its length, the continuous frame can be analyzed as a mechanism having a hinge at the small end of the haunch, rather than at the intersection point between connected members, with some attendant economy (ASCE, 1971).

Tests have shown that splices assembled with high-strength bolts are capable of developing the M_p value of the gross cross section of the connected part (Douty and McGuire, 1965). It has also been demonstrated that beam-to-column connections involving use of welded or mechanically fastened fittings, instead of full-penetration groove welds matching the full member cross section, not only are capable of developing the M_p value of the member, but that the resulting hinge rotation can be reversed several times without failure (Popov and Pinkney, 1968).

N9. LATERAL BRACING

Portions of members that would be required to rotate inelastically as a plastic hinge in reducing a continuous frame to a mechanism at ultimate load, need more bracing than similar parts of a continuous frame designed in accordance with the elastic theory. Not only must they reach yield point at a load factor of 1.7, they must also strain inelastically to provide the necessary hinge rotation. This is not true at the last hinge to form, since the factored load is assumed to have been reached when this hinge starts to rotate. When bending takes place about the strong axis, any I-shape member tends to buckle out of the plane of bending. For this reason, 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 the problem is less severe, since hinge rotation is not involved.

The Specification provisions governing unbraced length are based upon research on members with moment gradients (ASCE, 1971).

In keeping with similar usage of the parameter M/M_p in Chap. H of the Specification, the sign convention adopted in Equations (N9-1) and (N9-2) is generally found more convenient in frame analysis, namely that clockwise moments about a fixed point are positive and counterclockwise moments are negative.

APPENDIX B

DESIGN REQUIREMENTS

B5. LOCAL BUCKLING

2. Slender Compression Elements*

*See Commentary Chap. B for the discussion of provisions for Slender Compression Elements.

APPENDIX F

BEAMS AND OTHER FLEXURAL MEMBERS

F7. WEB-TAPERED MEMBERS

The provisions contained in Appendix F7 cover only those aspects of the design of tapered members that are unique to tapered members. For other criteria of design not covered specifically in Appendix F7, see the appropriate portions of Chaps. A through M.

3. Allowable Compressive Stress

The approach in formulating $F_{a\gamma}$ of tapered columns is based on the concept that the critical stress for an axially loaded tapered column is equal to that of a prismatic column of different length, but of the same cross section as the smaller end of the tapered column. This resulted in an equivalent effective length factor K_γ for a tapered member subjected to axial compression (Lee, Morrell and Ketter, 1972). This factor, used to determine the value of S in Equations (A-F7-2) and (A-F7-3), can be determined accurately for a symmetrical rectangular rigid frame composed of prismatic beams and tapered columns.

With modifying assumptions, such a frame can be used as a mathematical model to determine, with sufficient accuracy, the influence of the stiffness $\Sigma(I/b)_g$ of beams and rafters which afford restraint at the ends of a tapered column in other cases, such as those shown in Fig. C-A-F7.1. From Equations (A-F7-2) and (A-F7-3), the critical load P_{cr} can be expressed as $\pi^2 EI_o/(K_\gamma l)^2$. The value of K_γ can be obtained by interpolation, using the appropriate chart (Figs. C-A-F7.2 to C-A-F7.17), and restraint modifiers G_T and G_B. In each of these modifiers the tapered column, treated as a prismatic member having a moment of inertia I_o computed at the smaller end, and its actual length l, is assigned the stiffness I_o/l, which is then divided by the stiffness of the restraining members at the end of the tapered column under consideration. Such an approach is well documented.

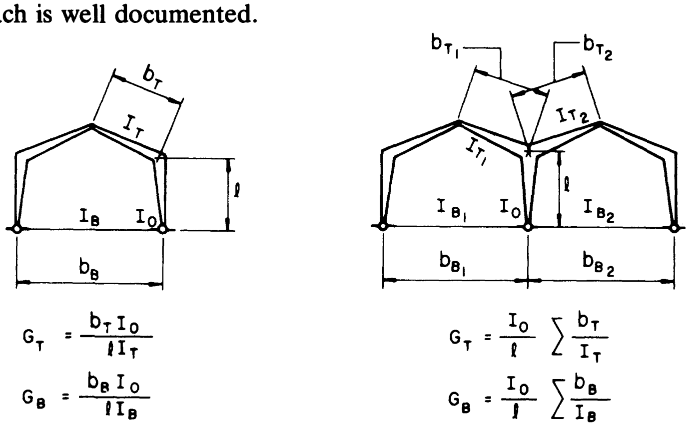


Fig. C-A-F7.1

4. Allowable Flexural Stress

The development of the allowable flexural stress for tapered beams follows closely with that for prismatic beams. The basic concept is to replace a tapered beam by an equivalent prismatic beam with a different length, but with a cross section identical with that of the smaller end of the tapered beam (Lee, Morrell and Ketter, 1972). This has led to the modified length factors h_s and h_w in Equations (A-F7-4) and (A-F7-5).

Equations (A-F7-4) and (A-F7-5) are based on total resistance to lateral buckling using both St. Venant and warping resistance. The factor B modifies the basic $F_{b\gamma}$ to account for moment gradient and lateral restraint offered by adjacent segments. For members which are continuous past lateral supports, categories a, b and c of Appendix F7.4 usually apply; however, note they apply only when the axial force is small and adjacent unbraced segments are approximately equal in length. For a single member, or segments which do not fall into category a, b, c or d, the recommended value for B is unity. The value of B should also be taken as unity when computing the value of $F_{b\gamma}$ to be used in Equation (A-F7-12), since the effect of moment gradient is provided for by the factor C_m. The background material is given in WRC Bulletin No. 192 (Morrell and Lee, 1974).

Thus, note that in these charts the values of K_γ represent the combined effects of end restraints and tapering. For the case $\gamma = 0$, K_γ becomes K, which can also be determined from the alignment chart for effective length of columns in continuous frames (Fig. C-C2.2). For cases when the restraining beams are also tapered, the procedure used in WRC Bulletin No. 173 can be followed, or appropriate estimation of K_γ can be made based on these charts (Lee et al. 1972).

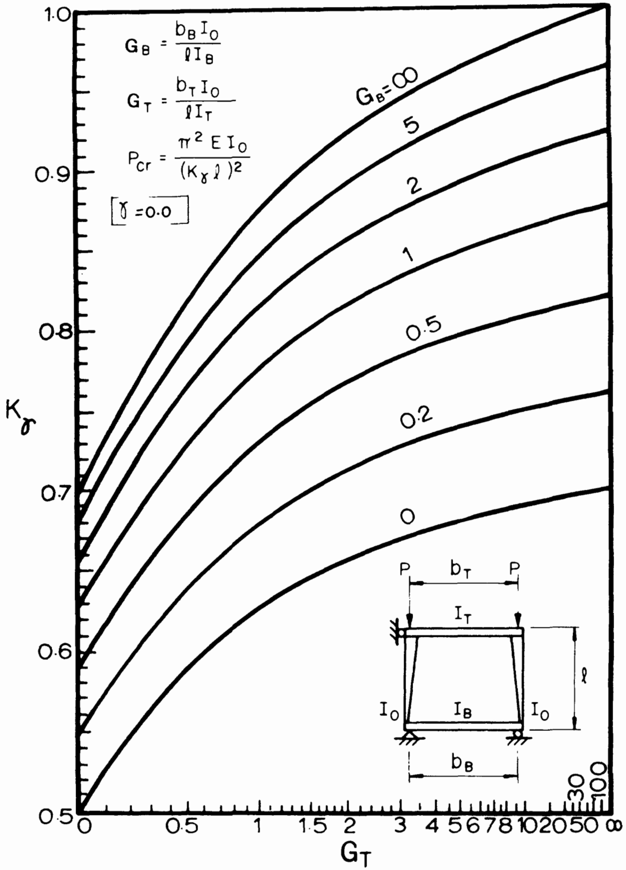


Fig. C-A-F7.2. Effective length factors for tapered columns: sidesway prevented ($\gamma = 0$)

