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STRUCTURAL STEEL DESIGN AND CONSTRUCTION

The many desirable characteristics of structural steels has led to their widespread use in a large variety of applications. Structural steels are available in many product forms and offer an inherently high strength. They have a very high modulus of elasticity, so deformations under load are very small. Structural steels also possess high ductility. They have a linear or nearly linear stress-strain relationship up to relatively large stresses, and the modulus of elasticity is the same in tension and compression. Hence, structural steels' behavior under working loads can be accurately predicted by elastic theory. Structural steels are made under controlled conditions, so purchasers are assured of uniformly high quality.

Standardization of sections has facilitated design and kept down the cost of structural steels. For tables of properties of these sections, see "Manual of Steel Construction," American Institute of Steel Construction, One East Wacker Dr., Chicago, IL 60601-2001 www.aisc.org.

This section provides general information on structural-steel design and construction. Any use of this material for a specific application should be based on a determination of its suitability for the application by professionally qualified personnel.

9.1 Properties of Structural Steels

The term *structural steels* includes a large number of steels that, because of their economy, strength, ductility, and other properties, are suitable for load-carrying members in a wide variety of fabricated structures. Steel plates and shapes intended for use in bridges, buildings, transportation equipment, construction equipment, and similar applications are generally ordered to a specific specification of ASTM and furnished in "Structural Quality" according to the requirements (tolerances, frequency of testing, and so on) of ASTM A6. Plate steels for pressure vessels are furnished in "Pressure Vessel Quality" according to the requirements of ASTM A20.

Each structural steel is produced to specified minimum mechanical properties as required by the specific ASTM designation under which it is ordered. Generally, the structural steels include steels with yield points ranging from about 30 to 100 ksi. The various strength levels are obtained by varying the chemical composition and by heat treatment. Other factors that may affect mechanical properties include product thickness, finishing temperature, rate of cooling, and residual elements.

The following definitions aid in understanding the properties of steel.

Yield point F_y is that unit stress, ksi, at which the stress-strain curve exhibits a well-defined increase in strain without an increase in stress. Many design rules are based on yield point.

Tensile strength, or ultimate strength, is the largest unit stress, ksi, the material can achieve in a tensile test.

Modulus of elasticity E is the slope of the stress-strain curve in the elastic range, computed by dividing the unit stress, ksi, by the unit strain, in/in. For all structural steels, it is usually taken as 29,000 ksi for design calculations.

Ductility is the ability of the material to undergo large inelastic deformations without fracture. It is generally measured by the percent elongation for a specified gage length (usually 2 or 8 in). Structural steel has considerable ductility, which is recognized in many design rules.

Weldability is the ability of steel to be welded without changing its basic mechanical properties. However, the welding materials, procedures, and techniques employed must be in accordance with the approved methods for each steel. Generally, weldability decreases with increase in carbon and manganese.

Notch toughness is an index of the propensity for brittle failure as measured by the impact energy necessary to fracture a notched specimen, such as a Charpy V-notch specimen.

Toughness reflects the ability of a smooth specimen to absorb energy as characterized by the area under a stress-strain curve.

Corrosion resistance has no specific index. However, relative corrosion-resistance ratings are based on the slopes of curves of corrosion loss (reduction in thickness) vs. time. The reference of comparison is usually the corrosion resistance of carbon steel without copper. Some high-strength structural steels are alloyed with copper and other elements to produce high resistance to atmospheric deterioration. These steels develop a tight oxide that inhibits further atmospheric corrosion. Figure 9.1 compares the rate of reduction of thickness of typical proprietary "corrosion-resistant" steels with that of ordinary structural steel. For standard methods of estimating the atmospheric corrosion resistance of low-alloy steels, see ASTM Guide G101, American Society of Testing and Materials, 100 Barr Harbor Drive West Conshchoken, PA, 19428-2959, www.astm.org.

(R. L. Brockenbrough and B. G. Johnston, "USS Steel Design Manual," R. L. Brockenbrough & Associates, Inc., Pittsburgh, PA 15243.)

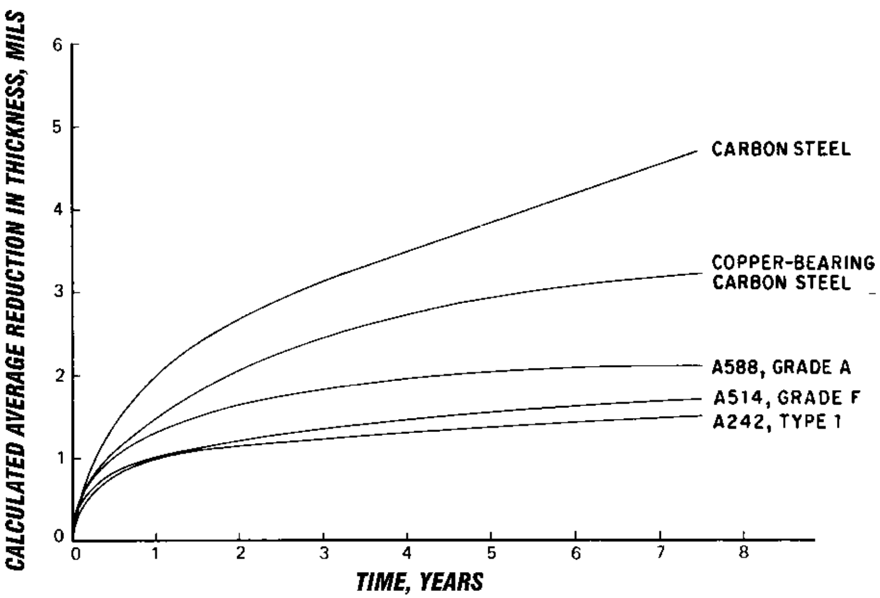

Fig. 9.1 Curves show corrosion rates for steels in an industrial atmosphere.

9.2 Summary of Available Structural Steels

The specified mechanical properties of typical structural steels are presented in Table 9.1. These steels may be considered in four general categories, depending on chemical composition and heat treatment, as indicated below. The tensile properties for structural shapes are related to the size groupings indicated in Table 9.2.

Carbon steels are those steels for which (1) the maximum content specified for any of the following elements does not exceed the percentages noted: manganese—1.65%, silicon—0.60%, and copper—0.60%, and (2) no minimum content is specified for the elements added to obtain a desired alloying effect.

The first carbon steel listed in Table 9.1—A36—is a weldable steel available as plates, bars, and structural shapes. The last steel listed in the table. A992, which is available only for W shapes (rolled wide flange shapes), was introduced in 1998 and has rapidly become the preferred steel for building construction. It is unique in that the steel has a maximum ratio specified for yield to tensile strength, which is 0.85. The specification also includes a maximum carbon equivalent of 0.47 percent to enhance weldability. A minimum average Charpy V-notch toughness of 20 ft-lb at 70 °F can be specified as a supplementary requirement. The other carbon steels listed in Table 9.1 are available only as plates. Although each steel is available in three or more strength levels, only one strength level is listed in the table for A283 and A285 plates.

A283 plates are furnished as structural-quality steel in four strength levels—designated as Grades A, B, C, and D—having specified minimum yield points of 24, 27, 30, and 33 ksi. This plate steel is of structural quality and has been used primarily for oil- and water-storage vessels. A573 steel, which is available in three strength levels, is a structural-quality steel intended for service at atmospheric temperatures at which improved notch toughness is important. The other plate steels—A285, A515, and A516—are all furnished in pressure-vessel quality only and are intended for welded construction in more critical applications, such as pressure vessels. A516 is furnished in four strength levels—designated as Grades 55, 60, 65, and 70 (denoting their tensile strength)—having specified minimum yield points of 30, 32, 35, and 38 ksi. A515 has similar grades except there is no Grade 55. A515 steel is for "intermediate and higher temperature service," whereas A516 is for "moderate and lower temperature service."

Carbon steel pipe used for structural purposes is usually A53 Grade B with a specified minimum yield point of 35 ksi. Structural carbon-steel hot-formed tubing, round and rectangular, is furnished to the requirements of A501 with a yield point of 36 ksi. Cold-formed tubing is also available in several grades with a yield point from 33 to 50 ksi.

High-strength, low-alloy steels have specified minimum yield points above about 40 ksi in the hot-rolled condition and achieve their strength by small alloying additions rather than through heat treatment. A588 steel, available in plates, shapes, and bars, provides a yield point of 50 ksi in plate thicknesses through 4 in and in all structural shapes and is the predominant steel used in structural applications in which durability is important. Its resistance to atmospheric corrosion is about four times that of carbon steel. A242 steel also provides enhanced atmospheric-corrosion resistance. Because of this superior atmospheric-corrosion resistance, A588 and A242 steels provide a longer paint life than other structural steels. In addition, if suitable precautions are taken, these steels can be used in the bare, uncoated condition in many applications in which the members are exposed to the atmosphere because a tight oxide is formed that substantially reduces further corrosion. Bolted joints in bare steel require special considerations as discussed in Art. 9.36.

A572 high-strength, low-alloy steel is used extensively to reduce weight and cost. It is produced in several grades that provide a yield point of 42 to 65 ksi. Its corrosion resistance is the same as that of carbon steel.

Heat-Treated Carbon and High-Strength, Low-Alloy Steels ▪ This group is comprised of carbon and high-strength, low-alloy steels that have been heat-treated to obtain more desirable mechanical properties.

A633, Grades A through E, are weldable plate steels furnished in the normalized condition to provide an excellent combination of strength (42 to 60 ksi minimum yield point) and toughness (up to 15 ft-lb at -75 °F).

Table 9.1 Specified Mechanical Properties of Steel*

ASTM Designation	Plate Thickness, in	ANSI/ASTM Group or Weight/ft for Structural Shapes	Yield Point or Yield Strength, ksi	Tensile Strength, ksi
		Carbon Steels		
A36	To 8, incl	To 426 lb/ft, incl.	36	58–80
	Not applicable	Over 426 lb/ft	36	58
	over 8	Not applicable	32	58–80
A283, Grade C	None specified	Not applicable	30	55–70
A285, Grade C	To 2, incl	Not applicable	30	55–75
A516, Grade 55	To 12, incl	Not applicable	30	55–75
A516, Grade 60	To 8, incl	Not applicable	32	60–80
A516, Grade 65	To 8, incl	Not applicable	35	65–85
A516, Grade 70	To 8, incl	Not applicable	38	70–90
A573, Grade 58	To 1½, incl	Not applicable	32	58–71
A573, Grade 65	To 1½, incl	Not applicable	35	65–77
A573, Grade 70	To 1½, incl	Not applicable	42	70–90
A992	Not Applicable	All w shapes	50–65	65
		High-Strength, Low-Alloy Steels		
A242	To ¾, incl	Groups 1 and 2	50	70
	Over ¾ to 1½, incl	Group 3	46	67
	Over 1½ to 4, incl	Groups 4 and 5	42	63
A588	To 4, incl	Groups 1–5	50	70
	Over 4 to 5, incl		46	67
	Over 5 to 8, incl		42	63
A572, Grade 42	To 6, incl	Groups 1–5	42	60
A572, Grade 50	To 4, incl	Groups 1–5	50	65
A572, Grade 60	To 1¼, incl	Groups 1 and 3	60	75
A572, Grade 65	To 1¼, incl	Groups 1 and 3	65	80
		Heat-Treated Carbon and High-Strength, Low-Alloy Steels		
A633, Grade C and D	To 2½, incl	Not applicable	50	70–90
	Over 2½ to 4, incl		46	65–85
A633, Grade E	To 4, incl		60	80–100
	Over 4 to 6, incl		55	75–95
A678, Grade C	To ¾, incl	Not applicable	75	95–115
	Over ¾ to 1½, incl		70	90–110
	Over 1½ to 2, incl		65	85–105
A852	To 4, incl	Not applicable	70	90–110
A913, Grade 50	Not applicable	Groups 1–5	50	65
A913, Grade 60	Not applicable	Groups 1–5	60	75
A913, Grade 65	Not applicable	Groups 1–5	65	80
A913, Grade 70	Not applicable	Groups 1–5	70	90

(*Table continued*)

Table 9.1 (*Continued*)

ASTM Designation	Plate Thickness, in	ANSI/ASTM Group or Weight/ft for Structural Shapes	Yield Point or Yield Strength, ksi	Tensile Strength, ksi
		Heat-Treated Constructional Alloy Steel		
A514	To $2\frac{1}{2}$, incl	Not applicable	100	110–130
	Over $2\frac{1}{2}$ to 6, incl		90	100–130

* Mechanical properties listed are specified minimum values except where a specified range of values (minimum to maximum) is given.
The following properties are approximate values for all the structural steels: modulus of elasticity—29,000 ksi; shear modulus—11,000 ksi; Poisson's ratio—0.30; yield stress in shear—0.57 times yield stress in tension; ultimate strength in shear—$\frac{2}{3}$ to $\frac{3}{4}$ times tensile strength; coefficient of thermal expansion—6.5×10^{-6} in/in/°F for temperature range -50 to $+150$ °F.

A678, Grades A through D, are weldable plate steels furnished in the quenched and tempered condition to provide a minimum yield point of 50 to 75 ksi.

A852 is a quenched and tempered, weathering, plate steel with corrosion resistance similar to that of A588 steel. It has been used for bridges and construction equipment.

A913 is a high-strength low-alloy steel for structural shapes, produced by the quenching and self-tempering process, and intended for buildings, bridges, and other structures. Four grades provide a minimum yield point of 50 to 70 ksi. Maximum carbon equivalents range from 0.38 to 0.45 percent, and the minimum average Charpy V-notch toughness is 40 ft-lb at 70 °F.

Heat-Treated, Constructional-Alloy Steels ▪ Heat-treated steels that contain alloying elements and are suitable for structural applications are called heat-treated, constructional-alloy steels. A514 (Grades A through Q) covers quenched and tempered alloy-steel plates with a minimum yield strength of 90 or 100 ksi.

Bridge Steels ▪ Steels for application in bridges are covered by A709, which includes steel in several of the categories mentioned above. Under this specification, Grades 36, 50, 70, and 100 are steels with yield strengths of 36, 50, 70, and 100 ksi, respectively. The grade designation is followed by the letter W, indicating whether ordinary or high atmospheric-corrosion resistance is required. An

Table 9.2 Wide-Flange Size Groupings for Tensile-Property Classification

Group 1	Group 2	Group 3	Group 4	Group 5
W24 × 55, 62	W40 × 149, 268	W40 × 277–328	W40 × 362–655	W36 × 920
W21 × 44–57	W36 × 135–210	W36 × 230–300	W36 × 328–798	W14 × 605–873
W18 × 35–71	W33 × 118–152	W33 × 201–291	W33 × 318–619	
W16 × 26–57	W30 × 99–211	W30 × 235–261	W30 × 292–581	
W14 × 22–53	W27 × 84–178	W27 × 194–258	W27 × 281–539	
W12 × 14–58	W24 × 68–162	W24 × 176–229	W24 × 250–492	
W10 × 12–45	W21 × 62–147	W21 × 166–223	W21 × 248–402	
W8 × 10–48	W18 × 76–143	W18 × 158–192	W18 × 211–311	
W6 × 9–25	W16 × 67–100	W14 × 145–211	W14 × 233–550	
W5 × 16, 19	W14 × 61–132	W12 × 120–190	W12 × 210–336	
W4 × 13	W12 × 65–106			
	W10 × 49–112			
	W8 × 58, 67			

additional letter, T or F, indicates that Charpy V-notch impact tests must be conducted on the steel. The T designation indicates the material is to be used in a nonfracture-critical application as defined by the American Association of State Highway and Transportation Officials (AASHTO). The F indicates use in a fracture-critical application. A trailing numeral, 1, 2, or 3, indicates the testing zone, which relates to the lowest ambient temperature expected at the bridge site. See Table 9.3. As indicated by the first footnote in the table, the service temperature for each zone is considerably less than the Charpy V-notch impact-test temperature. This accounts for the fact that the dynamic loading rate in the impact test is severer than that to which the structure is subjected. The toughness requirements depend on fracture criticality, grade, thickness, and method of connection. Additionally, A709-HPS70W, designated as a High Performance Steel (HPS), is also available for highway bridge construction. This is a weathering plate steel, designated HPS because it possesses superior weldability and notch toughness as compared to conventional steels of similar strength.

Table 9.3 Charpy V-Notch Toughness for A709 Bridge Steels*

Grade	Max Thickness, in, Inclusive	Joining/ Fastening Method	Min Avg Energy, ft-lb	Test Temp, °F Zone 1	Test Temp, °F Zone 2	Test Temp, °F Zone 3
		Non-Fracture-Critical Members				
36T	4	Mech/Weld	15	70	40	10
50T,† 50WT†	2	Mech/Weld	15	70	40	10
	2 to 4	Mechanical	15			
	2 to 4	Welded	20			
70WT‡	2½	Mech/Weld	20	50	20	−10
	2½ to 4	Mechanical	20			
	2½ to 4	Welded	25			
100T, 100WT	2½	Mech/Weld	25	30	0	−30
	2½ to 4	Mechanical	25			
	2½ to 4	Welded	35			
		Fracture-Critical Members				
36F	4	Mech/Weld	25	70	40	10
50F,† 50WF†	2	Mech/Weld	25	70	40	10
	2 to 4	Mechanical	25			−10
	2 to 4	Welded	30			−10
70WF‡	2½	Mech/Weld	30	50	20	−10
	2½ to 4	Mechanical	30			−10
	2½ to 4	Welded	35			−10
100F, 100WF	2½	Mech/Weld	35	30	0	−30
	2½ to 4	Mechanical	35			−30
	2½ to 4	Welded	45			NA

* Minimum service temperatures: Zone 1, 0 °F; Zone 2, <0 to −30 °F; Zone 3, <−30 to −60 °F.
† If yield strength exceeds 65 ksi, reduce test temperature by 15 °F for each 10 ksi above 65 ksi.
‡ If yield strength exceeds 85 ksi, reduce test temperature by 15 °F for each 10 ksi above 85 ksi.

Lamellar Tearing ▪ The information on strength and ductility presented generally pertains to loadings applied in the planar direction (longitudinal or transverse orientation) of the steel plate or shape. Note that elongation and area-reduction values may well be significantly lower in the through-thickness direction than in the planar direction. This inherent directionality is of small consequence in many applications, but it does become important in the design and fabrication of structures containing massive members with highly restrained welded joints.

With the increasing trend toward heavy welded-plate construction, there has been a broader recognition of occurrences of lamellar tearing in some highly restrained joints of welded structures, especially those in which thick plates and heavy structural shapes are used. The restraint induced by some joint designs in resisting weld-deposit shrinkage can impose tensile strain high enough to cause separation or tearing on planes parallel to the rolled surface of the structural member being joined.

The incidence of this phenomenon can be reduced or eliminated through use of techniques based on greater understanding by designers, detailers, and fabricators of the (1) inherent directionality of constructional forms of steel, (2) high restraint developed in certain types of connections, and (3) need to adopt appropriate weld details and welding procedures with proper weld metal for through-thickness connections. Furthermore, steels can be specified to be produced by special practices or processes to enhance through-thickness ductility and thus assist in reducing the incidence of lamellar tearing.

However, unless precautions are taken in both design and fabrication, lamellar tearing may still occur in thick plates and heavy shapes of such steels at restrained through-thickness connections. Some guidelines for minimizing potential problems have been developed by the American Institute of Steel Construction (AISC). (See "The Design, Fabrication, and Erection of Highly Restrained Connections to Minimize Lamellar Tearing," *AISC Engineering Journal*, vol. 10, no. 3, 1973, www.aisc.org.)

Welded Splices in Heavy Sections ▪ Shrinkage during solidification of large welds causes strains in adjacent restrained material that can exceed the yield-point strain. In thick material, triaxial stresses may develop because there is restraint in the thickness direction as well as the planar directions. Such conditions inhibit the ability of the steel to act in a ductile manner and increase the possibility of brittle fracture. Therefore, for building construction, AISC imposes special requirements when splicing either Group 4 or Group 5 rolled shapes, or shapes built up by welding plates more than 2 in thick, if the cross section is subject to primary tensile stresses due to axial tension or flexure. Included are notch toughness requirements, the removal of weld tabs and backing bars (ground smooth), generous-sized weld access holes, preheating for thermal cutting, and grinding and inspecting cut edges. Even when the section is used as a primary compression member, the same precautions must be taken for sizing the weld access holes, preheating, grinding, and inspection. See the AISC Specification for further details.

Cracking ▪ An occasional problem known as "k-area cracking" has been identified. Wide flange sections are typically straightened as part of the mill production process. Often a rotary straightening process is used, although some heavier members may be straightened in a gag press. Some reports in recent years have indicated a potential for crack initiation at or near connections in the "k" area of wide flange sections that have been rotary straightened. The k area is the region extending from approximately the midpoint of the web-to-flange fillet, into the web for a distance approximately 1 to 1-½ in. beyond the point of tangency. Apparently, in some cases, this limited region had a reduced notch toughness due to cold working and strain hardening. Most of the incidents reported occurred at highly restrained joints with welds in the "k" area. However, the number of examples reported has been limited and these have occurred during construction or laboratory tests, with no evidence of difficulties with steel members in service. Research has confirmed the need to avoid welding in the "k" area. AISC issued the following recommendations concerning fabrication and design practices for rolled wide flange shapes:

- Welds should be stopped short of the "k" area for transverse stiffeners (continuity plates).

- For continuity plates, fillet welds and/or partial joint penetration welds, proportioned to transfer the calculated stresses to the column web, should be considered instead of complete jount penetration welds. Weld volume should be minimized.
- Residual stresses in highly restrained joints may be decreased by increased preheat and proper weld sequencing.
- Magnetic particle or dye penetrant inspection should be considered for weld areas in or near the "k" area of highly restrained connections after the final welding has completely cooled.
- When possible, eliminate the need for column web doubler plates by increasing column size.

Good fabrication and quality control practices, such as inspection for cracks, gouges, etc., at flame-cut access holes or copes, should continue to be followed and any defects repaired and ground smooth. All structural wide flange members for normal service use in building construction should continue to be designed per AISC Specifications and the material furnished per ASTM standards.

(AISC Advisory Statement, *Modern Steel Construction*, February 1997.)

Fasteners ▪ Steels for structural bolts are covered by A307, A325, and A490 Specifications. A307 covers carbon-steel bolts for general applications, such as low-stress connections and secondary members. Specification A325 includes two type of quenched and tempered high-strength bolts for structural steel joints: Type 1—medium-carbon, carbon-boron, or medium-carbon alloy steel, and Type 3—weathering steel with atmospheric corrosion resistance similar to that of A588 steel. A previous Type 2 was withdrawn in 1991.

Specification A490 includes three types of quenched and tempered high-strength steel bolts for structural-steel joints: Type 1—bolts made of alloy steel; Type 2—bolts made from low-carbon martensite steel, and Type 3—bolts having atmospheric-corrosion resistance and weathering characteristics comparable to that of A588, A242, and A709 (W) steels. Type 3 bolts should be specified when atmospheric-corrosion resistance is required. Hot-dip galvanized A490 bolts should not be used.

Bolts having diameters greater than 1½ in are available under Specification A449.

Rivets for structural fabrication were included under Specification A502 but this designation has been discontinued.

9.3 Structural-Steel Shapes

Most structural steel used in building construction is fabricated from rolled shapes. In bridges, greater use is made of plates since girders spanning over about 90 ft are usually built-up sections.

Many different rolled shapes are available: W shapes (wide-flange shapes), M shapes (miscellaneous shapes), S shapes (standard I sections), angles, channels, and bars. The "Manual of Steel Construction," American Institute of Steel Construction, lists properties of these shapes.

Wide-flange shapes range from a W4 × 13 (4 in deep weighing 13 lb/lin ft) to a W36 × 920 (36 in deep weighing 920 lb/lin ft). "Jumbo" column sections range up to W14 × 873.

In general, wide-flange shapes are the most efficient beam section. They have a high proportion of the cross-sectional area in the flanges and thus a high ratio of section modulus to weight. The 14-in W series includes shapes proportioned for use as column sections; the relatively thick web results in a large area-to-depth ratio.

Since the flange and web of a wide-flange beam do not have the same thickness, their yield points may differ slightly. In accordance with design rules for structural steel based on yield point, it is therefore necessary to establish a "design yield point" for each section. In practice, all beams rolled from A36 steel (Art. 9.2) are considered to have a yield point of 36 ksi. Wide-flange shapes, plates, and bars rolled from higher-strength steels are required to have the minimum yield and tensile strength shown in Table 9.1.

Square, rectangular, and round structural tubular members are available with a variety of yield strengths. Suitable for columns because of their symmetry, these members are particularly useful in low buildings and where they are exposed for architectural effect.

Connection Material ▪ Connections are normally made with A36 steel. If, however, higher-strength steels are used, the structural size groupings for angles and bars are:

Group 1: Thicknesses of ½ in or less

Group 2: Thicknesses exceeding ½ in but not more than ¾ in
Group 3: Thicknesses exceeding ¾ in

Structural tees fall into the same group as the wide-flange or standard sections from which they are cut. (A WT7 × 13, for example, designates a tee formed by cutting in half a W 14 × 26 and therefore is considered a Group 1 shape, as is a W 14 × 26.)

9.4 Selecting Structural Steels

The following guidelines aid in choosing between the various structural steels. When possible, a more detailed study that includes fabrication and erection cost estimates is advisable.

A basic index for cost analysis is the cost-strength ratio, p/F_y, which is the material cost, cents per pound, divided by the yield point, ksi. For tension members, the relative material cost of two members, C_2/C_1, is directly proportional to the cost-strength ratios; that is,

$$\frac{C_2}{C_1} = \frac{p_2/F_{y2}}{p_1/F_{y1}} \tag{9.1a}$$

For bending members, the relationship depends on the ratio of the web area to the flange area and the web depth-to-thickness ratios. For fabricated girders of optimum proportions (half the total cross-sectional area is the web area),

$$\frac{C_2}{C_1} = \frac{p_2}{p_1}\left(\frac{F_{y1}}{F_{y2}}\right)^{1/2} \tag{9.1b}$$

For hot-rolled beams,

$$\frac{C_2}{C_1} = \frac{p_2}{p_1}\left(\frac{F_{y1}}{F_{y2}}\right)^{2/3} \tag{9.1c}$$

For compression members, the relation depends on the allowable buckling stress F_c, which is a function of the yield point directly; that is,

$$\frac{C_2}{C_1} = \frac{F_{c1}/p_1}{F_{c2}/p_2} \tag{9.1d}$$

Thus, for short columns, the relationship approaches that for tension members. Table 9.4 gives ratios of F_c that can be used, along with typical material prices p from producing mills, to calculate relative member costs.

Higher strength steels are often used for columns in buildings, particularly for the lower floors when the slenderness ratios is less than 100. When bending is dominant, higher strength steels are economical where sufficient lateral bracing is present. However, if deflection limits control, there is no advantage over A36 steel.

On a piece-for-piece basis, there is substantially no difference in the cost of fabricating and erecting the different grades. Higher-strength steels, however, may afford an opportunity to reduce the number of members, thus reducing both fabrication and erection costs.

9.5 Tolerances for Structural Shapes

ASTM Specification A6 lists mill tolerances for rolled-steel plates, shapes, sheet piles, and bars. Included are tolerances for rolling, cutting, section

Table 9.4 Ratio of Allowable Stress in Columns of High-Strength Steel to That of A36 Steel

Specified Yield Strength F_y, ksi	Slenderness Ratio Kl/r											
	5	15	25	35	45	55	65	75	85	95	105	115
65	1.80	1.78	1.75	1.72	1.67	1.62	1.55	1.46	1.35	1.22	1.10	1.03
60	1.66	1.65	1.63	1.60	1.56	1.52	1.47	1.40	1.32	1.21	1.10	1.03
55	1.52	1.51	1.50	1.48	1.45	1.42	1.38	1.33	1.27	1.20	1.10	1.03
50	1.39	1.38	1.37	1.35	1.34	1.32	1.29	1.26	1.22	1.17	1.10	1.03
45	1.25	1.24	1.24	1.23	1.22	1.21	1.19	1.17	1.15	1.12	1.08	1.03
42	1.17	1.16	1.16	1.15	1.15	1 14	1.13	1.12	1.10	1.08	1.06	1.03

area, and weight, ends out of square, camber, and sweep. The "Manual of Steel Construction" contains tables for applying these tolerances.

The AISC "Code of Standard Practice" gives fabrication and erection tolerances for structural steel for buildings. Figures 9.2 and 9.3 show permissible tolerances for column erection for a multistory building. In these diagrams, a working point for a column is the actual center of the member at each end of a shipping piece. The working line is a straight line between the member's working points.

Both mill and fabrication tolerances should be considered in designing and detailing structural steel. A column section, for instance, may have an actual depth greater or less than the nominal depth. An accumulation of dimensional variations, therefore, would cause serious trouble in erection of a building with many bays. Provision should be made to avoid such a possibility.

Tolerances for fabrication and erection of bridge girders are usually specified by highway departments.

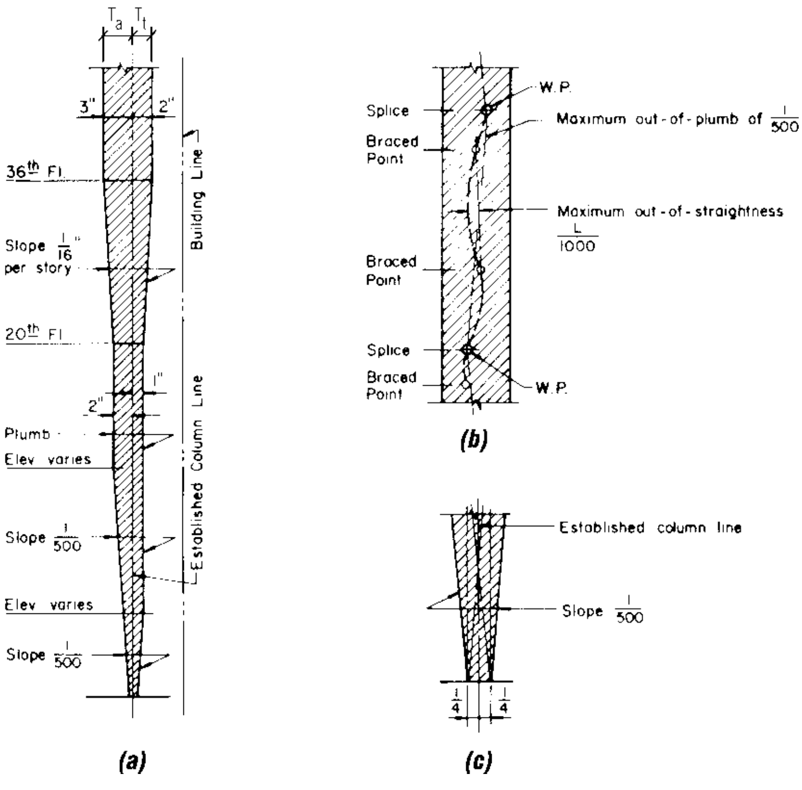

Fig. 9.2 Tolerances permitted for exterior columns for plumbness normal to the building line. (*a*) Envelope within which all working points must fall. (*b*) For individual column sections lying within the envelope shown in (*a*), maximum out-of-plumb of an individual shipping piece, as defined by a straight line between working points, is 1/500 and the maximum out-of-straightness between braced points is $L/1000$, where L is the distance between braced points. (*c*) Tolerance for the location of a working point at a column base. The plumb line through that point is not necessarily the precise plan location, inasmuch as the 2000 AISC "Code of Standard Practice" deals only with plumbness tolerance and does not include inaccuracies in location of established column lines, foundations, and anchor bolts beyond the erector's control.

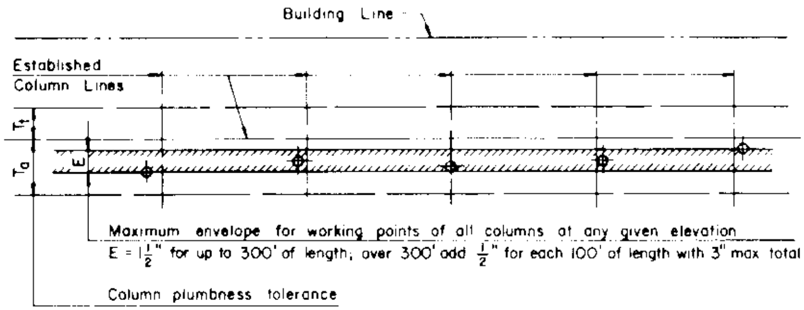

Fig. 9.3 Tolerance in plan permitted for exterior columns at any splice level. Circles indicate column working points. At any splice level, the horizontal envelope defined by E lies within the distances T_a and T_t from the established column line (Fig. 9.2*a*). Also, the envelope E may be offset from the corresponding envelope at the adjacent splice levels, above and below, by a distance not more than $L/500$, where L is the column length. Maximum E is 1½ in for buildings up to 300 ft long. E may be increased by ½ in for each additional 100 ft of length but not to more than 3 in.

9.6 Structural-Steel Design Specifications

The design of practically all structural steel for buildings in the United States is based on two specifications of the American Institute of Steel Construction. AISC has long maintained a traditional allowable-stress design (ASD) specification, including a comprehensive revised specification issued in 1989, "Specification for Structural Steel for Buildings—Allowable Stress Design and Plastic Design." AISC also publishes an LRFD specification, "Load and Resistance Factor Design Specification for Structural Steel for Buildings." Other important design specifications published by AISC include "Seismic Provisions for Structural Steel Buildings," "Specification for the Design of Steel Hollow Structural Sections," "Specification for the Design, Fabrication and Erection of Steel Safety Related Structures for Nuclear Facilities," and "Specification for Load and Resistance Factor Design of Single-Angle members."

Design rules for bridges are given in "Standard Specifications for Highway Bridges," (American Association of State Highway and Transportation Officials, N. Capitol St, Suite 249 N.W., Washington, DC 20001, www.ashto.org). They are somewhat more conservative than the AISC Specifications. AASHTO gives both an allowable-stress method and a load-factor method. However, the most recent developments in bridge design are reflected in the AASHTO publication. "LRFD Bridge Design Specifications."

Other important specifications for the design of steel structures include the following:

The design of structural members cold-formed from steel not more than 1 in thick follows the rules of AISI "Specification for the Design of Cold-Formed Steel Structural Members" (American Iron and Steel Institute, 1101 17th St., N.W., Washington, DC 20036-4700, www.aisc.org. See Sec. 10).

Codes applicable to welding steel for bridges, buildings, and tubular members are offered by AWS (American Welding Society, 550 N.W. LeJone Road, Miami, FL 33126).

Rules for the design, fabrication, and erection of steel railway bridges are developed by AREMA (American Railway Engineering and Maintenance-of-Way Association, 8201 Corporate Drive, Suite 1125, Landover, Md., 20785-2230). See Sec. 17.

Specifications covering design, manufacture, and use of open-web steel joists are available from SJI (Steel Joist Institute, www.steeljoist). See Sec. 10.