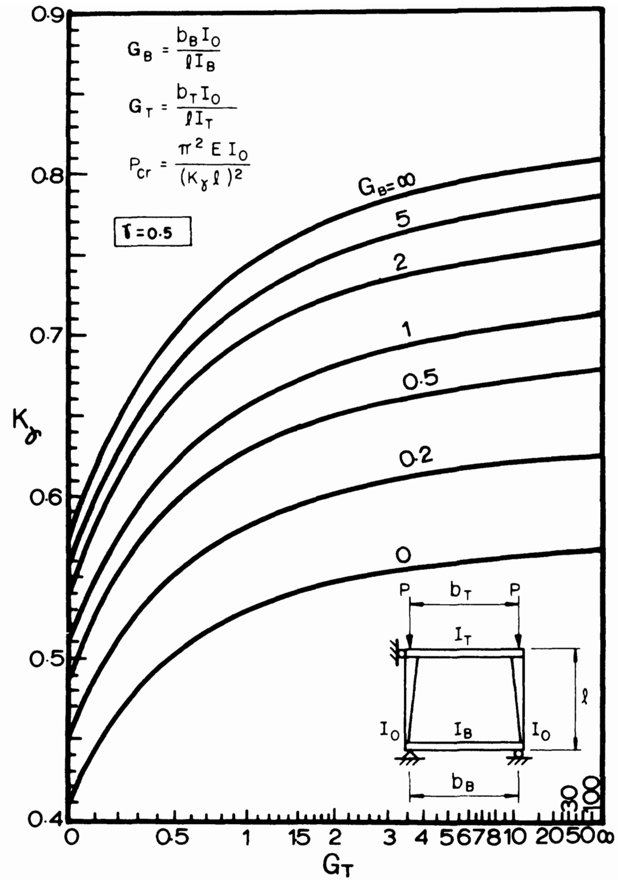


Fig. C-A-F7.3. Effective length factors for tapered columns: sidesway prevented ($\gamma = 0.5$)

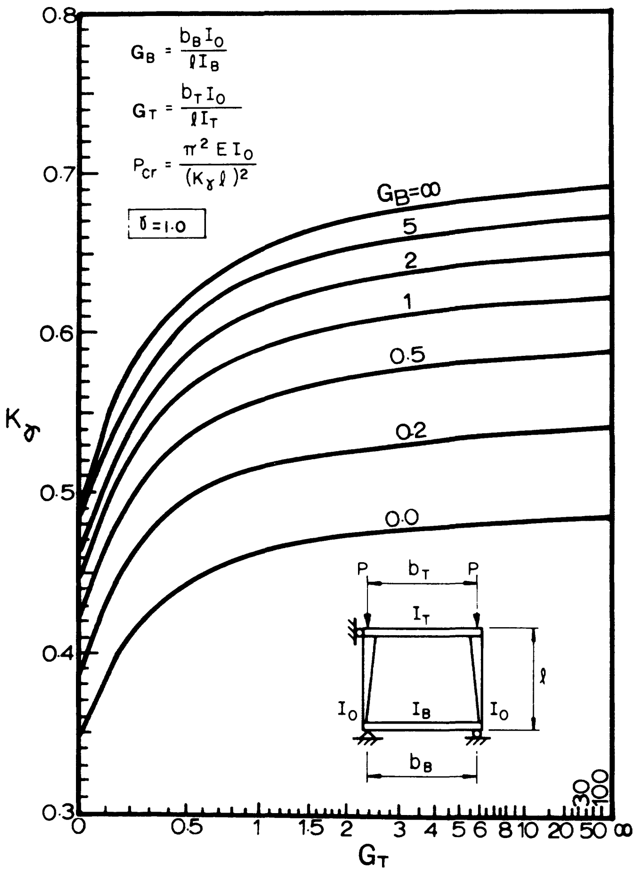


Fig. C-A-F7.4. Effective length factors for tapered columns: sidesway prevented (γ = 1.0)

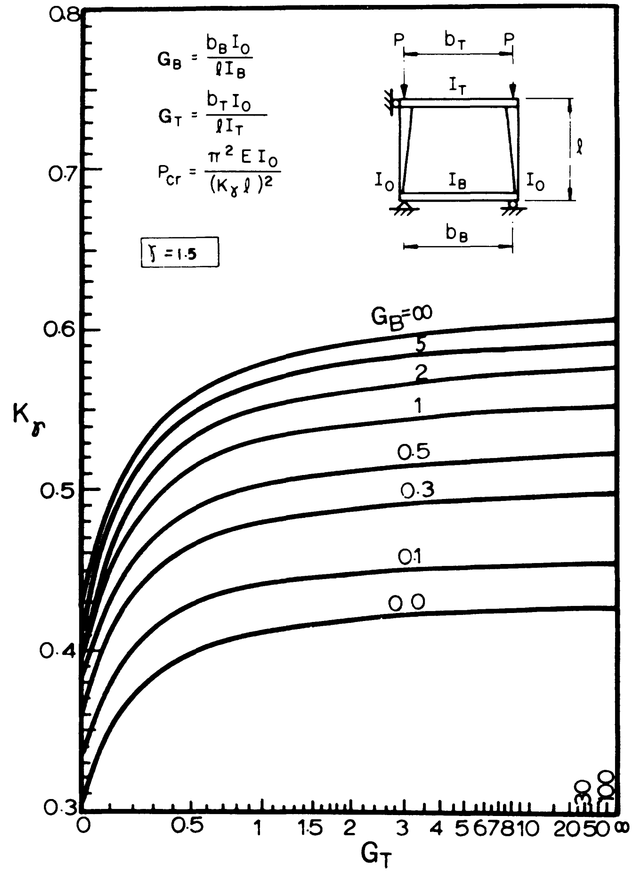


Fig. C-A-F7.5. Effective length factors for tapered columns: sidesway prevented (γ = 1.5)

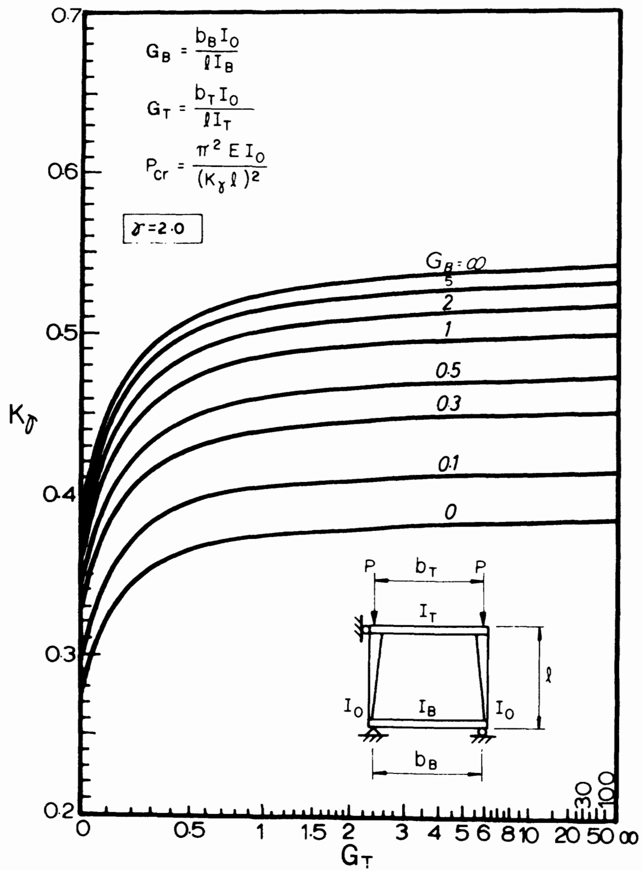


Fig. C-A-F7.6. Effective length factors for tapered columns: sidesway prevented (γ = 2.0)

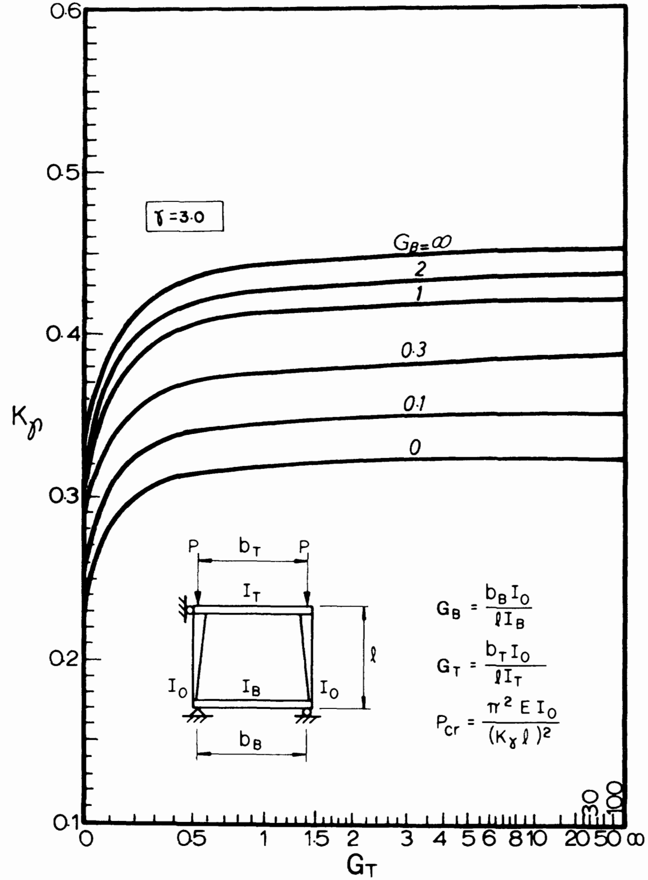


Fig. C-A-F7.7. Effective length factors for tapered columns: sidesway prevented (γ = 3.0)