9.7 Structural-Steel Design Methods

Structural steel for buildings may be designed by either the allowable-stress design (ASD) or load-and-resistance-factor design (LRFD) method

(Art. 9.6). The ASD Specification of the American Institute of Steel Construction follows the usual method of specifying allowable stresses that represent a "failure" stress (yield stress, buckling stress, etc.) divided by a safety factor. In the AISC-LRFD Specification, both the applied loads and the calculated strength or resistance of members are multiplied by factors. The load factors reflect uncertainties inherent in load determination and the likelihood of various load combinations. The resistance factors reflect variations in determining strength of members such as uncertainty in theory and variations in material properties and dimensions. The factors are based on probabilistic determinations, with the intent of providing a more rational approach and a design with a more uniform reliability. In general, the LRFD method can be expected to yield some savings in material requirements but may require more design time.

Factors to be applied to service loads for various loading combinations are given in Art. 15.5. Rules for "plastic design" are included in both specifications. This method may be applied for steels with yield points of 65 ksi or less used in braced and unbraced planar frames and simple and continuous beams. It is based on the ability of structural steel to deform plastically when strained past the yield point, thereby developing plastic hinges and redistributing loads (Art. 6.65). The hinges are not anticipated to form at service loads but at the higher factored loads.

Steel bridge structures may be designed by ASD, LFD, or LRFD methods in accordance with the specifications of the American Association of State Highway and Transportation Officials (AASHTO). With the load-factor design (LFD) method, only the loads are factored, but with the load-and-resistance-factor (LRFD) method, factors are applied to both loads and resistances. For load factors for highway bridges, see Art. 17.3. Railroad bridges are generally designed by the ASD method.

9.8 Dimensional Limitations on Steel Members

Design specifications, such as the American Institute of Steel Construction "Specification for Structural Steel Buildings—Allowable Stress Design and Plastic Design" and "Load and Resistance Factor Design for Structural Steel Buildings" and the American Association of State Highway and Transportation Officials "Standard Specifications for Highway Bridges" and "LRFD Bridge Design Specifications" set limits, maximum and minimum, on the dimensions and geometry of structural-steel members and their parts. The limitations generally depend on the types and magnitudes of stress imposed on the members and may be different for allowable-stress design (ASD) and load-and-resistance-factor design (LRFD).

These specifications require that the structure as a whole and every element subject to compression be constructed to be stable under all possible combinations of loads. The effects of loads on all parts of the structure when members or their components deform under loads or environmental conditions should be taken into account in design and erection.

(T. V. Galambos, "Guide to Stability Design Criteria for Metal Structures," 5th ed., John Wiley & Sons, Inc., New York.)

Vibration Considerations ▪ In large open areas of buildings, where there are few partitions or other sources of damping, transient vibrations caused by pedestrian traffic may become annoying. Beams and slender members supporting such areas should be designed with due regard for stiffness and damping. Special attention to vibration control should be given in design of bridges because of their exposure to wind, significant temperature changes, and variable, repeated, impact and dynamic loads. Some of the restrictions on member dimensions in standard building and bridge design specifications are intended to limit amplitudes of vibrations to acceptable levels.

Minimum Thickness ▪ Floor plates in buildings may have a nominal thickness as small as $\frac{1}{8}$ in. Generally, minimum thickness available for structural-steel bars 6 in or less wide is 0.203 in and for bars 6 to 8 in wide, 0.230 in. Minimum thickness for plates 8 to 48 in wide is 0.230 in and for plates over 48 in wide, 0.180 in.

The AASHTO Specification requires that, except for webs of certain rolled shapes, closed ribs in orthotropic-plate decks, fillers, and railings, structural-steel elements be at least $\frac{5}{16}$ in thick. Web thickness of rolled beams may be as small as 0.23 in. Thickness of closed ribs in orthotropic-plate decks should be at least $\frac{3}{16}$ in. No minimum is established for fillers. The American Railway

Engineering and Maintenance-of-Way Association "Manual for Railway Engineering" requires that bridge steel, except for fillers, be at least 0.335 in thick. Gusset plates connecting chords and web members of trusses should be at least ½ in thick. In any case, where the steel will be exposed to a substantial corrosive environment, the minimum thicknesses should be increased or the metal should be protected.

Maximum Slenderness Ratios ▪ The AISC Specifications require that the slenderness ratio, the ratio of effective length to radius of gyration of the cross section, should not exceed 200 for members subjected to compression in buildings. For steel highway bridges the AASHTO Specification limits slenderness ratios for compression members to a maximum of 120 for main members and 140 for secondary members and bracing. The AREMA Manual lists the following maximum values for slenderness ratios for compression members in bridges: 100 for main members, 120 for wind and sway bracing, 140 for single lacing, and 200 for double lacing.

For members in tension, the AISC Specifications limit slenderness ratio to a maximum of 300 in buildings. For tension members other than rods, eyebars, cables, and plates, AASHTO specifies for bridges a maximum ratio of unbraced length to radius of gyration of 200 for main tension members, 240 for bracing, and 140 for main-members subject to stress reversal. The AREMA Manual limits the ratio for tension members to 200 for bridges.

Compact Sections ▪ The AISC and AASHTO specifications classify structural-steel sections as compact, noncompact, slender, or hybrid. Slender members have elements that exceed the limits on width-thickness ratios for compact and noncompact sections and are designed with formulas that depend on the difference between actual width-thickness ratios and the maximum ratios permitted for noncompact sections. Hybrid beams or girders have flanges made of steel with yield strength different from that for the webs.

For a specific cross-sectional area, a compact section generally is permitted to carry heavier loads than a noncompact one of similar shape. Under loads stressing the steel into the plastic range, compact sections should be capable of forming plastic hinges with a capacity for inelastic rotation at least three times the elastic rotation corresponding to the plastic moment. To qualify as compact, a section must have flanges continuously connected to the webs, and thickness of its elements subject to compression must be large enough to prevent local buckling while developing a fully plastic stress distribution.

Tables 9.5 and 9.6 present, respectively, maximum width-thickness ratios for structural-steel compression elements in buildings and highway bridges. See also Arts. 9.12 and 9.13.

9.9 Allowable Tension in Steel

For buildings, AISC specifies a basic allowable unit tensile stress, ksi, $F_t = 0.60F_y$, on the gross cross section area, where F_y is the yield strength of the steel, ksi (Table 9.7). F_t is subjected to the further limitation that it should not exceed on the net cross section area, one-half the specified minimum tensile strength F_u of the material. On the net section through pinholes in eyebars, pin-connected plates, or built-up members, $F_t = 0.45F_y$.

For bridges, AASHTO specifies allowable tensile stresses as the smaller of $0.55F_y$ on the gross section, or $0.50F_u$ on the net section ($0.46F_y$ for 100 ksi yield strength steels), where F_u = tensile strength (Table 9.7). In determining gross area, area of holes for bolts and rivets must be deducted if over 15 percent of the gross area. Also, open holes larger than 1¼ in, such as perforations, must be deducted.

Table 9.7 and subsequent tables apply to two strength levels, $F_y = 36$ ksi and $F_y = 50$ ksi, the ones generally used for construction.

The net section for a tension member with a chain of holes extending across a part in any diagonal or zigzag line is defined in the AISC Specification as follows: The net width of the part shall be obtained by deducting from the gross width the sum of the diameters of all the holes in the chain and adding, for each gage space in the chain, the quantity $s^2/4g$, where s = longitudinal spacing (pitch), in, of any two consecutive holes and g = transverse spacing (gage), in, of the same two holes. The critical net section of the part is obtained from the chain that gives the least net width.

Table 9.5 Maximum Width-Thickness Ratios b/t[a] for Compression Elements for Buildings[b]

Description of Element	ASD and LRFD[c]	ASD[c]	LRFD[c]
	Compact—λ_p	Noncompact[d]	Noncompact—λ_r
Projecting flange element of I-shaped rolled beams and channels in flexure	$65/\sqrt{F_y}$	$95/\sqrt{F_y}$	$141/\sqrt{F_L}$[g]
Projecting flange element of I-shaped hybrid or welded beams in flexure	$65/\sqrt{F_y}$	$95/\sqrt{F_{yt}/k_c}$[e]	$162/\sqrt{F_L/k_{cc}}$[h]
Projecting flange element of I-shaped sections in pure compression, plates projecting from compression elements; outstanding legs of pairs of angles in continuous contact; flanges of channels in pure compression	Not specified	$95/\sqrt{F_y}$	$95/\sqrt{F_y}$
Flanges of square and rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds	$190/\sqrt{F_y}$ uniform comp. $160/\sqrt{F_y}$ plastic anal.	$238/\sqrt{F_y}$	$238/\sqrt{F_y}$
Unsupported width of cover plates perforated with a succession of access holes	Not specified	$317/\sqrt{F_y}$	$317/\sqrt{F_y}$
Legs of single-angle struts; legs of double-angle struts with separators; unstiffened elements; i.e., supported along one edge	Not specified	$76/\sqrt{F_y}$	$76/\sqrt{F_y}$
Stems of tees	Not specified	$127/\sqrt{F_y}$	$127/\sqrt{F_y}$
All other uniformly compressed stiffened elements; i.e., supported along two edges	Not specified	$253/\sqrt{F_y}$	$253/\sqrt{F_y}$
Webs in flexural compression[a]	$640/\sqrt{F_y}$	$760/\sqrt{F_y}$	$970/\sqrt{F_y}$
D/t for circular hollow sections[f]			
In axial compression for ASD	$3{,}300/F_y$		
In flexure for ASD	$3{,}300/F_y$		

(*Table continued*)

Table 9.5 (*Continued*)

Description of Element	ASD and LRFD[c] Compact—λ_p	ASD[c] Noncompact[d]	LRFD[c] Noncompact—λ_r
In axial compression for LRFD		Not specified	$3{,}190/F_y$
In flexure for LRFD	$2{,}030/F_y$		$8{,}990/F_y$
In plastic design for LRFD	$1{,}300/F_y$		

[a] b = width of element or projection (half the nominal width of rolled beams and tees; full width of angle legs and Z and channel flanges). For webs in flexural compression, b should be taken as h, the clear distance between flanges (less fillets for rolled shapes) or distance between adjacent lines of fasteners; t should be taken as t_w, web thickness.
[b] As required in AISC Specifications for ASD and LRFD. These specifications also set specific limitations on plate-girder components.
[c] F_y = specified minimum yield stress of the steel, ksi, but for hybrid beams, use F_{yt}, the yield strength, ksi, of flanges; F_b = allowable bending stress, ksi, in the absence of axial force; F_r = compressive residual stress in flange, ksi (10 ksi for rolled shapes, 16.5 ksi for welded shapes).
[d] Elements with width-thickness ratios that exceed the noncompact limits should be designed as slender sections.
[e] $k_c = 4.05/(h/t)^{0.46}$ for $h/t > 70$; otherwise $k_c = 1$.
[f] D = outside diameter; t = section thickness.
[g] F_L = smaller of $(F_{yf} - F_r)$ or F_{yw}, ksi; F_{yf} = yield strength, ksi, of flanges and F_{yw} = yield strength, ksi, of web.
[h] $k_{cc} = 4/(h/t)^{0.46}$ and $0.35 \le k_{cc} \le 0.763$.

For splice and gusset plates and other connection fittings, the design area for the net section taken through a hole should not exceed 85% of the gross area. When the load is transmitted through some but not all of the cross-sectional elements—for example, only through the flanges of a W shape—an effective net area should be used (75 to 90% of the calculated net area).

LRFD for Tension in Buildings ▪ The limit states for yielding of the gross section and fracture in the net section should be investigated. For yielding, the design tensile strength P_u, ksi, is given by

$$P_u = 0.90 F_y A_g \tag{9.2}$$

where F_y = specified minimum yield stress, ksi

A_g = gross area of tension member, in^2

For fracture,

$$P_u = 0.75 F_u A_e \tag{9.3}$$

where F_u = specified minimum tensile strength, ksi

A_e = effective net area, in^2

In determining A_e for members without holes, when the tension load is transmitted by fasteners or welds through some but not all of the cross-sectional elements of the member, a reduction factor U is applied to account for shear lag. The factor ranges from 0.75 to 1.00.

9.10 Allowable Shear in Steel

The AASHTO "Standard Specification for Highway Bridges" (Art. 9.6) specifies an allowable shear stress of $0.33F_y$, where F_y is the specified minimum yield stress of the web. Also see Art. 9.10.2. For buildings, the AISC Specification for ASD (Art. 9.6.) relates the allowable shear stress in flexural members to the depth-thickness ratio, h/t_w, where t_w is the web thickness and h is the clear distance between flanges or between adjacent lines of fasteners for built-up sections. In design of girders, other than hybrid girders, larger shears may be allowed when intermediate stiffeners are used. The stiffeners permit tension-field action; that is, a strip of web acting as a tension diagonal resisted by the transverse stiffeners acting as struts, thus enabling the web to carry greater shear.

9.10.1 ASD for Shear in Buildings

The AISC Specification for ASD specifies the following allowable shear stresses F_v, ksi:

$$F_v = 0.40F_y \quad h/t_w \le 380/\sqrt{F_y} \tag{9.4}$$

$$F_v = C_v F_y/2.89 \le 0.40F_y \quad h/t_w > 380/\sqrt{F_y} \tag{9.5}$$

Table 9.6 Maximum Width-Thickness Ratios b/t[a] for Compression Elements for Highway Bridges[b]

Load-and-Resistance-Factor Design[c]			
Description of Element	Compact	Noncompact[d]	
Flange projection of rolled or fabricated I-shaped beams	$65/\sqrt{F_y}$	$235\sqrt{\dfrac{1}{f_c\sqrt{\frac{2D_c}{t_w}}}}$[h]	
Webs in flexural compression without longitudinal stiffeners	$640/\sqrt{F_y}$	$\dfrac{2D_c}{t_w} = \dfrac{1150}{\sqrt{f_c}}$	
Allowable-Stress Design[e]			
Description of Element (Compression Members)	$f_a < 0.44\,F_y$	$f_a = 0.44\,F_y$	
		$F_y = 36$ ksi	$F_y = 50$ ksi
Plates supported on one side and outstanding legs of angles			
In main members	$51/\sqrt{f_a} \leq 12$	12	11
In bracing and other secondary members	$51/\sqrt{f_a} \leq 16$	12	11
Plates supported on two edges or webs of box shapes[f]	$126/\sqrt{f_a} \leq 45$	32	27
Solid cover plates supported on two edges or solid webs[g]	$158/\sqrt{f_a} \leq 50$	40	34
Perforated cover plates supported on two edges for box shapes	$190/\sqrt{f_a} \leq 55$	48	41

[a] b = width of element or projection; t = thickness. The point of support is the inner line of fasteners or fillet welds connecting a plate to the main segment or the root of the flange of rolled shapes. In LRFD, for webs of compact sections, $b = d$, the beam depth, and for noncompact sections, $b = D$, the unsupported distance between flange components.

[b] As required in AASHTO "Standard Specification for Highway Bridges." The specifications also provide special limitations on plate-girder elements.

[c] F_y = specified minimum yield stress, ksi, of the steel.

[d] Elements with width-thickness ratios that exceed the noncompact limits should be designed as slender elements.

[e] f_a = computed axial compression stress, ksi.

[f] For box shapes consisting of main plates, rolled sections, or component segments with cover plates.

[g] For webs connecting main members or segments for H or box shapes.

[h] D_c = depth of web in compression, in; f_c = stress in compression flange, ksi, due to factored loads; t_w = web thickness, in.

$$\begin{aligned} \text{where } C_v &= 45{,}000k_v/F_y(h/t_w)^2 && \text{for } C_v < 0.8 \\ &= \sqrt{36{,}000k_v/F_y(h/t_w)^2} && \text{for } C_v > 0.8 \\ k_v &= 4.00 + 5.34/(a/h)^2 && \text{for } a/h < 1.0 \\ &= 5.34 + 4.00/(a/h)^2 && \text{for } a/h > 1.0 \end{aligned}$$

a = clear distance between transverse stiffeners

The allowable shear stress with tension-field action is

$$F_v = \frac{F_y}{289}\left[C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}}\right] \leq 0.40F_y \qquad (9.6)$$

$C_v \leq 1$

Table 9.7 Allowable Tensile Stresses in Steel for Buildings and Bridges, ksi

	Buildings		Bridges	
Yield Strength	On Gross Section	On Net Section*	On Gross Section	On Net Section*
36	22.0	29.0	20.0	29.0
50	30.0	32.5	27.5	32.5

* Based on A36 and A572 Grade 50 steels with $F_u = 58$ ksi and 65 ksi, respectively.

When the shear in the web exceeds F_v, stiffeners are required. See also Art. 9.13.

The area used to compute shear stress in a rolled beam is defined as the product of the web thickness and the overall beam depth. The webs of all rolled structural shapes are of such thickness that shear is seldom the criterion for design.

At beam-end connections where the top flange is coped, and in similar situations in which failure might occur by shear along a plane through the fasteners or by a combination of shear along a plane through the fasteners and tension along a perpendicular plane, AISC employs the **block shear** concept. The load is assumed to be resisted by a shear stress of $0.30F_u$ along a plane through the net shear area and a tensile stress of $0.50F_u$ on the net tension area, where F_u is the minimum specified tensile strength of the steel.

Within the boundaries of a rigid connection of two or more members with webs lying in a common plane, shear stresses in the webs generally are high. The Commentary on the AISC Specification for buildings states that such webs should be reinforced when the calculated shear stresses, such as those along plane *AA* in Fig. 9.4, exceed F_v; that is, when ΣF is larger than $d_c t_w F_v$, where d_c is the depth and t_w is the web thickness of the member resisting ΣF. The shear may be calculated from

$$\Sigma F = \frac{M_1}{0.95d_1} + \frac{M_2}{0.95d_2} - V_s \qquad (9.7)$$

where V_s = shear on the section

$M_1 = M_{1L} + M_{1G}$

M_{1L} = moment due to the gravity load on the leeward side of the connection

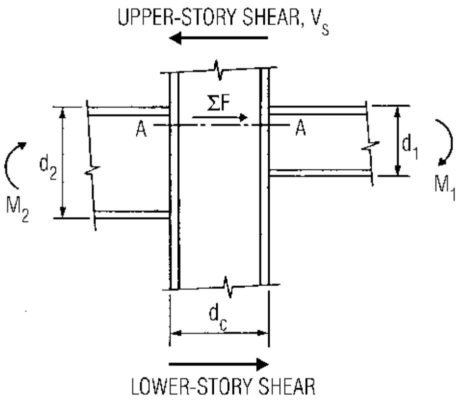

Fig. 9.4 Rigid connection of steel members with webs in a common plane.

M_{1G} = moment due to the lateral load on the leeward side of the connection

$M_2 = M_{2L} - M_{2G}$

M_{2L} = moment due to the lateral load on the windward side of the connection

M_{2G} = moment due to the gravity load on the windward side of the connection

9.10.2 ASD for Shear in Bridges

Based on the AASHTO Specification for Highway Bridges, transverse stiffeners are required where h/t_w exceeds 150 and must not exceed a spacing, a, of $3h$, where h is the clear unsupported distance between flange components, t_w is the web thickness, and all dimensions are in inches. Where transverse stiffeners are required, the allowable shear stress, ksi, may be computed from

$$F_v = \frac{F_y}{3}\left[C + \frac{0.87(1-C)}{\sqrt{1-(a/h)^2}}\right] \qquad (9.8)$$

where $C = 1.0$ when $\dfrac{h}{t_w} < \dfrac{190\sqrt{k}}{\sqrt{F_y}}$

$$C = \frac{190\sqrt{k}}{(h/t_w)\sqrt{F_y}} \quad \text{when } \frac{190\sqrt{k}}{\sqrt{F_y}} \le \frac{h}{t_w} \le \frac{237\sqrt{k}}{\sqrt{F_y}}$$

$$C = \frac{45{,}000\sqrt{k}}{(h/t_w)^2\sqrt{F_y}} \quad \text{when } \frac{h}{t_w} > \frac{237\sqrt{k}}{\sqrt{F_y}}$$

See also Art. 9.13.

9.10.3 LRFD for Shear in Buildings

Based on the AISC Specifications for LRFD for buildings, the shear capacity V_u, kips, of flexural members with unstiffened webs may be computed from the following:

$$V_u = 0.54F_{yw}A_w \quad \text{when } h/t_w = 417\sqrt{1/F_{yw}} \tag{9.9}$$

$$V_u = 0.54F_{yw}A_w\left(\frac{417\sqrt{1/F_{yw}}}{h/t_w}\right)$$
$$\text{when } 417\sqrt{1/F_{yw}} < h/t_w \le 523\sqrt{1/F_{yw}} \tag{9.10}$$

$$V_u = A_w\left[\frac{131{,}000}{(h/t_w)^2}\right] \quad \text{when } 523\sqrt{1/F_{yw}} < h/t_w \le 260 \tag{9.11}$$

where F_{yw} = specified minimum yield stress of web, ksi

A_w = web area, in$^2 = dt_w$

Stiffeners are required when the shear exceeds V_u (Art. 9.13). In unstiffened girders, h/t_w may not exceed 260. For shear capacity with tension-field action, see the AISC Specification for LRFD.

9.10.4 LFD Shear Strength Design for Bridges

Based on the AASHTO Specifications for load-factor design, the shear capacity, kips, may be computed from:

$$V_u = 0.58F_y h t_w C \tag{9.12a}$$

for flexural members with unstiffened webs with $h/t_w < 150$ or for girders with stiffened webs but a/h exceeding 3 or $67{,}600(h/t_w)^2$.

$$C = 1.0 \quad \text{when } \frac{k}{t_w} < \beta$$

$$= \frac{\beta}{h/t_w} \quad \text{when } \beta \le \frac{h}{t_w} \le 1.25\beta$$

$$= \frac{45{,}000k}{F_y(h/t_w)^2} \quad \text{when } \frac{h}{t_w} > 1.25\beta$$

where $\beta = 190\sqrt{k/F_y}$

$k = 5$ for unstiffened webs

$k = 5 + \lfloor 5/(a/h)^2 \rfloor$ for stiffened webs

For girders with transverse stiffeners and a/h less than 3 and $67{,}600(h/t_w)^2$, the shear capacity is given by

$$V_u = 0.58F_y d t_w\left[C + \frac{1-C}{1.15\sqrt{1+(a/h)^2}}\right] \tag{9.12b}$$

Stiffeners are required when the shear exceeds V_u (Art. 9.13).

9.11 Allowable Compression in Steel

The allowable compressive load or unit stress for a column is a function of its slenderness ratio. The slenderness ratio is defined as Kl/r, where K = effective-length factor, which depends on restraints at top and bottom of the column; l = length of column between supports, in; and r = radius of gyration of the column section, in. For combined compression and bending, see Art. 9.17. For maximum permissible slenderness ratios, see Art. 9.8. Columns may be designed by allowable-stress design (ASD) or load-and-resistance-factor design (LRFD).

9.11.1 ASD for Building Columns

The AISC Specification for ASD for buildings (Art. 9.7) provides two formulas for computing allowable compressive stress F_a, ksi, for main members. The formula to use depends on the relationship of the largest effective slenderness ratio Kl/r of the cross section of any unbraced segment to a factor C_c defined by Eq. (9.13*a*). See Table 9.8*a*.

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} = \frac{756.6}{\sqrt{F_y}} \tag{9.13a}$$

where E = modulus of elasticity of steel = 29,000 ksi

F_y = yield stress of steel, ksi

Table 9.8a Values of C_c

F_y	C_c
36	126.1
50	107.0

Table 9.8b Allowable Stresses F_a, ksi, in Steel Building Columns for $Kl/r \leq 120$

Kl/r	Yield Strength of Steel F_y, ksi	
	36	50
10	21.16	29.26
20	20.60	28.30
30	19.94	27.15
40	19.19	25.83
50	18.35	24.35
60	17.43	22.72
70	16.43	20.94
80	15.36	19.01
90	14.20	16.94
100	12.98	14.71
110	11.67	12.34*
120	10.28	10.37*

* From Eq. (9.13c) because $Kl/r > C_c$.

Table 9.8c Allowable Stresses, ksi, in Steel Building Columns for $Kl/r > 120$

Kl/r	F_a
130	8.84
140	7.62
150	6.64
160	5.83
170	5.17
180	4.61
190	4.14
200	3.73

When Kl/r is less than C_c,

$$F_a = \frac{[1 - (Kl/r)^2/2C_c^2]F_y}{\text{F.S.}} \tag{9.13b}$$

where F.S. = safety factor $= 5/3 + 3(Kl/r)/8C_c - (Kl/r^3)/8C_c^3$
(See Table 9.8*b*).

When Kl/r exceeds C_c,

$$F_a = \frac{12\pi^2 E}{23(Kl/r)^2} = \frac{150{,}000}{(Kl/r)^2} \tag{9.13c}$$

(See Table 9.8c.)

The effective-length factor K, equal to the ratio of effective-column length to actual unbraced length, may be greater or less than 1.0. Theoretical K values for six idealized conditions, in which joint rotation and translation are either fully realized or nonexistent, are tabulated in Fig. 9.5.

An alternative and more precise method of calculating K for an unbraced column uses a nomograph given in the "Commentary" on the AISC Specification for ASD. This method requires calculation of "end-restraint factors" for the top and bottom of the column, to permit K to be determined from the chart.

9.11.2 ASD for Bridge Columns

In the AASHTO bridge-design Specifications, allowable stresses in concentrically loaded columns are determined from Eq. (9.14*a*) or (9.14*b*). When Kl/r is less than C_c,

$$F_a = \frac{F_y}{2.12}\left[1 - \frac{(Kl/r)^2}{2C_c^2}\right] \tag{9.14a}$$

When Kl/r is equal to or greater than C_c,

$$F_a = \frac{\pi^2 E}{2.12(Kl/r^2)} = \frac{135{,}000}{(Kl/r)^2} \tag{9.14b}$$

See Table 9.9.

9.11.3 LRFD for Building Columns

For axially loaded members with $b/t < \lambda_r$ given in Table 9.5, the maximum load P_u, ksi, may be computed from

$$P_u = 0.85 A_g F_y \tag{9.15}$$

where A_g = gross cross-sectional area of the member

$$F_{cr} = (0.658^{\lambda_c^2})F_y \quad \text{for } \lambda \leq 1.5$$

$$F_{cr} = \left[\frac{0.877}{\lambda_c^2}\right]F_y \quad \text{for } \lambda > 1.5$$

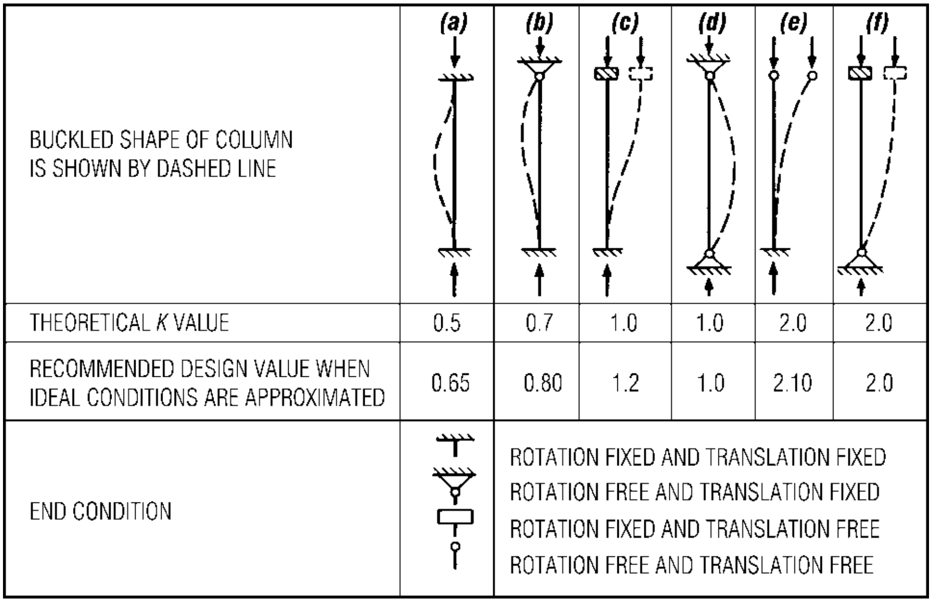

Fig. 9.5 Values of effective-length factor K for columns.

$$\lambda = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}}$$

The AISC Specification for LRFD also presents formulas for designing members with slender elements.

9.11.4 LFD for Bridge Columns

Compression members designed by load-factor design should have a maximum strength, kips,

$$P_u = 0.85 A_s F_{cr} \tag{9.16}$$

Table 9.9 Column Formulas for Bridge Design

Yield Strength, ksi	C_c	Allowable Stress, ksi $Kl/r < C_c$	Allowable Stress, ksi $Kl/r \geq C_c$
36	126.1	$16.98 - 0.00053\,(Kl/r)^2$	
50	107.0	$23.58 - 0.00103\,(Kl/r)^2$	
90	79.8	$42.45 - 0.00333\,(Kl/r)^2$	$135{,}000/(Kl/r)^2$
100	75.7	$47.17 - 0.00412\,(Kl/r)^2$	

where A_s = gross effective area of column cross section, in^2.

For $KL_c/r \leq \sqrt{2\pi^2 E/F_y}$,

$$F_{cr} = F_y\left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{KL_c}{r}\right)^2\right] \tag{9.17a}$$

For $KL_c/r > \sqrt{2\pi^2 E/F_y}$,

$$F_{cr} = \frac{\pi^2 E}{(KL_c/r)^2} = \frac{286{,}220}{(KL_c/r)^2} \tag{9.17b}$$

where F_{cr} = buckling stress, ksi

F_y = yield strength of the steel, ksi

K = effective-length factor in plane of buckling

L_c = length of member between supports, in

r = radius of gyration in plane of buckling, in

E = modulus of elasticity of the steel, ksi

Equations (9.17*a*) and (9.17*b*) can be simplified by introducing a Q factor:

$$Q = \left(\frac{KL_c}{r}\right)^2 \frac{F_y}{2\pi^2 E} \tag{9.18}$$

Then, Eqs. (9.17*a*) and (9.17*b*) can be rewritten as follows:
For $Q < 1.0$:

$$F_{cr} = \left(1 - \frac{Q}{2}\right) F_y \tag{9.19a}$$

For $Q > 1.0$:

$$F_{cr} = \frac{F_y}{2Q} \tag{9.19b}$$

9.12 Allowable Stresses and Loads in Bending

In allowable-stress design (ASD), bending stresses may be computed by elastic theory. The allowable stress in the compression flange usually governs the load-carrying capacity of steel beams and girders.

(T. V. Galambos, "Guide to Design Criteria for Metal Compression Members," 5th ed., John Wiley & Sons, Inc., New York.)

9.12.1 ASD for Building Beams

The maximum fiber stress in bending for laterally supported beams and girders is $F_b = 0.66F_y$ if they are compact (Art. 9.8), except for hybrid girders and members with yield points exceeding 65 ksi. $F_b = 0.60F_y$ for noncompact sections. F_y is the minimum specified yield strength of the steel, ksi. Table 9.10 lists values of F_b for two grades of steel.

Because continuous steel beams have considerable reserve strength beyond the yield point, a redistribution of moments may be assumed when compact sections are continuous over supports or rigidly framed to columns. In that case, negative gravity-load moments over the supports may be reduced 10%. If this is done, the maximum positive moment in each span should be increased by 10% of the average negative moments at the span ends.

The allowable extreme-fiber stress of $0.60F_y$ applies to laterally supported, unsymmetrical members, except channels, and to noncompact-box sections. Compression on outer surfaces of channels bent about their major axis should not exceed $0.60F_y$ or the value given by Eq. (9.22).

The allowable stress of $0.66F_y$ for compact members should be reduced to $0.60F_y$ when the compression flange is unsupported for a length, in, exceeding the smaller of

$$l_{\max} = \frac{76.0 b_f}{\sqrt{F_y}} \tag{9.20a}$$

$$l_{\max} = \frac{20{,}000}{F_y d / A_f} \tag{9.20b}$$

where b_f = width of compression flange, in
d = beam depth, in
A_f = area of compression flange, in^2

The allowable stress should be reduced even more when l/r_T exceeds certain limits, where l is the unbraced length, in, of the compression flange and r_T is the radius of gyration, in, of a portion of the beam consisting of the compression flange and one-third of the part of the web in compression.

For $\sqrt{102{,}000 C_b / F_y} \le l/r_T \le \sqrt{510{,}000 C_b / F_y}$, use

$$F_b = \left[\frac{2}{3} - \frac{F_y (l/r_T)^2}{1{,}530{,}000 C_b}\right] F_y \tag{9.21a}$$

For $l/r_T > \sqrt{510{,}000 C_b / F_y}$, use

$$F_b = \frac{170{,}000 C_b}{(l/r_T)^2} \tag{9.21b}$$

where C_b = modifier for moment gradient [Eq. (9.23)].

When, however, the compression flange is solid and nearly rectangular in cross section and its area is not less than that of the tension flange, the allowable stress may be taken as

$$F_b = \frac{12{,}000 C_b}{l d / A_f} \tag{9.22}$$

Table 9.10 Allowable Bending Stresses in Braced Beams for Buildings, ksi

Yield Strength, ksi	Compact ($0.66F_y$)	Noncompact ($0.60F_y$)
36	24	22
50	33	30

When Eq. (9.22) applies (except for channels), F_b should be taken as the larger of the values computed from Eqs. (9.22) and (9.21*a*) or (9.21*b*) but not more than $0.60F_y$.

The moment-gradient factor C_b in Eqs. (9.20) to (9.22) may be computed from

$$C_b = 1.75 + 1.05\frac{M_1}{M_2} + 0.3\left(\frac{M_1}{M_2}\right)^2 \leq 2.3 \qquad (9.23)$$

where M_1 = smaller beam end moment

M_2 = larger beam end moment

The algebraic sign of M_1/M_2 is positive for double-curvature bending and negative for single-curvature bending. When the bending moment at any point within an unbraced length is larger than that at both ends, the value of C_b should be taken as unity. For braced frames, C_b should be taken as unity for computation of F_{bx} and F_{by} with Eq. (9.65).

Equations (9.21*a*) and (9.21*b*) can be simplified by introduction of a new term:

$$Q = \frac{(l/r_T)^2 F_y}{510{,}000C_b} \qquad (9.24)$$

Now, for $0.2 \leq Q \leq 1$,

$$F_b = \frac{(2 - Q)F_y}{3} \qquad (9.25)$$

For $Q > 1$,

$$F_b = \frac{F_y}{3Q} \qquad (9.26)$$

As for the preceding equations, when Eq. (9.22) applies (except for channels), F_b should be taken as the largest of the values given by Eqs. (9.22) and (9.25) or (9.26), but not more than $0.60F_y$.

9.12.2 ASD for Bridge Beams

AASHTO (Art. 9.6) gives the allowable unit (tensile) stress in bending as $F_b = 0.55F_y$ (Table 9.11). The same stress is permitted for compression when the compression flange is supported laterally for its full length by embedment in concrete or by other means.