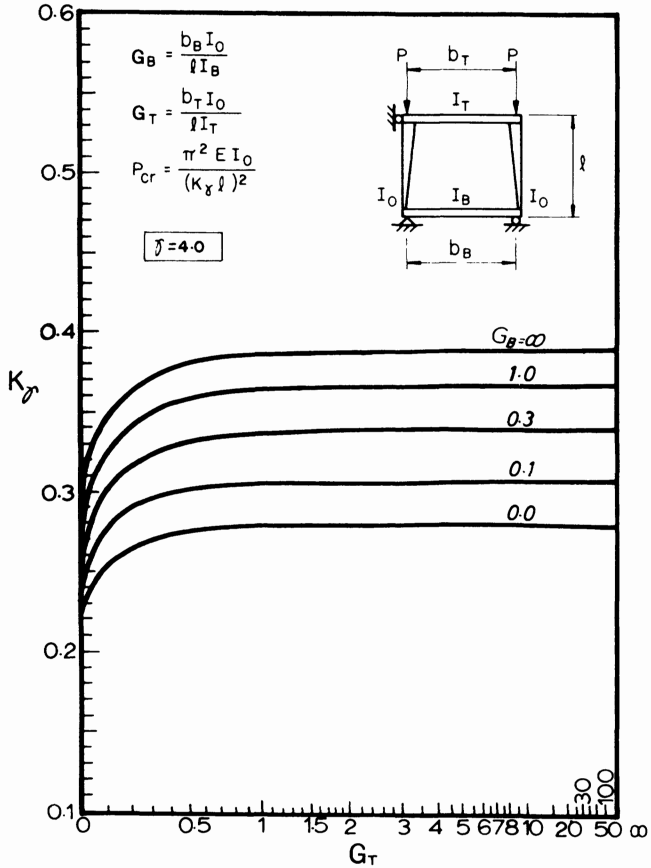


Fig. C-A-F7.8. Effective length factors for tapered columns: sidesway prevented (γ = 4.0)

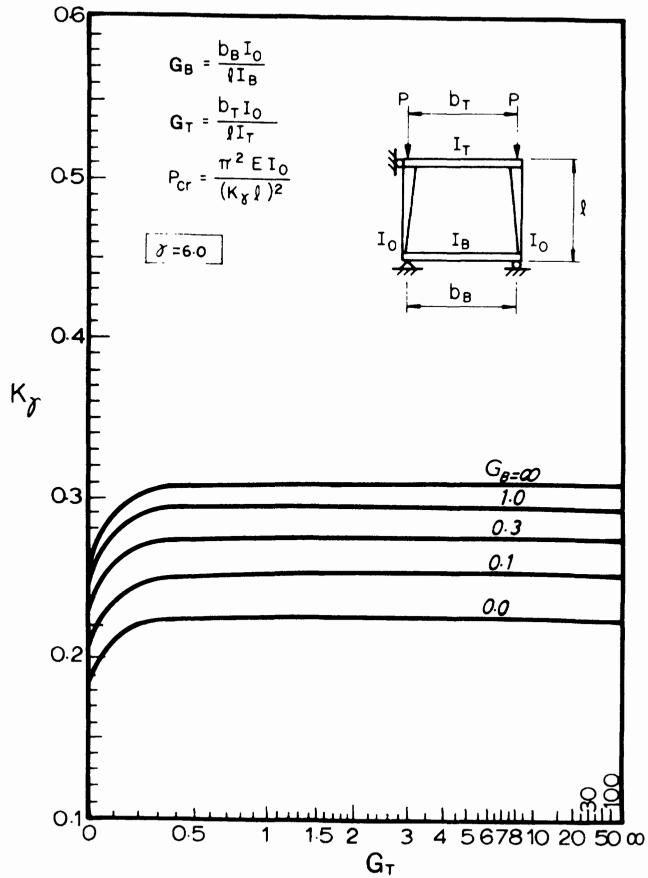


Fig. C-A-F7.9. Effective length factors for tapered columns: sidesway prevented (γ = 6.0)

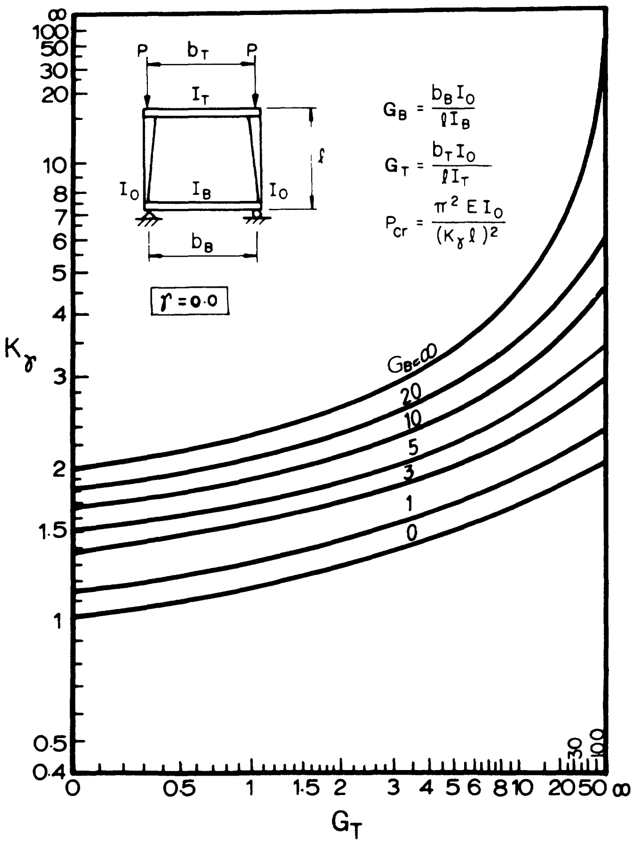


Fig. C-A-F7.10. Effective length factors for tapered columns. sidesway permitted ($\gamma = 0$)

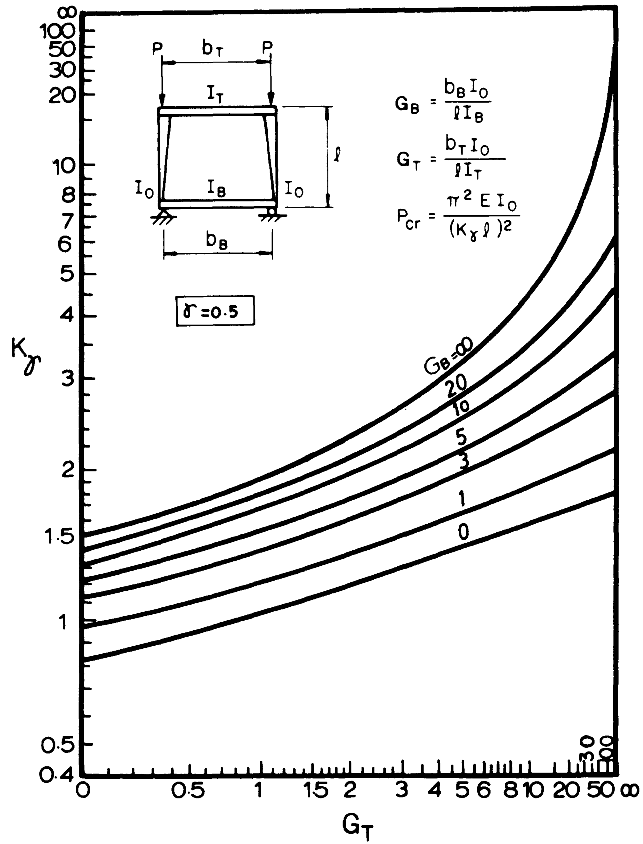


Fig. C-A-F7.11. Effective length factors for tapered columns: sidesway permitted ($\gamma = 0.5$)