When the compression flange is partly supported or unsupported in a bridge, the allowable bending stress, ksi, is

$$F_b = (5 \times 10^7 C_b/S_{xc})(I_{yc}/L) \times \sqrt{0.772J/I_{yc} + 9.87(d/L)^2} \leq 0.55F_y \qquad (9.27)$$

where L = length, in, of unsupported flange between connections of lateral supports, including knee braces

S_{xc} = section modulus, in^3, with respect to the compression flange

I_{yc} = moment of inertia, in^4, of the compression flange about the vertical axis in the plane of the web

$J = \tfrac{1}{3}(b_c t_c^3 + b_t t_t^3 + D t_w^3)$

b_c = width, in, of compression flange

b_t = width, in, of tension flange

t_c = thickness, in, of compression flange

t_t = thickness, in, of tension flange

t_w = thickness, in, of web

D = depth, in, of web

d = depth, in, of flexural member

In general, the moment-gradient factor C_b may be computed from Eq. (9.23). It should be taken as unity, however, for unbraced cantilevers and members in which the moment within a significant portion of the unbraced length is equal to or greater than the larger of the segment end moments. If cover plates are used, the allowable static stress at the point of cutoff should be computed from Eq. (9.27).

The allowable compressive stress for bridge beams may be roughly estimated from the expressions given in Table 9.12, which are based on a formula used prior to 1992.

Table 9.11 Allowable Bending Stress in Braced Bridge Beams, ksi

F_y	F_b
36	20
50	27

Table 9.12 Allowable Compressive Stress in Flanges of Bridge Beams, ksi

F_y	Max l/b	F_b
36	36	$20 - 0.0075\,(l/b)^2$
50	30	$27 - 0.0144\,(l/b)^2$

9.12.3 LRFD for Building Beams

The AISC Specification for LRFD (Art. 9.6) permits use of elastic analysis as described previously for ASD. Thus, negative moments produced by gravity loading may be reduced 10% for compact beams, if the positive moments are increased by 10% of the average negative moments. The reduction is not permitted for hybrid beams, members of A514 steel, or moments produced by loading on cantilevers.

For more accurate plastic design of multistory frames, plastic hinges are assumed to form at points of maximum bending moment. Girders are designed as three-hinged mechanisms. The columns are designed for girder plastic moments distributed to the attached columns plus the moments due to girder shears at the column faces. Additional consideration should be given to moment-end rotation characteristics of the column above and the column below each joint.

For a compact section bent about the major axis, however, the unbraced length L_b of the compression flange where plastic hinges may form at failure may not exceed L_{pd} given by Eqs. (9.28) and (9.29). For beams bent about the minor axis and square and circular beams, L_b is not restricted for plastic analysis.

For I-shaped beams, symmetric about both the major and the minor axis or symmetric about the minor axis but with the compression flange larger than the tension flange, including hybrid girders, loaded in the plane of the web,

$$L_{pd} = \frac{3480 + 2200(M_1/M_2)}{F_{yc}} r_y \qquad (9.28)$$

where F_{yc} = minimum yield stress of compression flange, ksi

M_1 = smaller of the moments, in-kips, at the ends of the unbraced length of beam

M_2 = larger of the moments in-kips, at the ends of the unbraced length of beam

r_y = radius of gyration, in, about minor axis

The plastic moment M_p equals F_yZ for homogenous sections, where Z = plastic modulus, in^3 (Art. 6.65), and for hybrid girders, it may be computed from the fully plastic distribution. M_1/M_2 is positive for beams with reverse curvature.

For solid rectangular bars and symmetric box beams,

$$L_{pd} = \frac{4930 + 2900(M_1/M_2)}{F_y} r_y \geq 2900 \frac{r_y}{F_y} \qquad (9.29)$$

The **flexural design strength** is limited to $0.90M_p$ or $0.90M_n$, whichever is less. M_n is determined by the limit state of lateral-torsional buckling and should be calculated for the region of the last hinge to form and for regions not adjacent to a plastic hinge. The Specification gives formulas for M_n that depend on the geometry of the section and the bracing provided for the compression flange.

For compact sections bent about the major axis, for example, M_n depends on the following unbraced lengths:

L_b = the distance, in, between points braced against lateral displacement of the compression flange or between points braced to prevent twist

L_p = limiting laterally unbraced length, in, for full plastic bending capacity

= $300r_y/\sqrt{F_{yf}}$ for I shapes and channels, $L_b \leq L_r$

= $3750(r_y/M_p)/\sqrt{JA}$ for solid rectangular bars and box beams, $L_p \leq L_r$

F_{yf} = flange yield stress, ksi

J = torsional constant, in^4 (see AISC "Manual of Steel Construction" on LRFD)

A = cross-sectional area, in^2

L_r = limiting laterally unbraced length, in, for inelastic lateral buckling

For doubly symmetric I-shaped beams and channels

$$L_r = \frac{r_y X_1}{F_L}\sqrt{1 + \sqrt{1 + X_2 F_L^2}} \qquad (9.30)$$

where F_L = smaller of $F_{yf} - F_r$ or F_{yw}

F_{yf} = specified minimum yield stress of flange, ksi

F_{yw} = specified minimum yield stress of web, ksi

F_r = compressive residual stress in flange

= 10 ksi for rolled shapes, 16.5 ksi for welded sections

$X_1 = (\pi/S_x)\sqrt{EGJA/2}$

$X_2 = (4C_w/I_y)(S_x/GJ)^2$

E = elastic modulus of the steel

G = shear modulus of elasticity

S_x = section modulus about major axis, in^3 (with respect to the compression flange if that flange is larger than the tension flange)

C_w = warping constant, in^6 (see AISC Manual—LRFD)

I_y = moment of inertia about minor axis, in^4

For the aforementioned shapes, the limiting buckling moment M_r, ksi, may be computed from,

$$M_r = F_L S_x \tag{9.31}$$

For doubly symmetric shapes and channels with $L_b \leq L_r$, bent about the major axis

$$M_n = C_b\left[M_p - (M_p - M_r)\frac{L_b - L_p}{L_r - L_p}\right] \leq M_p \tag{9.32}$$

where $C_b = \dfrac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C}$

$M_{\max}$ = absolute value of maximum moment in the unbraced segment, kip-in

M_A = absolute value of moment at quarter point of the unbraced segment, kip-in

M_B = absolute value of moment at centerline of the unbraced segment, kip-in

M_C = absolute value of moment at three-quarter point of the unbraced segment, kip-in

Also, C_b is permitted to be conservatively taken as 1.0 for all cases.

(See T. V. Galambos, "Guide to Stability Design Criteria for Metal Structures," 5th ed., John Wiley & Sons, Inc., New York, for use of larger values of C_b.)

For solid rectangular bars and box section bent about the major axis,

$$L_r = 58{,}000\left(\frac{r_y}{M_r}\right)\sqrt{JA} \tag{9.33}$$

and the limiting buckling moment is given by

$$M_r = F_y S_x \tag{9.34}$$

For doubly symmetric shapes and channels with $L_b > L_r$, bent about the major axis,

$$M_n = M_{cr} \leq C_b M_r \tag{9.35}$$

where M_{cr} = critical elastic moment, kip-in.

For shapes to which Eq. (9.30) applies,

$$M_{cr} = C_b\frac{\pi}{L_b}\sqrt{EI_yGJ + I_yC_w\left(\frac{\pi E}{L_b}\right)^2} \tag{9.36a}$$

For solid rectangular bars and symmetric box sections,

$$M_{cr} = \frac{57{,}000C_b\sqrt{JA}}{L_b/r_y} \tag{9.36b}$$

For determination of the flexural strength of noncompact plate girders and other shapes not covered by the preceding requirements, see the AISC Manual on LRFD.

9.12.4 LFD for Bridge Beams

For load-factor design of symmetrical beams, there are three general types of members to consider: compact, braced noncompact, and unbraced sections. The maximum strength of each (moment, in-kips) depends on member dimensions and unbraced length as well as on applied shear and axial load (Table 9.13).

The maximum strengths given by the formulas in Table 9.13 apply only when the maximum axial stress does not exceed $0.15F_yA$, where A is the area of the member. Symbols used in Table 9.13 are defined as follows:

D_c = depth of web in compression

F_y = steel yield strength, ksi

Z = plastic section modulus, in^3 (See Art. 6.65.)

S = section modulus, in^3

b' = width of projection of flange, in

d = depth of section, in

h = unsupported distance between flanges, in

M_1 = smaller moment, in-kips, at ends of unbraced length of member

$M_u = F_yZ$

M_1/M_u is positive for single-curvature bending.

Table 9.13 Design Criteria for Symmetrical Flexural Sections for Load-Factor Design of Bridges

Type of Section	Maximum Bending Strength M_u, in-kips	Flange Minimum Thickness t_f, in**	Web Minimum Thickness t_w, in**	Maximum Unbraced Length L_b, in
Compact*	$F_y Z$	$(b'\sqrt{F_y})/65.0$	$(d\sqrt{F_y})/608$	$([3600 - 2200(M_1/M_u)]r_y)/F_y$
Braced noncompact*	$F_y S$	$(b'\sqrt{F_y})/69.6$	$(D_c\sqrt{F_y})/487$	$(20{,}000A_f)/(F_y d)$
Unbraced	See AASHTO Specification			

* Straight-line interpolation between compact and braced noncompact moments may be used for intermediate criteria, except that $t_w \leq d\sqrt{F_y}/608$ should be maintained.

** For compact sections, when both b'/t_f and d/t_w exceed 75% of the limits for these ratios, the following interaction equation applies:

$$\frac{d}{t_w} + 9.35\frac{b'}{t_f} \leq \frac{1064}{\sqrt{F_{yf}}}$$

where F_{yf} is the yield strength of the flange, ksi; t_w is the web thickness, in; and t_f = flange thickness, in.

9.13 Plate Girders

Flexural members built up of plates that form horizontal flanges at top and bottom and joined to vertical or near vertical webs are called plate girders. They differ from beams primarily in that their web depth-to-thickness ratio is larger, for example, exceeds $760/\sqrt{F_b}$ in buildings, where F_b is the allowable bending stress, ksi, in the compression flange.

The webs generally are braced by perpendicular plates called stiffeners, to control local buckling or withstand excessive web shear. Plate girders are most often used to carry heavy loads or for long spans for which rolled shapes are not economical.

9.13.1 Allowable-Stress Design

In computation of stresses in plate girders, the moment of inertia I, in^4, of the gross cross section generally is used. Bending stress f_b due to bending moment M is computed from $f_b = Mc/I$, where c is the distance, in, from the neutral axis to the extreme fiber. For determination of stresses in bolted or riveted girders for bridges, no deduction need be made for rivet or bolt holes unless the reduction in flange area, calculated as indicated in Art. 9.9, exceeds 15%; then the excess should be deducted. For girders for buildings, no deduction need be made provided that

$$0.5F_u A_{fn} \geq 0.6F_y A_{fg} \tag{9.37a}$$

where F_y is the yield stress, ksi; F_u is the tensile strength, ksi; A_{fg} is the gross flange area, in^2; and A_{fn} is the net flange area, in^2, calculated as indicated in Art. 9.9. If this condition is not met, member flexural properties must be based on an effective tension flange area, A_{fe}, given by

$$A_{fe} = \frac{5F_u A_{fn}}{6F_y} \tag{9.37b}$$

In welded-plate girders, each flange should consist of a single plate. It may, however, comprise a series of shorter plates of different thickness joined end to end by full-penetration groove welds. Flange thickness may be increased or decreased at a slope of not more than 1 in 2.5 at transition points. In bridges, the ratio of compression-flange width to thickness should not exceed 24 or $103/\sqrt{f_b}$, where f_b = computed maximum bending stress, ksi.

The web depth-to-thickness ratio is defined as h/t, where h is the clear distance between flanges, in, and t is the web thickness, in. Several design rules for plate girders depend on this ratio.

9.13.2 Load-and-Resistance-Factor Design

The AISC and AASHTO specifications (Art. 9.6) provide rules for LRFD for plate girders. These are not given in the following.

9.13.3 Plate Girders in Buildings

For greatest resistance to bending, as much of a plate girder cross section as practicable should be concentrated in the flanges, at the greatest distance from the neutral axis. This might require, however, a web so thin that the girder would fail by web buckling before it reached its bending capacity. To preclude this, the AISC Specification (Art. 9.6) limits h/t. (See also Art. 9.8).

For an unstiffened web, this ratio should not exceed

$$\frac{h}{t} = \frac{14{,}000}{\sqrt{F_y(F_y + 16.5)}} \tag{9.38}$$

where F_y = yield strength of compression flange, ksi.

Larger values of h/t may be used, however, if the web is stiffened at appropriate intervals.

For this purpose, vertical plates may be welded to it. These transverse stiffeners are not required, though, when h/t is less than the value computed from Eq. (9.38) or given in Table 9.14.

With transverse stiffeners spaced not more than 1.5 times the girder depth apart, the web clear-depth-to-thickness ratio may be as large as

$$\frac{h}{t} = \frac{2000}{\sqrt{F_y}} \tag{9.39}$$

(See Table 9.14.) If, however, the web depth-to-thickness ratio h/t exceeds $760/\sqrt{F_b}$ where F_b, ksi, is the allowable bending stress in the compression flange that would ordinarily apply, this stress should be reduced to F'_b, given by Eqs. (9.40) and (9.41).

$$F'_b = R_{PG} R_e F_b \tag{9.40}$$

$$R_{PG} = \left[1 - 0.0005\frac{A_w}{A_f}\left(\frac{h}{t} - \frac{760}{\sqrt{F_b}}\right)\right] \leq 1.0 \tag{9.41a}$$

$$R_e = \left[\frac{12 + (A_w/A_f)(3\alpha - \alpha^3)}{12 + 2(A_w/A_f)}\right] \leq 1.0 \tag{9.41b}$$

where A_w = web area, in^2

A_f = area of compression flange, in^2

$\alpha = 0.6F_{yw}/F_b \leq 1.0$

F_{yw} = minimum specified yield stress, ksi, of web steel

In a hybrid girder, where the flange steel has a higher yield strength than the web, Eq. (9.41*b*) protects against excessive yielding of the lower-strength web in the vicinity of the higher-strength flanges. For nonhybrid girders, $R_e = 1.0$.

Table 9.14 Critical h/t for Plate Girders in Buildings

F_y, ksi	$14{,}000/\sqrt{F_y(F_y+16.5)}$	$2000/\sqrt{F_y}$
36	322	333
50	243	283

Stiffeners on Building Girders ▪ The shear and allowable shear stress may determine required web area and stiffener spacing. Equations (9.5) and (9.6) give the allowable web shear F_v, ksi, for any panel of a building girder between transverse stiffeners.

The average shear stress f_v, ksi, in a panel of a plate girder (web between successive stiffeners) is defined as the largest shear, kips, in the panel divided by the web cross-sectional area, in^2. As f_v approaches F_v given by Eq. (9.6), combined shear and tension become important. In that case, the tensile stress in the web due to bending in its plane should not exceed $0.6F_y$ or $(0.825 - 0.375f_v/F_v)F_y$, where F_v is given by Eq. (9.6).

The spacing between stiffeners at end panels, at panels containing large holes, and at panels adjacent to panels containing large holes, should be such that f_v does not exceed the value given by Eq. (9.5).

Intermediate stiffeners, when required, should be spaced so that a/h is less than 3 and less than $[260/(h/t)]^2$, where a is the clear distance, in, between stiffeners. Such stiffeners are not required when h/t is less than 260 and f_v is less than F_v computed from Eq. (9.5).

An infinite combination of web thicknesses and stiffener spacings is possible with a particular girder. Figure 9.6, developed for A36 steel, facilitates the trial-and-error process of selecting a suitable combination. Similar charts can be developed for other steels.

The required area of intermediate stiffeners is determined by

$$A_{st} = \frac{1 - C_v}{2}\left[\frac{a}{h} - \frac{(a/h)^2}{\sqrt{1 + (a/h)^2}}\right] YDht \tag{9.42}$$

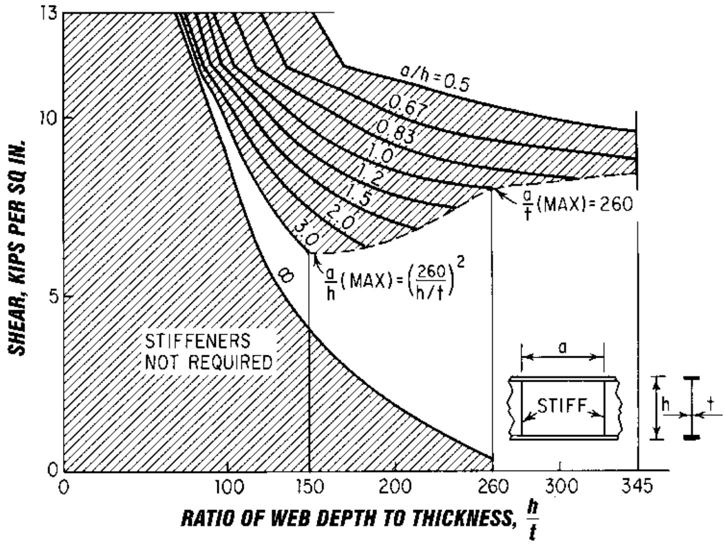

Fig. 9.6 Chart for determining spacing of girder stiffeners of A36 steel.

where A_{st} = gross stiffener area, in^2 (total area, if in pairs)

Y = ratio of yield point of web steel to yield point of stiffener steel

D = 1.0 for stiffeners in pairs

= 1.8 for single-angle stiffeners

= 2.4 for single-plate stiffeners

If the computed web-shear stress f_v is less than F_v computed from Eq. (9.6), A_{st} may be reduced by the ratio f_v/F_v.

The moment of inertia of a stiffener or pair of stiffeners should be at least $(h/50)^4$.

The stiffener-to-web connection should be designed for a shear, kips/lin in of single stiffener, or pair of stiffeners, of at least

$$f_{vs} = h\sqrt{\left(\frac{F_y}{340}\right)^3} \qquad (9.43)$$

This shear may also be reduced by the ratio f_v/F_v.

Spacing of fasteners connecting stiffeners to the girder web should not exceed 12 in c to c. If intermittent fillet welds are used, the clear distance between welds should not exceed 10 in or 16 times the web thickness.

Bearing stiffeners are required on webs where ends of plate girders do not frame into columns or other girders. They may also be needed under concentrated loads and at reaction points. Bearing stiffeners should be designed as columns, assisted by a strip of web. The width of this strip may be taken as $25t$ at interior stiffeners and $12t$ at the end of the web. Effective length for l/r (slenderness ratio) should be 0.75 of the stiffener length. See Art. 9.18 for prevention of web crippling.

Butt-welded splices should be complete-penetration groove welds and should develop the full strength of the smaller spliced section. Other types of splices in cross sections of plate girders should develop the strength required by the stresses at the point of splice but not less than 50% of the effective strength of the material spliced.

Flange connections may be made with rivets, high-strength bolts, or welds connecting flange to web, or cover plate to flange. They should be proportioned to resist the total horizontal shear from bending. The longitudinal spacing of the fasteners, in, may be determined from

$$P = \frac{R}{q} \qquad (9.44)$$

where R = allowable force, kips, on rivets, bolts, or welds that serve length p

q = horizontal shear, kips/in

For a rivet or bolt, $R = A_v F_v$, where A_v is the cross-sectional area, in^2, of the fastener and F_v

the allowable shear stress, ksi. For a weld, R is the product of the length of weld, in, and allowable unit force, kips/in. Horizontal shear may be computed from

$$q = \frac{VQ}{I} \tag{9.45a}$$

where $V =$ shear, kips, at point where pitch is to be determined

$I =$ moment of inertia of section, in^4

$Q =$ static moment about neutral axis of flange cross-sectional area between outermost surface and surface at which horizontal shear is being computed, in^3

Approximately,

$$q = \frac{V}{d}\frac{A}{A_f + A_w/6} \tag{9.45b}$$

where $d =$ depth of web, in, for welds between flange and web; distance between centers of gravity of tension and compression flanges, in, for bolts between flange and web; distance back to back of angles, in, for bolts between cover plates and angles

$A =$ area of flange, in^2, for welds, rivets, and bolts between flange and web; area of cover plates only, in^2, for bolts and rivets between cover plates and angles

$A_f =$ flange area, in^2

$A_w =$ web area, in^2

If the girder supports a uniformly distributed load w, kips/in, on the top flange, the pitch should be determined from

$$p = \frac{R}{\sqrt{q^2 + w^2}} \tag{9.46}$$

(See also Art. 9.16.)

Maximum longitudinal spacing permitted in the compression-flange cover plates is 12 in or the thickness of the thinnest plate times $127\sqrt{F_y}$ when fasteners are provided on all gage lines at each section or when intermittent welds are provided along the edges of the components. When rivets or bolts are staggered, the maximum spacing on each gage line should not exceed 18 in or the thickness of the thinnest plate times $190\sqrt{F_y}$. Maximum spacing in tension-flange cover plates is 12 in or 24 times the thickness of the thinnest plate. Maximum spacing for connectors between flange angles and web is 24 in.

9.13.4 Girders in Bridges

For highway bridges, Table 9.15 gives critical web thicknesses t, in, for two grades of steel as a fraction of h, the clear distance, in, between flanges. When t is larger than the value in column 1, intermediate transverse (vertical) stiffeners are not required. If shear stress is less than the allowable, the web may be thinner. Thus, stiffeners may be omitted if $t \geq h\sqrt{f_n}/271$, where $f_v =$ average unit shear, ksi (vertical shear at section, kips, divided by web cross-sectional area). But t should not be less than $h/150$.

When t lies between the values in columns 1 and 2, transverse intermediate stiffeners are required. Webs thinner than the values in column 2 are permissible if they are reinforced by a longitudinal (horizontal) stiffener. If the computed maximum compressive bending stress f_b, ksi, at a section is less than the allowable bending stress, a longitudinal stiffener is not required if $t \geq h\sqrt{f_b}/727$; but t should not be less than $h/170$. When used, a plate longitudinal stiffener should be attached to the web at a distance $h/5$ below the inner surface of the compression flange. [See also Eq. (9.49).]

Webs thinner than the values in column 3 are not permitted, even with transverse stiffeners and one longitudinal stiffener, unless the computed

Table 9.15 Minimum Web Thickness, in, for Highway-Bridge Plate Girders*

Yield, Strength, ksi	Without Intermediate Stiffeners (1)	Transverse Stiffeners, No Longitudinal Stiffeners (2)	Longitudinal Stiffener, Transverse Stiffeners (3)
36	$h/78$	$h/165$	$h/327$
50	$h/66$	$h/140$	$h/278$

* "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.

compressive bending stress is less than the allowable. When it is, t may be reduced in accord with AASHTO formulas, but it should not be less than $h/340$.

Stiffeners on Bridge Girders ▪ The shear and allowable shear stress may determine required web area and stiffener spacing. Equation (9.8) gives the allowable web shear F_v, ksi, for panels between intermediate transverse stiffeners. Maximum spacing a, in, for such panel is $3h$ but not more than $67{,}600h(h/t_w)^2$. The first intermediate stiffener from a simple support should be located not more than $1.5h$ from the support and the shear in the end panel should not exceed F_v given by Eq. (9.8) nor $F_y/3$.

Intermediate stiffeners may be a single angle fastened to the web or a single plate welded to the web. But preferably they should be attached in pairs, one on each side of the web. Stiffeners on only one side of the web should be attached to the outstanding leg of the compression flange. At points of concentrated loading, stiffeners should be placed on both sides of the web and designed as bearing stiffeners.

The minimum moment of inertia, in^4, of a transverse stiffener should be at least

$$I = a_o t^3 J \tag{9.47}$$

where $J = 2.5h^2/a_o^2 - 2 \geq 0.5$

h = clear distance between flanges, in

a_o = actual stiffener spacing, in

t = web thickness, in

For paired stiffeners, the moment of inertia should be taken about the centerline of the web; for single stiffeners, about the face in contact with the web.

The gross cross-sectional area of intermediate stiffeners should be at least

$$A = \left[0.15BDt_w(1 - C)\frac{V}{V_u} - 18t_w^2\right]Y \tag{9.48}$$

where Y is the ratio of web-plate yield strength to stiffener-plate yield strength: $B = 1.0$ for stiffener pairs, 1.8 for single angles, and 2.4 for single plates; and C is defined in Eq. (9.8). V_u should be computed from Eq. (9.12*a*) or (9.12*b*).

The width of an intermediate transverse stiffener, plate or outstanding leg of an angle, should be at least 2 in plus $\frac{1}{30}$ of the depth of the girder and preferably not less than one-fourth the width of the flange. Minimum thickness is $\frac{1}{16}$ of the width.

Transverse intermediate stiffeners should have a tight fit against the compression flange but need not be in bearing with the tension flange. The distance between the end of the stiffener weld and the near edge of the web-to-flange fillet weld should not be less than $4t$ or more than $6t$. However, if bracing or diaphragms are connected to an intermediate stiffener, care should be taken in design to avoid web flexing, which can cause premature fatigue failures.

Bearing stiffeners are required at all concentrated loads, including supports. Such stiffeners should be attached to the web in pairs, one on each side, and they should extend as nearly as practicable to the outer edges of the flanges. If angles are used, they should be proportioned for bearing on the outstanding legs of the flange angles or plates. (No allowance should be made for the portion of the legs fitted to the fillets of flange angles.) The stiffener angles should not be crimped.

Bearing stiffeners should be designed as columns. The allowable unit stress is given in Table 9.9, with $L = h$. For plate stiffeners, the column section should be assumed to consist of the plates and a strip of web. The width of the strip may be taken as 18 times the web thickness t for a pair of plates. For stiffeners consisting of four or more plates, the strip may be taken as the portion of the web enclosed by the plates plus a width of not more than $18t$. Minimum bearing stiffener thickness is $(b'/12)\sqrt{F_y/33}$, where b' = stiffener width, in.

Bearing stiffeners must be ground to fit against the flange through which they receive their load or attached to the flange with full-penetration groove welds. But welding transversely across the tension flanges should be avoided to prevent creation of a severe fatigue condition.

Termination of Top Flange ▪ Upper corners of through-plate girders, where exposed, should be rounded to a radius consistent with the size of the flange plates and angles and the vertical height of the girder above the roadway. The first flange plate, or a plate of the same width, should be bent around the curve and continued to the bottom of the girder. In a bridge consisting of two or more spans, only the corners at the extreme ends of the bridge need to be rounded, unless the spans have girders of different heights. In such a case, the higher girders should have the top flanges curved down at the ends to meet the top corners of the girders in adjacent spans.

Seating at Supports ▪ Sole plates should be at least ¾ in thick. Ends of girders on masonry should be supported on pedestals so that the bottom flanges will be at least 6 in above the bridge seat. Elastomeric bearings often are cost-effective.

Longitudinal Stiffeners ▪ These should be placed with the center of gravity of the fasteners $h/5$ from the toe, or inner face, of the compression flange. Moment of inertia, in^4, should be at least

$$I = ht^3\left(2.4\frac{a_o^2}{h^2} - 0.13\right) \tag{9.49}$$

where a_o = actual distance between transverse stiffeners, in

t = web thickness, in

Thickness of stiffener, in, should be at least $b\sqrt{F_y}/96$ where b is the stiffener width, in, and F_y is the yield strength of the compression flange, ksi. The bending stress in the stiffener should not exceed the allowable for the material.

Longitudinal stiffeners usually are placed on one side of the web. They need not be continuous. They may be cut at their intersections with transverse stiffeners.

Splices ▪ These should develop the strength required by the stresses at the splices but not less than 75% of the effective strength of the material spliced. Splices in riveted flanges usually are avoided. In general, not more than one part of a girder should be spliced at the same cross section. Bolted web splices should have plates placed symmetrically on opposite sides of the web. Splice plates for shear should extend the full depth of the girder between flanges. At least two rows of bolts on each side of the joint should fasten the plates to the web.

Rivets, high-strength bolts, or welds connecting flange to web, or cover plate to flange, should be proportioned to resist the total horizontal shear from bending, as described for plate girders in buildings. In riveted bridge girders, legs of angles 6 in or more wide connected to webs should have two lines of rivets. Cover plates over 14 in wide should have four lines of rivets.

Hybrid Bridge Girders ▪ These may have flanges with larger yield strength than the web and may be composite or noncomposite with a concrete slab, or they may utilize an orthotropic-plate deck as the top flange. At any cross section where the bending stress in either flange exceeds 55 percent of the minimum specified yield strength of the web steel, the compression-flange area must not be less than the tension-flange area. The top-flange area includes the transformed area of any portion of the slab or reinforcing steel that acts compositely with the girder.

Computation of bending stresses and allowable stresses is generally the same as for girders with uniform yield strength. The bending stress in the web, however, may exceed the allowable bending stress if the computed flange bending stress does not exceed the allowable stress multiplied by a factor R.

$$R = 1 - \frac{\beta\psi(1-\alpha)^2(3-\psi+\psi\alpha)}{6+\beta\psi(3-\psi)} \tag{9.50}$$

where α = ratio of web yield strength to flange yield strength

ψ = distance from outer edge of tension flange or bottom flange of orthotropic deck to neutral axis divided by depth of steel section

β = ratio of web area to area of tension flange or bottom flange of orthotropic-plate bridge

The rules for shear stresses are as previously described, except that for transversely stiffened girders, the allowable shear stress (throughout the length of the girder) is given by the following instead of Eq. (9.8): $F_v = CF_y/3 \le F_y/3$.

9.14 Deflection Limitations

For buildings, beams and girders supporting plastered ceilings should not deflect under live load more than 1/360 of the span. To control deflection, fully stressed floor beams and girders should have a minimum depth of $F_y/800$ times the span, where F_y is the steel yield strength, ksi. Depth of fully stressed roof purlins should be at least $F_y/1000$ times the span, except for flat roofs, for which ponding conditions should be considered (Art. 9.15).

For bridges, simple-span or continuous girders should be designed so that deflection due to live load plus impact should not exceed $\frac{1}{800}$ of the span.

For bridges located in urban areas and used in part by pedestrians, however, deflection preferably should not exceed $\frac{1}{1000}$ of the span. To control deflections, depth of noncomposite girders should be at least $\frac{1}{25}$ of the span. For composite girders, overall depth, including slab thickness, should be at least $\frac{1}{25}$ of the span, and depth of steel girder alone, at least $\frac{1}{30}$ of the span. For continuous girders, the span for these ratios should be taken as the distance between inflection points.

9.15 Ponding Considerations in Buildings

Flat roofs on which water may accumulate may require analysis to ensure that they are stable under ponding conditions. A flat roof may be considered stable and an analysis need not be made if both Eqs. (9.51) and (9.52) are satisfied.

$$C_p + 0.9C_s \leq 0.25 \tag{9.51}$$

$$I_d \geq 25S^4/10^6 \tag{9.52}$$

where $C_p = 32L_sL_p^4/10^7I_p$

$C_s = 32SL_s^4/10^7I_s$

L_p = length, ft, of primary member or girder

L_s = length, ft, of secondary member or purlin

S = spacing, ft, of secondary members

I_p = moment of inertia of primary member, in^4

I_s = moment of inertia of secondary member, in^4

I_d = moment of inertia of steel deck supported on secondary members, in^4/ft

For trusses and other open-web members, I_s should be decreased 15%. The total bending stress due to dead loads, gravity live loads, and ponding should not exceed $0.80F_y$, where F_y is the minimum specified yield stress for the steel.

9.16 Allowable Bearing Stresses and Loads

Load transfer between steel members and their supports may be designed by the allowable-stress or load-and-resistance-factor method (Art. 9.7). The AISC and AASHTO Specifications provide rules for these methods, but the following covers only ASD.

The Specifications require that provision be made for safe transfer of loads in bearing between steel components and between steel members and supports of different materials. In the latter case, base plates are generally set under columns and bearing plates are placed under beams to transfer loads between the steel members and their supports. When the supports are rigid, such as concrete or masonry, axial loads may be assumed to be uniformly distributed over the bearing areas. It is essential that the load be spread over an area such that the average pressure on the concrete or masonry does not exceed the allowable stress for the material. In the absence of building code or other governing regulations, the allowable bearing stresses in Table 9.16 may be used.

Bearing on Fasteners ▪ See Art. 9.24.

Bearing Plates ▪ To resist a beam reaction, the minimum bearing length N in the direction of the beam span for a bearing plate is determined by equations for prevention of local web yielding and web crippling (Art. 9.18). A larger N is generally desirable but is limited by the available wall thickness.

When the plate covers the full area of a concrete support, the area, in^2, required by the bearing plate is

$$A_1 = \frac{R}{0.35f'_c} \tag{9.53}$$

where R = beam reaction, kips

f'_c = specified compressive strength of the concrete, ksi

Table 9.16 Allowable Bearing Stress, F_p, on Concrete and Masonry, ksi

Full area of concrete support	$0.35f'_c$
Less than full area of concrete support	$0.35f'_c\sqrt{A_2/A_1} \leq 0.70f'_c$
Sandstone and limestone	0.40
Brick in cement mortar	0.25

When the plate covers less than the full area of the concrete support, then, as determined from Table 9.16,

$$A_1 = \left(\frac{R}{0.35f'_c\sqrt{A_2}}\right)^2 \tag{9.54}$$

where A_2 = full cross-sectional area of concrete support, in^2

With N established, usually rounded to full inches, the minimum width of plate B, in, may be calculated by dividing A_1 by N and then rounded off to full inches so that $BN \geq A_1$. Actual bearing pressure f_p, ksi, under the plate then is

$$f_p = \frac{R}{BN} \tag{9.55}$$

The plate thickness usually is determined with the assumption of cantilever bending of the plate.

$$t = \left(\frac{1}{2}B - k\right)\sqrt{\frac{3f_p}{F_b}} \tag{9.56}$$

where t = minimum plate thickness, in

k = distance, in, from beam bottom to top of web fillet (Fig. 9.7)

F_b = allowable bending stress of plate, ksi

Column Base Plates ▪ The area A_1, in^2, required for a base plate under a column supported by concrete should be taken as the larger of the values calculated from Eq. (9.54), with R taken as the total column load, kips, or

$$A_1 = \frac{R}{0.70f'_c} \tag{9.57}$$

Unless the projections of the plate beyond the column are small, the plate may be designed as a cantilever assumed to be fixed at the edges of a rectangle with sides equal to $0.80b$ and $0.95d$, where b is the column flange width, in, and d the column depth, in.