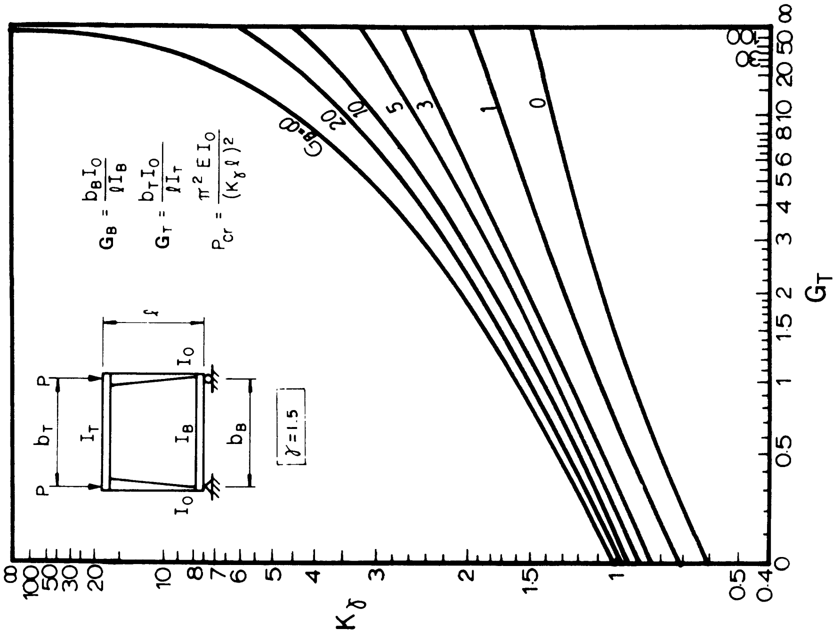


Fig. C-A-F7.13. Effective length factors for tapered columns: sidesway permitted (γ = 1.5)

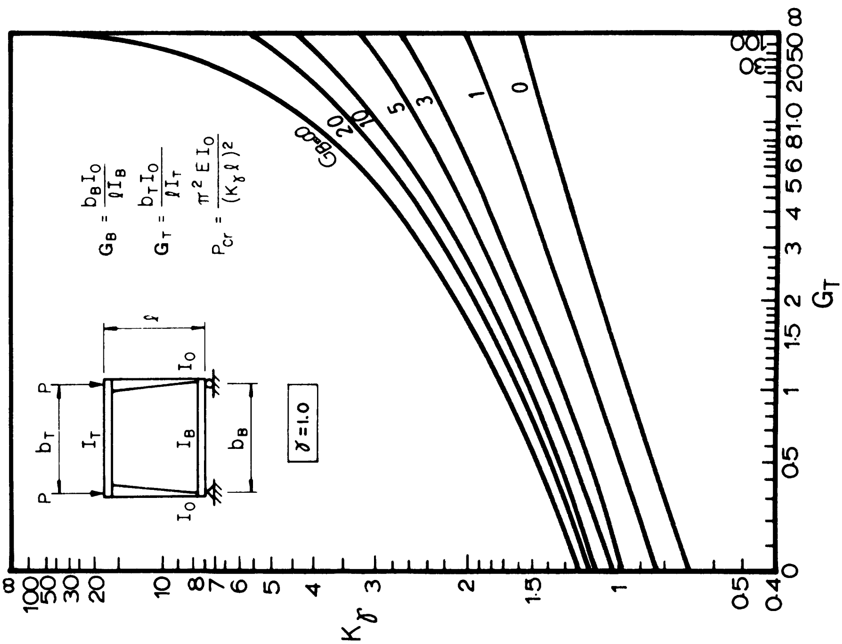


Fig. C-A-F7.12. Effective length factors for tapered columns: sidesway permitted (γ = 1.0)

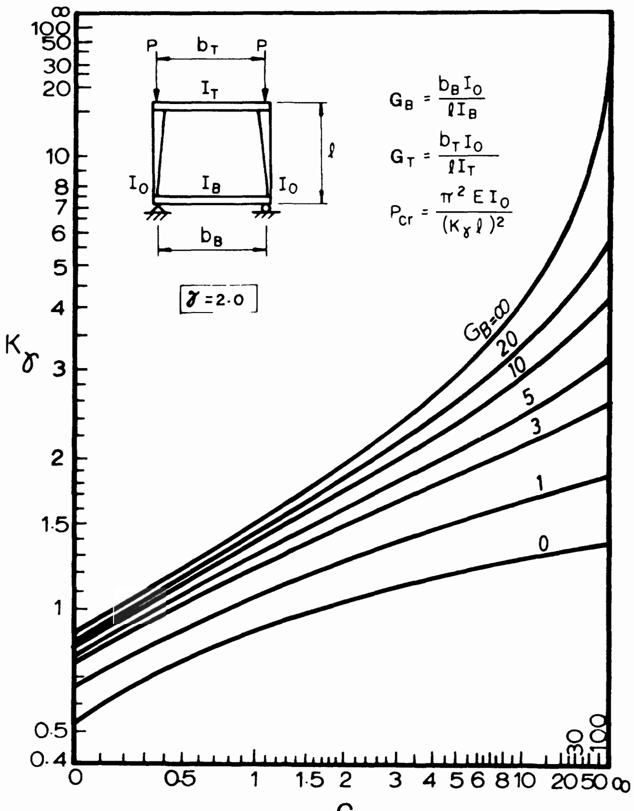


Fig. C-A-F7.14. Effective length factors for tapered columns: sidesway permitted (γ = 2.0)

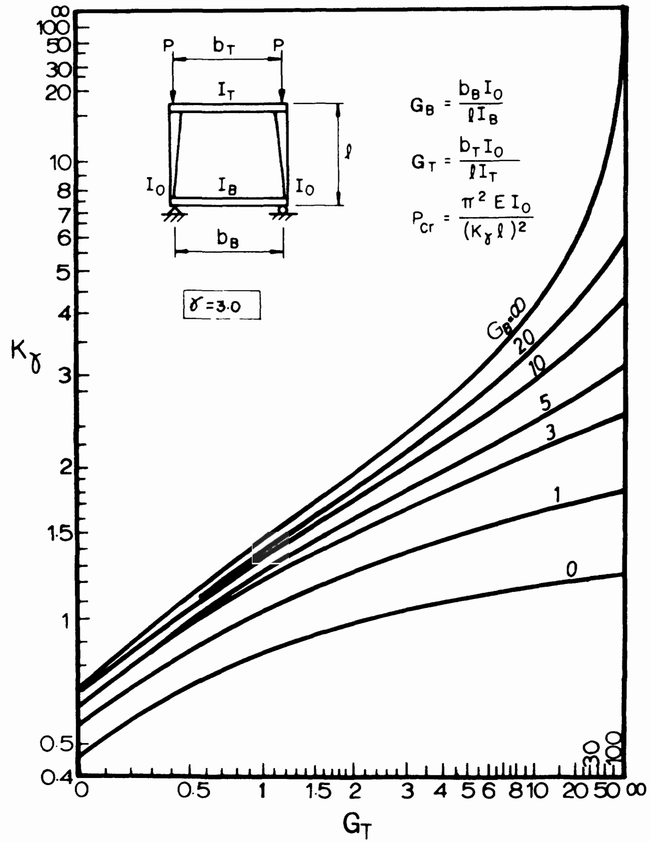


Fig. C-A-F7.15. Effective length factors for tapered columns: sidesway permitted (γ = 3.0)

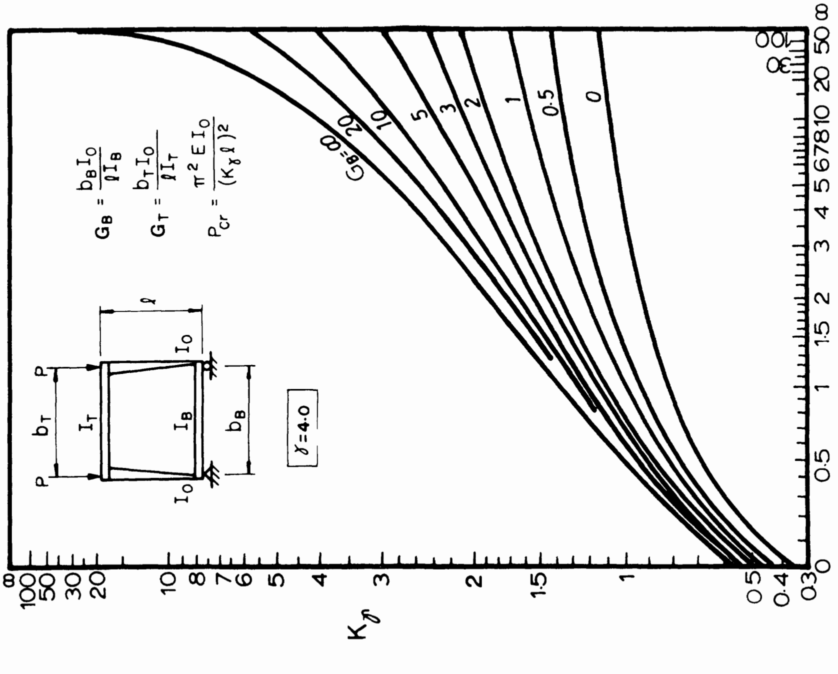


Fig. C-A-F7.16. Effective length factors for tapered columns: sidesway permitted (γ = 4.0)

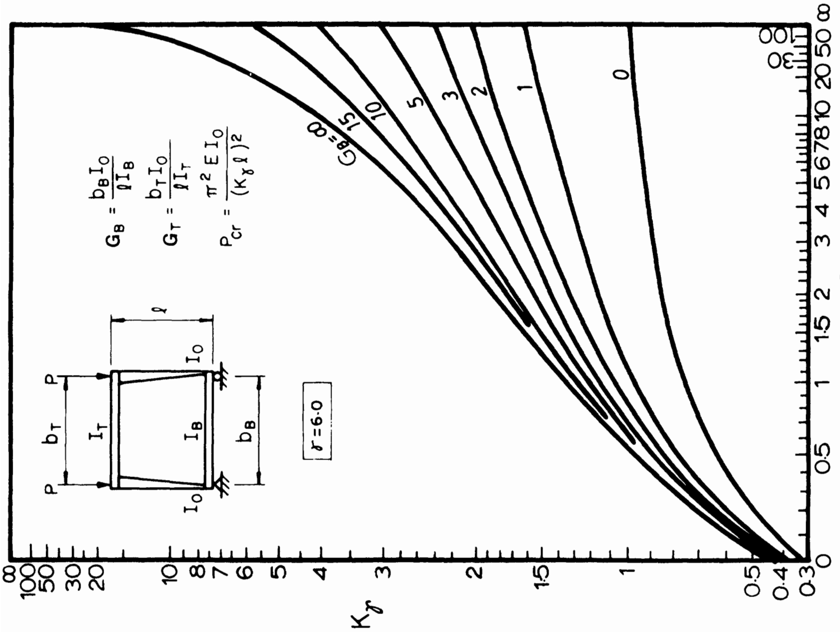


Fig. C-A-F7.17. Effective length factors for tapered columns: sidesway permited (γ = 6.0)

SYMBOLS

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

- A Gross area of an axially loaded compression member, in.^{2} (N4)
- A_b Nominal body area of a fastener, in.^{2} (J3.5); area of an upset rod based upon the major diameter of its threads, i.e., the diameter of a coaxial cylinder which would bound the crests of the threads, in.^{2} (J3.4)
- A_c Actual area of effective concrete flange in composite design, in.^{2} (I4)
- A_e Effective net area of an axially loaded tension member, in.^{2} (B3)
- A_f Area of compression flange, in.^{2} (F1.1)
- A_{fe} Effective tension flange area, in.^{2} (B10)
- A_{fg} Gross beam flange area, in.^{2} (B10)
- A_{fn} Net beam flange area, in.^{2} (B10)
- A_g Gross area of member, in.^{2} (B3)
- A_n Net area of an axially loaded tension member, in.^{2} (B2)
- A_s Area of steel beam in composite design, in.^{2} (I4)
- A'_s Area of compressive reinforcing steel, in.^{2} (I4)
- A_{sr} Area of reinforcing steel providing composite action at point of negative moment, in.^{2} (I4)
- A_{st} Cross-sectional area of stiffener or pair of stiffeners, in.^{2} (G4)
- A_t Net tension area, in.^{2} (J4)
- A_v Net shear area, in.^{2} (J4)
- A_w Area of girder web, in.^{2} (G2)
- A_1 Area of steel concentrically bearing on a concrete support, in.^{2} (J9)
- A_2 Maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2} (J9)
- B Bending coefficient dependent upon computed moment or stress at the ends of unbraced segments of a tapered member (Appendix F7.4)
- C_a Coefficient used in Table 4 of Numerical Values
- C_b Bending coefficient dependent upon moment gradient (F1.3)
- C_c Column slenderness ratio separating elastic and inelastic buckling (E2)
- C'_c Slenderness ratio of compression elements (Appendix B5.2)
- C_h Coefficient used in Table 12 of Numerical Values
- C_m Coefficient applied to bending term in interaction equation for prismatic members and dependent upon column curvature caused by applied moments (H1)
- C'_m Coefficient applied to bending term in interaction equation for tapered members and dependent upon axial stress at the small end of the member (Appendix F7.6)
- C_p Stiffness factor for primary member in a flat roof (K2)

C_s Stiffness factor for secondary member in a flat roof (K2)

C_v Ratio of "critical" web stress, according to the linear buckling theory, to the shear yield stress of web material (F4)

C_1 Increment used in computing minimum spacing of oversized and slotted holes (J3.8)

C_2 Increment used in computing minimum edge distance for oversized and slotted holes (J3.9)

D Factor depending upon type of transverse stiffeners (G4); outside diameter of tubular member, in. (Appendix B5.2)

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

E_c Modulus of elasticity of concrete, ksi (I2)

F_a Axial compressive stress permitted in a prismatic member in the absence of bending moment, ksi (E2)

$F_{a\gamma}$ Axial compressive stress permitted in a tapered member in the absence of bending moment, ksi (Appendix F7.3)

F_b Bending stress permitted in a prismatic member in the absence of axial force, ksi (F1.1)

F'_b Allowable bending stress in compression flange of plate girders as reduced for hybrid girders or because of large web depth-to-thickness ratio, ksi (G2)

$F_{b\gamma}$ Bending stress permitted in a tapered member in the absence of axial force, ksi (Appendix F7.6)

F'_e Euler stress for a prismatic member divided by factor of safety, ksi (H1)

$F'_{e\gamma}$ Euler stress for a tapered member divided by factor of safety, ksi (Appendix F7.6)

F_p Allowable bearing stress, ksi (J3.7)

$F_{s\gamma}$ St. Venant torsion resistance bending stress in a tapered member, ksi (Appendix F7.4)

F_t Allowable axial tensile stress, ksi (D1)

F_u Specified minimum tensile strength of the type of steel or fastener being used, ksi (B10)

F_v Allowable shear stress, ksi (F4)

$F_{w\gamma}$ Flange warping torsion resistance bending stress in a tapered member, ksi (Appendix F7.4)

F_y Specified minimum yield stress of the type of steel being used, ksi (B5.1). As used in this Specification, "yield stress" denotes either the specified minimum yield point (for those steels with a yield point) or specified minimum yield strength (for those steels without a yield point)

F_{yc} Specified minimum column yield stress, ksi (K1.2)

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

F_{yr} Specified minimum yield stress of the longitudinal reinforcing steel, ksi (I4)

F_{yst} Specified minimum stiffener yield stress, ksi (K1.8)

F_{yw} Specified minimum yield stress of beam web, ksi (B5.1)

H_s Length of a stud shear connector after welding, in. (I5.2)

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

I_{eff} Effective moment of inertia of composite sections for deflection computations, in.^{4} (I4)

I_p Moment of inertia of primary member in flat roof framing, in.^{4} (K2)

I_s Moment of inertia of secondary member in flat roof framing, in.^{4} (K2); moment of inertia of steel beam in composite construction, in.^{4} (I4)

I_{tr} Moment of inertia of transformed composite section, in.^{4} (I4)

K Effective length factor for a prismatic member (B7)

K_γ Effective length factor for a tapered member (Appendix F7.3)

L Unbraced length of tensile members, in. (B7); actual unbraced length of a column, in. (C2); unbraced length of member measured between the centers of gravity of the bracing members, in. (Appendix F7.1)

L_c Maximum unbraced length of the compression flange at which the allowable bending stress may be taken at $0.66F_y$ or as determined by AISC Specification Equation (F1-3) or Equation (F2-3), when applicable, ft (F1)

L_e Distance from free edge to center of the bolt, in. (J3.6)

L_p Length of primary member in flat roof framing, ft (K2)

L_s Length of secondary member in flat roof framing, ft (K2)

M Moment, kip-ft. (I4); maximum factored bending moment, kip-ft, (N4)

M_1 Smaller moment at end of unbraced length of beam-column (F3.1); larger moment at one end of three-segment portion of a tapered member (Appendix F7.4)

M_2 Larger moment at end of unbraced length of beam-column (F3.1); maximum moment in three adjacent segments of a tapered member (Appendix F7.4)

M_m Critical moment that can be resisted by a plastically designed member in the absence of axial load, kip-ft (N4)

M_p Plastic moment, kip-ft (N4)

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

N_r Number of stud shear connectors on a beam in one transverse rib of a metal deck, not to exceed 3 in calculations (I5.2)

N_1 Number of shear connectors required between point of maximum moment and point of zero moment (I4)