To minimize material requirements, the plate projections should be nearly equal. For this purpose, the plate length N, in (in the direction of d), may be taken as

$$N = \sqrt{A_1} + 0.5(0.95d - 0.80b) \tag{9.58}$$

The width B, in, of the plate then may be calculated by dividing A_1 by N. Both B and N may be selected in full inches so that $BN \geq A_1$. In that case, the bearing pressure f_p, ksi, may be determined from Eq. (9.55). Thickness of plate, determined by cantilever bending, is given by

$$t = 2p\sqrt{\frac{f_p}{F_y}} \tag{9.59}$$

where F_y = minimum specified yield strength, ksi, of plate

p = larger of $0.5(N - 0.95d)$ and $0.5(B - 0.80b)$

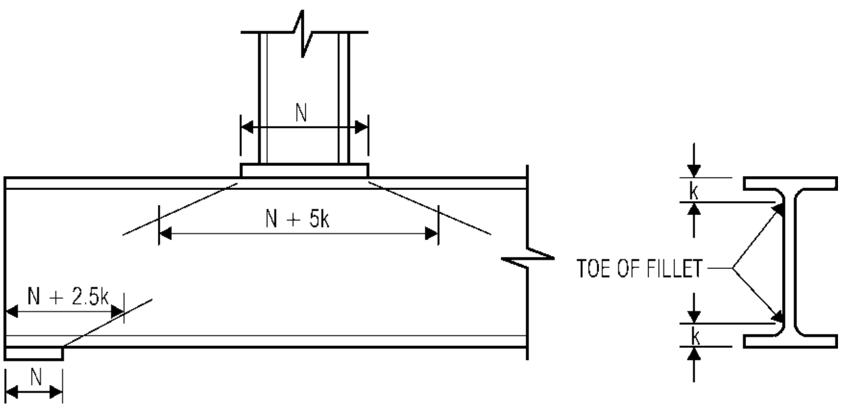

Fig. 9.7 For investigating web yielding, stresses are assumed to be distributed over lengths of web indicated at the bearings, where N is the length of bearing plates and k is the distance from outer surface of beam to the toe of the fillet.

When the plate projections are small, the area A_2 should be taken as the maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area. Thus, for an H-shaped column, the column load may be assumed distributed to the concrete over an H-shaped area with flange thickness L, in, and web thickness $2L$.

$$L = \frac{1}{4}(d+b) - \frac{1}{4}\sqrt{(d+b)^2 - \frac{4R}{F_p}} \tag{9.60}$$

where F_p = allowable bearing pressure, ksi, on support. (If L is an imaginary number, the loaded portion of the supporting surface may be assumed rectangular as discussed above.) Thickness of the base plate should be taken as the larger of the values calculated from Eq. (9.59) and

$$t = L\sqrt{\frac{3f_p}{F_b}} \tag{9.61}$$

Bearing on Milled Surfaces ▪ In building construction, allowable bearing stress for milled surfaces, including bearing stiffeners, and pins in reamed, drilled, or bored holes, is $F_p = 0.90F_y$, where F_y is the yield strength of the steel, ksi.

For expansion rollers and rockers, the allowable bearing stress, kips/lin in, is

$$F_p = \frac{F_y - 13}{20} 0.66d \tag{9.62}$$

where d is the diameter, in, of the roller or rocker. When parts in contact have different yield strengths, F_y is the smaller value.

For highway design, AASHTO limits the allowable bearing stress on milled stiffeners and other steel parts in contact to $F_p = 0.80F_y$. Allowable bearing stresses on pins are in Table 9.17.

The allowable bearing stress for expansion rollers and rockers used in bridges depends on the yield point in tension F_y of the steel in the roller or the base, whichever is smaller. For diameters up to 25 in, the allowable stress, kips/lin in, is

$$p = \frac{F_y - 13}{20} 0.6d \tag{9.63}$$

For diameters from 25 to 125 in,

$$p = \frac{F_y - 13}{20} 3\sqrt{d} \tag{9.64}$$

where d = diameter of roller or rocker, in.

Table 9.17 Allowable Bearing Stresses on Pins, ksi

		Bridges	
F_y	Buildings $F_p = 0.90F_y$	Pins Subject to Rotation $F_p = 0.40F_y$	Pins Not Subject to Rotation $F_p = 0.80F_y$
36	33	14	29
50	45	20	40

9.17 Combined Axial Compression or Tension and Bending

The AISC Specification for allowable stress design for buildings (Art. 9.6) includes three interaction formulas for combined axial compression and bending:

When the ratio of computed axial stress to allowable axial stress f_a/F_a exceeds 0.15, both Eqs. (9.65) and (9.66) must be satisfied.

$$\frac{f_a}{F_a} + \frac{C_{mx}f_{bx}}{(1 - f_a/F'_{ex})F_{bx}} + \frac{C_{my}f_{by}}{(1 - f_a/F'_{ey})F_{by}} \le 1 \tag{9.65}$$

$$\frac{f_a}{0.60F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{f_{by}} \le 1 \tag{9.66}$$

When $f_a/F_a \le 0.15$, Eq. (9.67) may be used instead of Eqs. (9.65) and (9.66).

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \le 1 \tag{9.67}$$

In the preceding equations, subscripts x and y indicate the axis of bending about which the stress occurs, and

F_a = axial stress that would be permitted if axial force alone existed, ksi (see Arts. 9.9 and 9.11)

F_b = compressive bending stress that would be permitted if bending moment alone existed, ksi (see Art. 9.12)

F'_e = $149{,}000/(Kl_b/r_b)^2$, ksi; as for F_a, F_b, and $0.6F_y$, F'_e may be increased one-third for wind and seismic loads where permitted by codes

l_b = actual unbraced length in plane of bending, in

r_b = radius of gyration about bending axis, in

K = effective-length factor in plane of bending

f_a = computed axial stress, ksi

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

C_m = adjustment coefficient

For compression members in frames subject to joint translation (sidesway), $C_m = 0.85$ in Eq. (9.65). For restrained compression members in frames braced against joint translation and not subject to transverse loading between supports in the plane of bending, $C_m = 0.6 - 0.4M_1/M_2$. M_1/M_2 is the ratio of the smaller to larger moment at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1/M_2 is positive when the member is bent in reverse curvature and negative when it is bent in single curvature. For compression members in frames braced against joint translation in the plane of loading and subject to transverse loading between supports, the value of C_m may be determined by rational analysis. But in lieu of such analysis, the following values may be used: For members whose ends are restrained, $C_m = 0.85$. For members whose ends are unrestrained, $C_m = 1.0$.

Building members subject to combined axial tension and bending should satisfy Eq. (9.67), with f_b and F_b, respectively, as the computed and permitted bending tensile stress. But the computed bending compressive stress is limited by Eqs. (9.22) and (9.21*a*) or (9.21*b*).

Combined compression and bending stresses in bridge design are covered by equations similar to Eqs. (9.65) and (9.66) but adjusted to reflect the lower allowable stresses of AASHTO.

9.18 Webs under Concentrated Loads

Yielding or crippling of webs of rolled beams and plate girders at points of application of concentrated loads should be investigated.

Criteria for Buildings ▪ The AISC Specification for allowable-stress design for buildings (Art. 9.6) places a limit on compressive stress in webs to prevent local web yielding. For a rolled beam, bearing stiffeners are required at a concentrated load if the stress f_a, ksi, at the toe of the web fillet exceeds $F_a = 0.66F_{yw}$, where F_{yw} is the minimum specified yield stress of the web steel, ksi. In the calculation of the stressed area, the load may be assumed distributed over the distance indicated in Fig. 9.7.

For a concentrated load applied at a distance larger than the depth of the beam from the end of the beam,

$$f_a = \frac{R}{t_w(N + 5k)} \qquad (9.68)$$

where R = concentrated load of reaction, kips

t_w = web thickness, in

N = length of bearing, in (for end reaction, not less than k)

k = distance, in, from outer face of flange to web toe of fillet (Fig. 9.7)

For a concentrated load applied close to the beam end,

$$f_a = \frac{R}{t_w(N + 2.5k)} \qquad (9.69)$$

To prevent web crippling, the AISC Specification requires that bearing stiffeners be provided on webs where concentrated loads occur when the compressive force exceeds R, kips, computed from the following:

For a concentrated load applied at a distance from the beam end of at least $d/2$, where d is the depth of beam

$$R = 67.5t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{F_{yw}t_f/t_w} \qquad (9.70)$$

where t_f = flange thickness, in.

For a concentrated load applied closer than $d/2$ from the beam end,

$$R = 34t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{F_{yw}t_f/t_w} \qquad (9.71)$$

If stiffeners are provided and extend at least one-half the web, R need not be computed.

Another consideration is prevention of sidesway web buckling. The AISC Specification requires bearing stiffeners when the compressive force from a concentrated load exceeds limits that depend on the relative slenderness of web and flange r_{wf} and whether or not the loaded flange is restrained against rotation.

$$r_{wf} = \frac{d_c/t_w}{l/b_f} \tag{9.72}$$

where l = largest unbraced length, in, along either top or bottom flange at point of application of load

b_f = flange width, in

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

Stiffeners are required if the concentrated load exceeds R, kips, computed from

$$R = \frac{6800t_w^2}{h}(1 + 0.4r_{wf}^3) \tag{9.73a}$$

where h = clear distance, in, between flanges, and r_{wf} is less than 2.3 when the loaded flange is restrained against rotation. If the loaded flange is not restrained and r_{wf} is less than 1.7,

$$R = 0.4r_{wf}^3 \frac{6800t_w^3}{h} \tag{9.73b}$$

R need not be computed for larger values of r_{wf}.

Criteria for Bridges ▪ Rolled beams used as flexural members in bridges should be provided with stiffeners at bearings when the unit shear in the web exceeds $0.25F_{yw}$.

For plate-girder bridges, bearing stiffeners should always be installed over the end bearings and over the intermediate bearings of continuous girders. See Art. 9.13.

9.19 Design of Stiffeners under Loads

AISC requires that fasteners or welds for end connections of beams, girders, and trusses be designed for the combined effect of forces resulting from moment and shear induced by the rigidity of the connection. 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 calculated from Eq. (9.74) must be provided whenever the calculated value of A_{st} is positive.

$$A_{st} = \frac{P_{bf} - F_{yc}t_{wc}(t_b + 5k)}{F_{yst}} \tag{9.74}$$

where F_{yc} = column yield stress, ksi

F_{yst} = stiffener yield stress, ksi

k = distance, in, between outer face of column flange and web toe of its fillet, if column is rolled shape, or equivalent distance if column is welded shape

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

t_{wc} = thickness of column web, in

t_b = thickness of flange or moment-connection plate delivering concentrated force, in

Notwithstanding the above requirements, a stiffener or a pair of stiffeners must be provided opposite the beam compression flange when the column-web depth clear of fillets d_c is greater than

$$d_c = \frac{4100t_{wc}^3\sqrt{F_{yc}}}{P_{bf}} \tag{9.75}$$

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

$$t_f = 0.4\sqrt{\frac{P_{bf}}{F_{yc}}} \tag{9.76}$$

When the preceding conditions for Eqs. (9.74) to (9.76) or for prevention of web yielding, crippling, or sidesway buckling (Art. 9.18) require stiffeners, they should be applied in pairs. Stiffeners required to keep within the limits for Eqs. (9.68) (9.69), (9.74), and (9.76) need not extend more than half the depth of the web.

Each stiffener pair required by Eqs. (9.70), (9.71), and (9.74) should be designed as a column consisting of the stiffener pair and an effective portion of the beam or column web having a width of 25 times the web thickness at interior points and 12 times the web thickness at the column ends. Effective length of the column should be taken as 75% of the clear depth of the web.

Stiffeners required by Eqs. (9.74) to (9.76) should comply with the following additional criteria:

1. The width of each stiffener plus half the thickness of the column web should not be less than one-third the width of the flange or moment-connection plate delivering the concentrated force.
2. The thickness of stiffeners should not be less than $t_b/2$.
3. The weld joining stiffeners to the column web must be sized to carry the force in the stiffener caused by unbalanced moments on opposite sides of the column.

Connections having high shear in the column web should be investigated as described by AISC. Equation (9.7) gives the condition for investigating high shear in the column web within the boundaries of the connection.

Stiffeners opposite the beam compression flange may be fitted to bear on the inside of the column flange. Stiffeners opposite the tension flange should be welded, and the welds must be designed for the forces involved.

See also Art. 9.13 and Art. 9.2 (cracking).

9.20 Design of Beam Sections for Torsion

Torsional stresses may be induced in steel beams either by unsymmetrical loading or by symmetrical loading on unsymmetrical shapes, such as channels or angles. In most applications, they are much smaller than the concurrent axial or bending stresses, but the resultant of the combined stresses should not exceed the allowable stress. In bridge design, torsional effects are important in design of curved girders.

9.21 Wind and Seismic Stresses

In allowable-stress design for buildings, allowable stresses may be increased one-third under wind and seismic forces acting alone or with gravity loads, where permitted by codes. The resulting design, however, should not be less than that required for dead and live loads without the increase in allowable stress. The increased stress is permitted because of the short duration of the load. Its validity has been justified by many years of satisfactory performance.

For allowable stresses, including wind and seismic effects on bridges, see Art. 17.4.

Successful wind or seismic design is dependent on close attention to connection details. It is good practice to provide as much ductility as practical in such connections so that the fasteners are not overstressed.

In load-and-resistance-factor design, load factors are applied to adjust for wind and seismic effects.

9.22 Fatigue Strength of Structural Components

Extensive research programs have been conducted to determine the fatigue strength of structural members and connections. These programs included large-scale beam specimens with various details such as flange-to-web fillet welds, flange cover plates, flange attachments, and web stiffeners. These studies showed that the stress range (algebraic difference between maximum and minimum stress) and the notch severity of the details were the dominant variables. For design purposes, the effects of steel yield point and stress ratio are not considered significant.

Allowable stress ranges based on this research have been adopted by the American Institute of Steel Construction (AISC), the American Association of State Highway and Transportation Officials (AASHTO), and the American Welding Society (AWS) as indicated in Table 9.18. Plain material and various details have been grouped in categories of increasing severity, A through F. The allowable-stress ranges are given for various numbers of cycles, from 20,000 to over 2 million. The over-2 million-cycles life corresponds to the fatigue limit; the detail is considered to have infinite life if the allowable stress range listed for over 2 million cycles is not exceeded. The allowable fatigue-stress range is applicable to any of the structural steels, but the maximum stress cannot exceed the maximum permitted under static loadings.

Under AASHTO allowable-stress design, the allowable stress ranges given in Table 9.18 are for redundant load path structures and more conservative values are given for non-redundant load

Table 9.18 Allowable Range of Stress under Fatigue Loading, ksi*

	Number of Cycles			
Stress. Category†	From 20,000–100,000	From 100,000–500,000	From 500,000–2,000,000	Over 2,000,000
A	63	37	24	24
B	49	29	18	16
B′	39	23	15	12
C	35	21	13	10‡
D	28	16	10	7
E	22	13	8	5
E′	16	9	6	3
F	15	12	9	8

* Based on the requirements of AISC and AWS. Allowable ranges F_{sr} are for tension or reversal, except as noted. Values given represent 95% confidence limits for 95% survival. See AISC and AWS for description of stress categories. AASHTO requirements are similar, except as noted below, for structures with redundant load paths but are more severe for structures with nonredundant load paths.

† Typical details included in each category are as follows (see specifications for complete descriptions): A—base metal of plain material; B—base metal and weld metal at full-penetration groove welds, reinforcement ground off; B′—base metal in full-penetration groove welds in built-up members, backing bars not removed; C—base metal and weld metal at full-penetration groove welds, reinforcement not removed; D—base metal at certain attachment details; E—base metal at end of cover plate; E′—base metal at end of cover plate over 0.8 in thick; F—shear in weld metal of fillet welds.

‡ Flexural stress range of 12 ksi permitted at toe of stiffener welds on webs or flanges.

path structures. LRFD specifications of AISC and AASHTO use a somewhat different approach to fatigue design. Under that method, the stress range is a design consideration only when it exceeds a fatigue threshold stress given for the various details. In such cases, a design stress range for various stress categories is calculated from an equation that includes a fatigue constant for each detail. For plain material away from any welding, non-coated weathering steel is classified as stress Category B (16 ksi stress threshold) because of its rougher surface, rather than Category A (24 ksi stress threshold) for other base metal. However, the prevailing structural detail category, such as groove welds or attachments, usually controls and these are unaffected by the weathered surface.

Note that the AISC, AASHTO, and AWS Specifications do not require fatigue checks in elements of members where calculated stresses are always in compression because although a crack may initiate in a region of tensile residual stress, the crack will generally not propagate beyond.

In design of a structural member to resist fatigue, each detail should be checked for the stress conditions that exist at that location. When a severe detail cannot be avoided, it is often advantageous to locate it in a region where the stress range is low so that the member can withstand the desired number of cycles.

9.23 Load Transfer and Stresses at Welds

Various coated stick electrodes for shielded metal-arc welding and various wire electrodes and flux or gas combinations for other processes may be selected to produce weld metals that provide a wide range of specified minimum-strength levels. AWS Specifications give the electrode classes and welding processes that can be used to obtain matching weld metal, that is, weld metal that has a minimum tensile strength similar to that of various groups of steel. As indicated in Tables 9.19 and 9.20, however, matching weld metal is not always required, particularly in the case of fillet welds.

The differential cooling that accompanies welding causes residual stresses in the weld and the material joined. Although these stresses have an important effect on the strength of compression members, which is included in the design

Table 9.19 Allowable Stresses for Welds in Building Construction

Stresses in Weld*	Allowable Stress	Required Weld Strength Level†
	Complete-Penetration Groove Welds	
Tension normal to effective area	Same as base metal	Matching weld metal must be used
Compression normal to effective area	Same as base metal	Weld metal with strength level equal to or less than matching weld metal may be used.
Tension or compression parallel to axis of weld	Same as base metal	Weld metal with strength level equal to or less than matching weld metal may be used.
Shear on effective area	0.30 nominal tensile strength of weld metal, ksi	Weld metal with strength level equal to or less than matching weld metal may be used.
	Partial-Penetration Groove Welds‡	
Compression normal to effective area	Same as base metal	Weld metal with strength level equal to or less than matching weld metal may be used.
Tension or compression parallel to axis of weld§	Same as base metal	Weld metal with strength level equal to or less than matching weld metal may be used.
Shear parallel to axis of weld	0.30 nominal tensile strength of weld metal, ksi	Weld metal with strength level equal to or less than matching weld metal may be used.
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 strength of base metal	Weld metal with strength level equal to or less than matching weld metal may be used.
	Fillet Welds	
Shear on effective area	0.30 nominal tensile strength of weld metal, ksi	Weld metal with strength level equal to or less than matching weld metal may be used.
Tension or compression parallel to axis of weld§	Same as base metal	Weld metal with strength level equal to or less than matching weld metal may be used.
	Plug and Slot Welds	
Shear parallel to faying surfaces (on effective area)	0.30 nominal tensile strength of weld metal, ksi	Weld metal with strength level equal to or less than matching weld metal may be used.