N_2 Number of shear connectors required between concentrated load and point of zero moment (I4)

P Force transmitted by a fastener, kips (J3.8); factored axial load, kips (N3); normal force, kips (J10.2); axial load, kips (C1)

P_{bf} Factored beam flange or connection plate force in a restrained connection, kips (K1.2)

P_{cr} Maximum strength of an axially loaded compression member or beam, kips (N3.1)

P_e Euler buckling load, kips (N4)

P_y Plastic axial load, equal to profile area times specified minimum yield stress, kips (N3.1)

Q Full reduction factor for slender compression elements (Appendix B5.2)

Q_a Ratio of effective profile area of an axially loaded member to its total profile area (Appendix B5.2)

Q_s Axial stress reduction factor where width-thickness ratio of unstiffened elements exceeds noncompact section limits given in Sect. B5 (Appendix B5.2)

R Reaction or concentrated load applied to beam or girder, kips (K1.3); radius, in. (J2.1)

R_{PG} Plate girder bending strength reduction factor (G2)

R_e Hybrid girder factor (G2)

S Spacing of secondary members in a flat roof, ft (K2); governing slenderness ratio of a tapered member (Appendix F7.3)

S_{eff} Effective section modulus corresponding to partial composite action, in.^{3} (I2)

S_s Section modulus of steel beam used in composite design, referred to the bottom flange, in.^{3} (I2)

S_{tr} Section modulus of transformed composite cross section, referred to the bottom flange; based upon maximum permitted effective width of concrete flange, in.^{3} (I2)

T_b Specified pretension of a high-strength bolt, kips (J3.6)

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

V Shear produced by factored loading, kips (N5); friction force, kips (J10.2)

V_h Total horizontal shear to be resisted by connectors under full composite action, kips (I2)

V'_h Total horizontal shear provided by the connectors providing partial composite action, kips (I2)

Y Ratio of yield stress of web steel to yield stress of stiffener steel (G4)

Z Plastic section modulus, in.^{3} (N4)

a Clear distance between transverse stiffeners, in. (F4); dimension parallel to the direction of stress, in. (Appendix K4)

a' Distance beyond theoretical cut-off point required at ends of welded partial length cover plate to develop stress, in. (B10)

b Actual width of stiffened and unstiffened compression elements as defined in Sect. B5.1, in.; dimension normal to the direction of stress, in. (Appendix K4)

b_e Effective width of stiffened compression element, in (Appendix B5.2)

b_f Flange width of rolled beam or plate girder, in. (F1.1)

d Depth of beam or girder, in. (B5.1); diameter of a roller or rocker bearing, in. (J8); nominal diameter of a fastener, in. (J3.7)

d_L Depth at the larger end of a tapered member, in. (Appendix F7.1)

d_c Web depth clear of fillets, in. (K1.5)

d_o Depth at the smaller end of a tapered member or unbraced segment thereof, in. (Appendix F7.1)

f Axial compression stress on member based on effective area, ksi (Appendix B5.2)

f_a Computed axial stress, ksi (B5.1)

f_{ao} Computed axial stress at the smaller end of a tapered member or unbraced segment thereof, ksi (Appendix F7.6)

f_b Computed bending stress, ksi (H1)

f_{b1} Smallest computed bending stress at one end of a tapered segment, ksi (Appendix F7.4)

f_{b2} Largest computed bending stress at one end of a tapered segment, ksi (Appendix F7.4)

f_{bl} Computed bending stress at the larger end of a tapered member or unbraced segment thereof, ksi (Appendix F7.6)

f'_c Specified compression strength of concrete, ksi (I2)

f_t Computed tensile stress, ksi (J3.6)

f_v Computed shear stress, ksi (F5)

f_{vs} Shear between girder web and transverse stiffeners, kips per linear in. of single stiffener or pair of stiffeners (G4)

g Transverse spacing between fastener gage lines, in. (B2)

h Clear distance between flanges of a beam or girder at the section under investigation, in. (B5)

h_r Nominal rib height for steel deck, in. (I5.2)

h_s Factor applied to the unbraced length of a tapered member (Appendix F7.4)

h_w Factor applied to the unbraced length of a tapered member (Appendix F7.4)

k Distance from outer face of flange to web toe of fillet of rolled shape or equivalent distance on welded section, in. (K1.3)

k_c Compression element restraint coefficient (B5)

k_v Shear buckling coefficient for girder webs (F4)

l For beams, distance between cross sections braced against twist or lateral displacement of the compression flange, in. (F1.3); for columns, actual unbraced length of member, in. (B7); unsupported length of a lacing bar, in. (E4); weld length, in. (B3); largest laterally unbraced length along either flange at the point of load, in. (K1.5)

l_b Actual unbraced length in plane of bending, in. (H1)

l_{cr} Critical unbraced length adjacent to plastic hinge, in. (N9)

n Modular ratio (E/E_c) (I2)

q Allowable horizontal shear to be resisted by a shear connector, kips (I4)

r Governing radius of gyration, in. (B7)

r_T Radius of gyration of a section comprising the compression flange plus ⅓ of the compression web area, taken about an axis in the plane of the web, in. (F1.3)

r_{To} Radius of gyration at the smaller end of a tapered member or unbraced segment thereof, considering only the compression flange plus ⅓ of the compression web area, taken about an axis in the plane of the web, in. (Appendix F7.4)

r_b Radius of gyration about axis of concurrent bending, in. (H1)

r_{bo} Radius of gyration about axis of concurrent bending at the smaller end of a tapered member or unbraced segment thereof, in. (Appendix F7.6)

r_o Radius of gyration at the smaller end of a tapered member, in. (Appendix F7.3)

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

t Thickness of a connected part, in. (J3.9); wall thickness of a tubular member, in. (Appendix B5); compression element thickness, in. (B5.1); filler thickness, in. (J6)

t_b Thickness of beam flange or moment connection plate at rigid beam-to-column connection, in. (K1.8)

t_f Flange thickness, in. (F1.1)

t_w Web thickness, in. (B5.1)

t_{wc} Column web thickness, in. (K1.6)

w Length of channel shear connectors, in. (I4); plate width (distance between welds), in. (B3)

w_r Average width of rib or haunch of concrete slab on formed steel deck, in. (I5.1)

x Subscript relating symbol to strong axis bending

y Subscript relating symbol to weak axis bending

z Distance from the smaller end of a tapered member, in. (Appendix F7.3)

α $= 0.6\ F_{yw}/F_b < 1.0$ (G2)

β Ratio S_{tr}/S_s or S_{eff}/S_s (I4)

γ Tapering ratio of a tapered member or unbraced segment of a tapered member (Appendix F7.1); subscript relating symbol to tapered members

Δ Displacement of the neutral axis of a loaded member from its position when the member is not loaded, in. (C1)

μ Coefficient of friction (J10.2)

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GLOSSARY

Alignment chart for columns. A nomograph for determining the effective length factor K for some types of columns

Amplification factor. A multiplier of the value of moment or deflection in the unbraced length of an axially loaded member to reflect the secondary values generated by the eccentricity of the applied axial load within the member

Aspect ratio. In any rectangular configuration, the ratio of the lengths of the sides

Batten plate. A plate element used to join two parallel components of a built-up column, girder or strut rigidly connected to the parallel components and designed to transmit shear between them

Beam. A structural member whose primary function is to carry loads transverse to its longitudinal axis

Beam-column. A structural member whose primary function is to carry loads both transverse and parallel to its longitudinal axis

Bent. A plane framework of beam or truss members which support loads and the columns which support these members

Biaxial bending. Simultaneous bending of a member about two perpendicular axes

Bifurcation. The phenomenon whereby a perfectly straight member under compression may either assume a deflected position or may remain undeflected, or a beam under flexure may either deflect and twist out of plane or remain in its in-plane deflected position.

Braced frame. A frame in which the resistance to lateral load or frame instability is primarily provided by a diagonal, a K-brace or other auxiliary system of bracing

Brittle fracture. Abrupt cleavage with little or no prior ductile deformation

Buckling load. The load at which a perfectly straight member under compression assumes a deflected position

Built-up member. A member made of structural metal elements that are welded, bolted or riveted together

Cladding. The exterior covering of the structural components of a building

Cold-formed members. Structural members formed from steel without the application of heat

Column. A structural member whose primary function is to carry loads parallel to its longitudinal axis

Column curve. A curve expressing the relationship between axial column strength and slenderness ratio

Combined mechanism. A mechanism determined by plastic analysis procedure which combines elementary beam, panel and joint mechanisms

Compact section. Compact sections are capable of developing a fully plastic stress distribution and possess rotation capacity of approximately 3 before the onset of local buckling

Composite beam. A steel beam structurally connected to a concrete slab so that the beam and slab respond to loads as a unit. See also Concrete-encased beam

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

Concrete-encased beam. A beam totally encased in concrete cast integrally with the slab

Connection. Combination of joints used to transmit forces between two or more members. Categorized by the type and amount of force transferred (moment, shear, end reaction). See also splices

Critical load. The load at which bifurcation occurs as determined by a theoretical stability analysis

Curvature. The rotation per unit length due to bending

Design documents. See structural design documents

Design strength. Resistance (force, moment, stress, as appropriate) provided by element or connection; the product of the nominal strength and the resistance factor

Diagonal bracing. Inclined structural members carrying primarily axial load employed to enable a structural frame to act as a truss to resist horizontal loads

Diaphragm. Floor slab, metal wall or roof panel possessing a large in-plane shear stiffness and strength adequate to transmit horizontal forces to resisting systems

Diaphragm action. The in-plane action of a floor system (also roofs and walls) such that all columns framing into the floor from above and below are maintained in their same position relative to each other

Double curvature. A bending condition in which end moments on a member cause the member to assume an S-shape

Drift. Lateral deflection of a building

Drift index. The ratio of lateral deflection to the height of the building

Ductility factor. The ratio of the total deformation at maximum load to the elastic-limit deformation

Effective length. The equivalent length KL used in compression formulas and determined by a bifurcation analysis

Effective length factor K. The ratio between the effective length and the unbraced length of the member measured between the centers of gravity of the bracing members

Effective moment of inertia. The moment of inertia of the cross section of a member that remains elastic when partial plastification of the cross section takes place, usually under the combination of residual stress and applied stress. Also, the moment of inertia based on effective widths of elements that buckle locally. Also, the moment of inertia used in the design of partially composite members

Effective stiffness. The stiffness of a member computed using the effective moment of inertia of its cross section

Effective width. The reduced width of a plate or slab which, with an assumed uniform stress distribution, produces the same effect on the behavior of a structural member as the actual plate width with its nonuniform stress distribution

Elastic analysis. Determination of load effects (force, moment, stress as appropriate) on members and connections based on the assumption that material deformation disappears on removal of the force that produced it

Elastic-perfectly plastic. A material which has an idealized stress-strain curve that varies linearly from the point of zero strain and zero stress up to the yield point of the material, and then increases in strain at the value of the yield stress without any further increases in stress

Embedment. A steel component cast in a concrete structure which is used to transmit externally applied loads to the concrete structure by means of bearing, shear, bond, friction or any combination thereof. The embedment may be fabricated of structural-steel plates, shapes, bars, bolts, pipe, studs, concrete reinforcing bars, shear connectors or any combination thereof

Encased steel structure. A steel-framed structure in which all of the individual frame members are completely encased in cast-in-place concrete

Euler formula. The mathematical relationship expressing the value of the Euler load in terms of the modulus of elasticity, the moment of inertia of the cross section and the length of a column

Euler load. The critical load of a perfectly straight, centrally loaded, pin-ended column

Eyebar. A particular type of pin-connected tension member of uniform thickness with forged or flame cut head of greater width than the body proportioned to provide approximately equal strength in the head and body

Factored load. The product of the nominal load and a load factor

Fastener. Generic term for welds, bolts, rivets or other connecting device

Fatigue. A fracture phenomenon resulting from a fluctuating stress cycle

First-order analysis. Analysis based on first-order deformations in which equilibrium conditions are formulated on the undeformed structure

Flame-cut plate. A plate in which the longitudinal edges have been prepared by oxygen cutting from a large plate

Flat width. For a rectangular tube, the nominal width minus twice the outside corner radius. In absence of knowledge of the corner radius, the flat width may be taken as the total section width minus three times the thickness

Flexible connection. A connection permitting a portion, but not all, of the simple beam rotation of a member end

Floor system. The system of structural components separating the stories of a building

Force. Resultant of distribution of stress over a prescribed area. A reaction that develops in a member as a result of load (formerly called total stress or stress). Generic term signifying axial loads, bending moment, torques and shears

Fracture toughness. Measurement of the ability to absorb energy without fracture. Generally determined by impact loading of specimens containing a notch having a prescribed geometry

Frame buckling. A condition under which bifurcation may occur in a frame

Frame instability. A condition under which a frame deforms with increasing lateral deflection under a system of increasing applied monotonic loads until a maximum value of the load called the stability limit is reached, after which the frame will continue to deflect without further increase in load

Fully composite beam. A composite beam with sufficient shear connectors to develop the full flexural strength of the composite section

High-cycle fatigue. Failure resulting from more than 20,000 applications of cyclic stress

Hybrid beam. A fabricated steel beam composed of flanges with a greater yield strength that that of the web. Whenever the maximum flange stress is less than or equal to the web yield stress the girder is considered homogeneous

Hysteresis loop. A plot of force versus displacement of a structure or member subjected to reversed, repeated load into the inelastic range, in which the path followed during release and removal of load is different from the path for the addition of load over the same range of displacement

Inclusions. Nonmetallic material entrapped in otherwise sound metal

Incomplete fusion. Lack of union by melting of filler and base metal over entire prescribed area

Inelastic action. Material deformation that does not disappear on removal of the force that produced it

Instability. A condition reached in the loading of an element or structure in which continued deformation results in a decrease of load-resisting capacity

Joint. Area where two or more ends, surfaces, or edges are attached. Categorized by type of fastener or weld used and method of force transfer

K-bracing. A system of struts used in a braced frame in which the pattern of the struts resembles the letter *K*, either normal or on its side

Lamellar tearing. Separation in highly restrained base metal caused by through-thickness strains induced by shrinkage of adjacent weld metal

Lateral bracing member. A member utilized individually or as a component of a lateral bracing system to prevent buckling of members or elements and/or to resist lateral loads

Lateral (or lateral-torsional) buckling. Buckling of a member involving lateral deflection and twist

Limit state. A condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function *(serviceability limit state)* or to be unsafe *(strength limit state)*

Limit states. Limits of structural usefulness, such as brittle fracture, plastic collapse, excessive deformation, durability, fatigue, instability and serviceability

Load factor. A factor that accounts for unavoidable deviations of the actual load from the nominal value and for uncertainties in the analysis that transform the load into a load effect

Loads. Forces or other actions that arise on structural systems from the weight of all permanent construction, occupants and their possessions, environmental effects, differential settlement and restrained dimensional changes. *Permanent* loads are those loads in which variations in time are rare or of small magnitude. All other loads are *variable* loads. See *Nominal loads*.

LRFD (Load and Resistance Factor Design). A method of proportioning structural components (members, connectors, connecting elements and assemblages) such that no applicable limit state is exceeded when the structure is subjected to all appropriate load combinations

Local buckling. The buckling of a compression element which may precipitate the failure of the whole member

Low-cycle fatigue. Fracture resulting from a relatively high stress range resulting in a relatively small number of cycles to failure

Lower bound load. A load computed on the basis of an assumed equilibrium moment diagram in which the moments are not greater than M_p, that is, less than or at best equal to the true ultimate load

Mechanism. An articulated system able to deform without an increase in load, used in the special sense that the linkage may include real hinges or plastic hinges, or both

Mechanism method. A method of plastic analysis in which equilibrium between external forces and internal plastic hinges is calculated on the basis of an assumed mechanism. The failure load so determined is an upper bound

Nominal loads. The magnitudes of the loads specified by the applicable code

Nominal strength. The capacity of a structure or component to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions

Noncompact section. Noncompact sections can develop yield stress in compression elements before local buckling occurs, but will not resist inelastic local buckling at strain levels required for a fully plastic stress distribution

P-Delta effect. Secondary effect of column axial loads and lateral deflection on the moments in members

Panel zone. The zone in a beam-to-column connection that transmits moments by a shear panel

Partially composite beam. A composite beam for which the shear strength of shear connectors governs the flexural strength

Plane frame. A structural system assumed for the purpose of analysis and design to be two-dimensional

Plastic analysis. Determination of load effects (force, moment, stress, as appropriate) on members and connections based on the assumption of rigid-plastic behavior, i.e., that equilibrium is satisfied throughout the structure and yield is not exceeded anywhere. Second order effects may need to be considered

Plastic design section. The cross section of a member which can maintain a full plastic moment through large rotations so that a mechanism can develop; the section suitable for plastic design

Plastic hinge. A yielded zone which forms in a structural member when the plastic moment is attained. The beam is assumed to rotate as if hinged, except that it is restrained by the plastic moment M_p

Plastic-limit load. The maximum load that is attained when a sufficient number of yield zones have formed to permit the structure to deform plastically without further increase in load. It is the largest load a structure will support, when perfect plasticity is assumed and when such factors as instability, second-order effects, strain hardening and fracture are neglected

Plastic mechanism. See mechanism

Plastic modulus. The section modulus of resistance to bending of a completely yielded cross-section. It is the combined static moment about the neutral axis of the cross-sectional areas above and below that axis

Plastic moment. The resisting moment of a fully yielded cross section

Plastic strain. The difference between total strain and elastic strain

Plastic zone. The yielded region of a member

Plastification. The process of successive yielding of fibers in the cross section of a member as bending moment is increased

Plate girder. A built-up structural beam

Post-buckling strength. The load that can be carried by an element, member or frame after buckling

Redistribution of moment. A process which results in the successive formation of plastic hinges so that less highly stressed portions of a structure may carry increased moments

Required strength. Load effect (force, moment, stress, as appropriate) acting on an element or connection determined by structural analysis from the factored loads (using most appropriate critical load combinations)

Residual stress. The stress that remains in an unloaded member after it has been formed into a finished product. (Examples of such stresses include, but are not limited to, those induced by cold bending, cooling after rolling, or welding.)

Resistance. The capacity of a structure or component to resist the effects of loads. It is determined by computations using specified material strengths, dimensions and formulas derived from accepted principles of structural mechanics, or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions. Resistance is a generic term that includes both strength and serviceability limit states

Resistance factor. A factor that accounts for unavoidable deviations of the actual strength from the nominal value and the manner and consequences of failure

Rigid frame. A structure in which connections maintain the angular relationship between beam and column members under load

Root of the flange. Location on the web of the corner radius termination point or the toe of the flange-to-web weld. Measured as the k-distance from the far side of the flange

Rotation capacity. The incremental angular rotation that a given shape can accept prior to local failure defined as $R = (\theta_u/\theta_p) - 1$ where θ_u is the overall rotation attained at the factored load state and θ_p is the idealized rotation corresponding to elastic theory applied to the case of $M = M_p$

St. Venant torsion. That portion of the torsion in a member that induces only shear stresses in the member

Second-order analysis. Analysis based on second-order deformations, in which equilibrium conditions are formulated on the deformed structure

Service load. Load expected to be supported by the structure under normal usage; often taken as the nominal load

Serviceability limit state. Limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability or the comfort of its occupants or function of machinery under normal usage.

Shape factor. The ratio of the plastic moment to the yield moment, or the ratio of the plastic modulus to the section modulus for a cross section

Shear-friction. Friction between the embedment and the concrete that transmits shear loads. The relative displacement in the plane of the shear load is considered to be resisted by shear-friction anchors located perpendicular to the plane of the shear load

Shear lugs. Plates, welded studs, bolts and other steel shapes that are embedded in the concrete and located transverse to the direction of the shear force and that transmit shear loads introduced into the concrete by local bearing at the shear lug-concrete interface

Shear wall. A wall that in its own plane resists shear forces resulting from applied wind, earthquake or other transverse loads or provides frame stability. Also called a structural wall

Sidesway. The lateral movement of a structure under the action of lateral loads, unsymmetrical vertical loads or unsymmetrical properties of the structure

Sidesway buckling. The buckling mode of a multistory frame precipitated by the relative lateral displacements of joints, leading to failure by sidesway of the frame

Simple plastic theory. See Plastic design

Single curvature. A deformed shape of a member having one smooth continuous arc, as opposed to double curvature which contains a reversal

Slender section. The cross section of a member which will experience local buckling in the elastic range

Slenderness ratio. The ratio of the effective length of a column to the radius of gyration of the column, both with respect to the same axis of bending

Slip-critical joint. A bolt joint in which the slip resistance of the connection is required

Space frame. A three-dimensional structural framework (as contrasted to a plane frame)

Splice. The connection between two structural elements joined at their ends to form a single, longer element

Stability-limit load. Maximum (theoretical) load a structure can support when second-order instability effects are included

Stepped column. A column with changes from one cross section to another occurring at abrupt points within the length of the column

Stiffener. A member, usually an angle or plate, attached to a plate or web of a beam or girder to distribute load, to transfer shear or to prevent buckling of the member to which it is attached

Stiffness. The resistance to deformation of a member or structure measured by the ratio of the applied force to the corresponding displacement

Story drift. The difference in horizontal deflection at the top and bottom of a story

Strain hardening. Phenomenon wherein ductile steel, after undergoing considerable deformation at or just above yield point, exhibits the capacity to resist substantially higher loading than that which caused initial yielding

Strain-hardening strain. For structural steels that have a flat (plastic) region in the stress-strain relationship, the value of the strain at the onset of strain hardening

Strength design. A method of proportioning structural members using load factors and resistance factors such that no applicable limit state is exceeded (also called load and resistance factor design)

Strength limit state. Limiting condition affecting the safety of the structure, in which the ultimate load-carrying capacity is reached

Stress. Force per unit area

Stress concentration. Localized stress considerably higher than average (even in uniformly loaded cross sections of uniform thickness) due to abrupt changes in geometry or localized loading

Strong axis. The major principal axis of a cross section

Structural design documents. Documents prepared by the designer (plans, design details and job specifications)

Structural system. An assemblage of load-carrying components which are joined together to provide regular interaction or interdependence

Stub column. A short compression-test specimen, long enough for use in measuring the stress-strain relationship for the complete cross section, but short enough to avoid buckling as a column in the elastic and plastic ranges

Subassemblage. A truncated portion of a structural frame

Supported frame. A frame which depends upon adjacent braced or unbraced frames for resistance to lateral load or frame instability. (This transfer of load is frequently provided by the floor or roof system through diaphragm action or by horizontal cross bracing in the roof.)

Tangent modulus. At any given stress level, the slope of the stress-strain curve of a material in the inelastic range as determined by the compression test of a small specimen under controlled conditions.

Temporary structure. A general term for anything that is built or constructed (usually to carry construction loads) that will eventually be removed before or after completion of construction and does not become part of the permanent structural system

Tensile strength. The maximum tensile stress that a material is capable of sustaining

Tension field action. The behavior of a plate girder panel under shear force in which diagonal tensile stresses develop in the web and compressive forces develop in the transverse stiffeners in a manner analogous to a Pratt truss

Toe of the fillet. Termination point of fillet weld or of rolled section fillet

Torque-tension relationship Term applied to the wrench torque required to produce specified pre-tension in high-strength bolts

Turn-of-nut method. Procedure whereby the specified pre-tension in high-strength bolts is controlled by rotation of the wrench a predetermined amount after the nut has been tightened to a snug fit

Unbraced frame. A frame in which the resistance to lateral load is provided by the bending resistance of frame members and their connections

Unbraced length. The distance between braced points of a member, measured between the centers of gravity of the bracing members

Undercut. A notch resulting from the melting and removal of base metal at the edge of a weld

Universal-mill plate. A plate in which the longitudinal edges have been formed by a rolling process during manufacture. Often abbreviated as UM plate

Upper bound load. A load computed on the basis of an assumed mechanism which will always be at best equal to or greater than the true ultimate load

Vertical bracing system. A system of shear walls, braced frames or both, extending throughout one or more floors of a building

Von Mises yield criterion. A theory which states that inelastic action at any point in a body under any combination of stresses begins only when the strain energy of distortion per unit volume absorbed at the point is equal to the strain energy of distortion absorbed per unit volume at any point in a simple tensile bar stressed to the elastic limit under a state of uniaxial stress. It is often called the maximum strain-energy-of-distortion theory. Accordingly, shear yield occurs at 0.58 times yield strength

Warping torsion. That portion of the total resistance to torsion that is provided by resistance to warping of the cross section

Weak axis. The minor principal axis of a cross section

Weathering steel. A type of high-strength, low-alloy steel which can be used in normal environments (not marine) and outdoor exposures without protective paint covering. This steel develops a tight adherent rust at a decreasing rate with respect to time

Web buckling. The buckling of a web plate

Web crippling. The local failure of a web plate in the immediate vicinity of a concentrated load or reaction

Working load. Also called service load. The actual load assumed to be acting on the structure.

Yield moment. In a member subjected to bending, the moment at which an outer fiber first attains the yield stress

Yield plateau. The portion of the stress-strain curve for uniaxial tension or compression in which the stress remains essentially constant during a period of substantially increased strain

Yield point. The first stress in a material at which an increase in strain occurs without an increase in stress, the yield point less than the maximum attainable stress

Yield strength. The stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain. Deviation expressed in terms of strain

Yield stress. Yield point, yield strength or yield-stress level as defined

Yield-stress level. The average stress during yielding in the plastic range, the stress determined in a tension test when the strain reaches 0.005 in. per in.