* For definition of effective area, see AWS, D1.1.
† For matching weld metal, see AWS, D1.1. Weld metal one strength level stronger than matching weld metal may be used.
‡ See the AISC Specification for allowable-stress design for buildings for limitations on use of partial-penetration groove welds.
§ 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 those elements parallel to the axis of the welds.

Table 9.20 Allowable Stresses for Welds in Bridge Construction†

Stresses in Weld*	Allowable Stress
Complete-Penetration Groove Welds	
Tension normal to effective area	Same as base metal
Compression normal to the effective area	Same as base metal
Tension or compression parallel to axis of weld	Same as base metal
Shear on effective area	0.27 nominal tensile strength of weld metal, ksi, but not greater than the tensile strength of the connected part.
Partial-Penetration Groove Welds	
Compression normal to effective area	Same as base metal
Tension or compression parallel to axis of weld‡	Same as base metal
Shear parallel to axis of weld	0.27 nominal tensile strength of weld metal, ksi, except shear stress on base metal shall not exceed 0.36 yield strength of base metal
Fillet Welds	
Shear on effective area	0.27 nominal tensile strength of weld metal, ksi, except shear stress on base metal shall not exceed 0.36 yield strength of base metal
Tension or compression parallel to axis of weld‡	Same as base metal
Plug and Slot Welds	
Shear parallel to faying surfaces (on effective area)	0.27 nominal tensile strength of weld metal, ksi, except shear stress on base metal shall not exceed 0.36 yield strength of base metal

* For definition of effective area, see AWS, D1.5.

† Matching weld metal is usually specified. However, designers may use an electrode classification with strength less than the base metal in the case of quenched and tempered steels. For matching weld metal, see AWS, D1.5

‡ 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 those elements parallel to the axis of the welds.

equations, they do not usually have a significant effect on the strength of welded connections.

In groove welds, the loads are transferred directly across the weld by tensile or compressive stresses. For complete-penetration groove welds, the welding grade or electrode class is selected so that the resulting weld is as strong as the steel-joined. Partial-penetration groove welds, in which only part of the metal thickness is welded, are sometimes used when stresses are low and there is no need to develop the complete strength of the material. The stress area of such a weld is the product of the length of the weld and an effective throat thickness. In single J- or U-type joints, the effective throat thickness is equal to the depth of the groove, and in bevel- or V-type joints, it is equal to the depth of the chamfer or the depth of the chamfer minus ⅛ in, depending on the included angle and the welding process. AWS does not permit partial-penetration groove welds to be used for cyclic tension normal to the weld axis; also, if the weld is made from one side only, it must be restrained from rotation. AISC permits such welds to be used for cyclic loading, but the allowable stress range is only one-third to one-half that of a complete-penetration groove weld. Details of recommended types of joints are given by AWS.

In fillet welds, the load is transferred between the connected plates by shear stresses in the welds. The shear stress in a fillet weld is computed on an area equal to the product of the length of the weld by the effective throat thickness.

The **effective throat thickness** is the shortest distance from the root to the face of the weld, a flat face being assumed, and is 0.707 times the nominal size or leg of a fillet weld with equal legs. AISC specifies that the effective throat for submerged-arc fillet welds be taken equal to the leg size for welds $\frac{3}{8}$ in or less and to the theoretical throat plus 0.11 in for larger welds.

Plug welds and slot welds are occasionally used to transfer shear stresses between plates. The shear area for the weld is the nominal cross-sectional area of the hole or slot. This type of connection should be avoided because of the difficulty in inspecting to ensure a satisfactory weld and the severe stress concentration created.

The basic allowable stresses for welds in buildings and bridges are shown in Tables 9.19 and 9.20. As indicated in the tables, complete-penetration groove welds in building or bridge construction and certain other welds in building construction have the same allowable stress as the steel that is joined. The allowable stresses shown for fillet welds provide a safety factor against ultimate weld shear failure of about 3 for building construction and about 10% higher for bridge construction.

9.24 Stresses for Bolts

In bolted connections, shear is transmitted between connected parts by friction until slip occurs. Then, the load is resisted by shear on the bolts, bearing on the connected parts, and residual friction between the faying surfaces of those parts. Where slippage is undesirable, for example, when a joint would be subjected to frequent reversals of load direction, slip-critical joints, formerly called friction-type, may be specified. To prevent slippage, the parts are squeezed together by pretensioning the bolts during installation to create enough friction to resist service loads without slippage (Art. 9.27). High-strength bolts are required. A325 and A490 bolts are usually tightened to a minimum tension of at least 70% of the tensile strength.

In bearing-type connections, load is transferred between parts by shear on a bolt and bearing on the parts, and bolts may be tightened to a *snug-tight* condition (Art. 9.27). Higher shear stresses are permitted for high-strength bolts in such joints than in slip critical joints. Also, lower-cost, but lower-strength A307 bolts may be used.

Fasteners in Buildings ▪ The AISC Specification for allowable stresses for buildings (Art. 9.6) specifies allowable unit tension and shear stresses on the cross-sectional area on the unthreaded body area of bolts and threaded parts, as given in Table 9.21. (Generally, rivets should not be used in direct tension.) When wind or seismic load are combined with gravity loads, the allowable stresses may be increased one-third, where permitted by code.

Most building construction is done with bearing-type connections. Allowable bearing stresses apply to both bearing-type and slip-critical connections. In buildings the allowable bearing stress F_p, ksi, on projected area of fasteners with two or more bolts in the line of force is

$$F_p = 1.2F_u \tag{9.77}$$

where F_u is the tensile strength of the connected part, ksi. Distance measured in the line of force to the nearest edge of the connected part (end distance) should be at least $1.5d$, where d is the fastener diameter. The center-to-center spacing of fasteners should be at least $3d$.

Bridge Fasteners ▪ For bridges, AASHTO (Art. 9.6) specifies the working stresses for bolts listed in Table 9.22. Bearing-type connections with high-strength bolts are limited to members in compression and secondary members. The allowable bearing stress, ksi, in standard holes for high-strength bolts is

$$\frac{0.5L_cF_u}{d} \leq F_u \tag{9.78}$$

where L_c is the clear distance between holes or between hole and edge in the direction of the load, in; d is the bolt diameter, in; and F_u is the tensile strength of the connected material, ksi. For other than high-strength bolts, the bearing stress is limited by allowable bearing on the

Table 9.21 Allowable Stresses for Bolts in Buildings*

Type of Fastener	Tensile Stress, ksi	Shear Stress, ksi: Slip-Critical Connections	Shear Stress, ksi: Bearing-Type Connections
A307 bolts	20	Not applicable	10
Threaded parts of suitable steels, threads not excluded from shear plane†	$0.33F_u$	Not applicable	$0.17F_u$
Threaded parts of suitable steels, threads excluded from shear plane	$0.33F_u$	Not applicable	$0.22F_u$
A325 bolts, threads not excluded from shear plane	44	17	21
A325 bolts, threads excluded from shear plane	44	17	30
A490 bolts, threads not excluded from shear plane	54	21	28
A490 bolts, threads excluded from shear plane	54	21	40

* Stresses are for nominal bolt areas, except as noted, and are based on AISC Specifications. F_u is tensile strength, ksi. F_y is yield point, ksi. Allowable stresses are decreased for slip-critical joints with oversized or slotted holes.

Allowable stresses in tension are for static loads only, except for A325 and A490 bolts. For fatigue loadings, see AISC Specifications.

Allowable shear stresses for friction-type connections are for clean mill scale on faying surfaces. See AISC for allowable stresses for other surface conditions.

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, allowable shear stresses must be reduced by 20%.

† In addition, the tensile capacity of the threaded portion of an upset rod, based on the cross-sectional area at its major thread diameter, must be larger than the nominal body area of the rod before upsetting times $0.60F_y$.

fasteners. The allowable bearing stress on A307 bolts is 20 ksi, and on structural-steel rivets 40 ksi.

Combined Stresses in Fasteners ▪ The AISC and AASHTO Specifications for allowable-stress design provide formulas that limit unit stresses in bolts subjected to a combination of tension and shear.

For buildings, the allowable tension stress is based on the calculated shear stress f_v, ksi, with an upper limit on allowable tension based on type and grade of fastener. The shear stress for bearing-type joints, however, should not exceed the allowable shear in Table 9.21. For slip-critical joints, the allowable shear stress for bolts is based on the calculated tensile stress in the bolts, f_t, ksi, and the specified pretension on the bolts, kips.

For highway bridges, the shear and tension stresses for bolts are required to satisfy an interaction formula involving f_v, f_t, and the allowable shear shear stress given in Table 9.22. For slip-critical joints, the allowable shear stress is based on f_t.

9.25 Composite Construction

In composite construction, steel beams and a concrete slab are connected so that they act together to resist the load on the beam. The slab, in effect, serves as a cover plate. As a result, a lighter steel section may be used.

Construction in Buildings ▪ There are two basic methods of composite construction.

Method 1. The steel beam is entirely encased in the concrete. Composite action in this case depends on steel-concrete bond alone. Since the beam is completely braced laterally, the allowable stress in the flanges is $0.66F_y$, where F_y is the yield strength, ksi, of the steel. Assuming the steel to

Table 9.22 Allowable Stresses for Bolts in Bridges*

Type of Fastener	Tensile Stress, ksi	Shear Stress, ksi: Slip-Critical Connections	Shear Stress, ksi: Bearing-Type Connections
A307 bolts	18.0†	Not applicable	11.0
A325 bolts, threads not excluded from shear plane	38.0	15.0	19.0
A325 bolts, threads excluded from shear plane	38.0	15.0	23.8
A490 bolts, threads not excluded from shear plane	47.0	19.0	24.0
A490 bolts, threads excluded from shear plane	47.0	19.0	30.0

* Stresses are for nominal bolt area, except as noted, and are based on AASHTO Specifications. AASHTO specifies reduced shear stress values under certain conditions.

For fatigue loadings, see AASHTO Specifications.

Allowable shear stresses for friction-type connections are for clean mill scale on faying surfaces. See AASHTO for allowable stresses for other surface conditions.

In bearing-type connections whose length between extreme fasteners in each of the spliced parts measured parallel to the line of an axial force exceeds 50 in, allowable shear stresses must be reduced by 20%.

† Based on area at the root of thread.

carry the full dead load and the composite section to carry the live load, the maximum unit stress, ksi, in the steel is

$$f_s = \frac{M_D}{S_s} + \frac{M_L}{S_{tr}} \leq 0.66F_y \tag{9.79}$$

where M_D = dead-load moment, in-kips

M_L = live-load moment, in-kips

S_s = section modulus of steel beam, in^3

S_{tr} = section modulus of transformed composite section, in^3

An alternative, shortcut method is permitted by the AISC Specification (Art. 9.6). It assumes the steel beam will carry both live and dead loads and compensates for this by permitting a higher stress in the steel:

$$f_s = \frac{M_D + M_L}{S_s} \leq 0.76F_y \tag{9.80}$$

Method 2. The steel beam is connected to the concrete slab by shear connectors. Design is based on ultimate load and is independent of use of temporary shores to support the steel until the concrete hardens. The maximum stress in the bottom flange is

$$f_s = \frac{M_D + M_L}{S_{tr}} \leq 0.66F_y \tag{9.81}$$

To obtain the transformed composite section, treat the concrete above the neutral axis as an equivalent steel area by dividing the concrete area by n, the ratio of modulus of elasticity of steel to that of the concrete. In determination of the transformed section, only a portion of the concrete slab over the beam may be considered effective in resisting compressive flexural stresses (positive-moment regions). None of the concrete is assumed capable of resisting tensile flexural stresses, although the longitudinal steel reinforcement in the effective width of slab may be included in the computation of the properties of composite beams if shear connectors are provided. The width of slab on either side of the beam centerline that may be considered effective should not exceed any of the following:

1. One-eighth of the beam span between centers of supports
2. Half the distance to the centerline of the adjacent beam

3. The distance from beam centerline to edge of slab (see Fig. 9.8)

When the steel beam is not shored during the casting of the concrete slab, the steel section alone should be considered to be carrying all loads until the concrete attains 75% of its required strength. Stresses in the steel should not exceed $0.90F_y$ for this condition in building construction. Later, the compressive flexural stress in the concrete should not exceed 45% of its specified compressive strength f'_c. The section modulus used for calculating that stress should be that of the transformed composite section.

9.25.1 Shear on Connectors

The total horizontal shear to be resisted by the shear connectors in building construction is taken as the smaller of the values given by Eqs. (9.82) and (9.83).

$$V_h = \frac{0.85f'_cA_c}{2} \tag{9.82}$$

$$V_h = \frac{A_sF_y}{2} \tag{9.83}$$

where V_h = total horizontal shear, kips, between maximum positive moment and each end of steel beams (or between point of maximum positive moment and point of contraflexure in continuous beam)

f'_c = specified compressive strength of concrete at 28 days, ksi

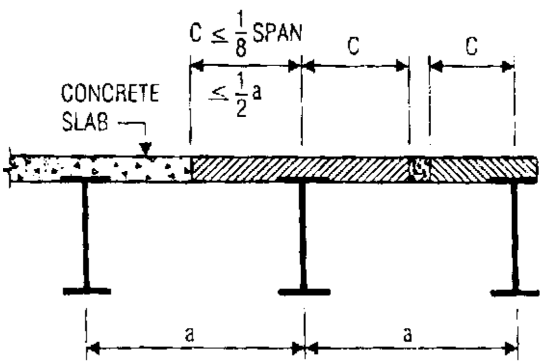

Fig. 9.8 Limitations on effective width of concrete slab in a composite steel-concrete beam.

A_c = actual area of effective concrete flange, in^2

A_s = area of steel beam, in^2

In continuous composite construction, longitudinal reinforcing steel may be considered to act compositely with the steel beam in negative-moment regions. In this case, the total horizontal shear, kips, between an interior support and each adjacent point of contraflexure should be taken as

$$V_h = \frac{A_{sr}F_{yr}}{2} \tag{9.84}$$

where A_{sr} = area of longitudinal reinforcement at support within effective area, in^2

F_{yr} = specified minimum yield stress of longitudinal reinforcement, ksi

9.25.2 Number of Connectors Required for Building Construction

The total number of connectors to resist V_h is computed from V_h/q, where q is the allowable shear for one connector, kips. Values of q for connectors in buildings are in Table 9.23.

Table 9.23 is applicable only to composite construction with concrete made with stone aggregate conforming to ASTM C33. For lightweight concrete weighing at least 90 lb/ft^3 and made with rotary-kiln-produced aggregates conforming to ASTM C330, the allowable shears in Table 9.23 should be reduced by multiplying by the appropriate coefficient of Table 9.24.

The required number of shear connectors may be spaced uniformly between the sections of maximum and zero moment. Shear connectors should have at least 1 in of concrete cover in all directions, and unless studs are located directly over the web, stud diameters may not exceed 2.5 times the beam-flange thickness.

With heavy concentrated loads, the uniform spacing of shear connectors may not be sufficient between a concentrated load and the nearest point of zero moment. The number of shear connectors in this region should be at least

$$N_2 = \frac{N_1[(M\beta/M_{\max}) - 1]}{\beta - 1} \tag{9.85}$$

Table 9.23 Allowable Shear Loads on Connectors for Composite Construction in Buildings

Type of Connector	Allowable Horizontal Shear Load q, kips (Applicable Only to Concrete Made with ASTM C33 Aggregates) f_c', ksi 3.0	3.5	4.0
½-in dia × 2-in hooked or headed stud	5.1	5.5	5.9
⅝-in dia × 2½-in hooked or headed stud	8.0	8.6	9.2
¾-in dia × 3-in hooked or headed stud	11.5	12.5	13.3
⅞-in dia × 3½-in hooked or headed stud	15.6	16.8	18.0
3-in channel, 4.1 lb	4.3w*	4.7w	5.0w
4-in channel, 5.4 lb	4.6w	5.0w	5.3w
5-in channel, 6.7 lb	4.9w	5.3w	5.6w

* w = length of channel, in.

where M = moment at concentrated load, ft-kips

$M_{\max}$ = maximum moment in span, ft-kips

N_1 = number of shear connectors required between $M_{\max}$ and zero moment

$\beta = S_{tr}/S_s$ or S_{eff}/S_s, as applicable

S_{eff} = effective section modulus for partial composite action, in^3

9.25.3 Partial Composite Construction

This is used when the number N_1 of shear connectors required would provide a beam considerably stronger than necessary. In that case, the effective section modulus is used in stress computation instead of the transformed section modulus, and S_{eff} is calculated from Eq. (9.86).

$$S_{\text{eff}} = S_s + \sqrt{\frac{V_h'}{V_h}}(S_{tr} - S_s) \qquad (9.86)$$

where V_h' = number of shear connectors provided times allowable shear load q of Table 9.23 (times coefficient of Table 9.24, if applicable).

Composite construction of steel beams and concrete slab cast on a cold-formed-steel deck can also be designed with the information provided, but certain modifications are required as described in the AISC Specification for allowable stress. Various dimensional requirements must be met. Also, the allowable shear loads for the stud connectors must be multiplied by a reduction factor. The ribs in the steel deck may be oriented perpendicular to or parallel to the steel beam or girder. The studs are typically welded directly through the deck by following procedures recommended by stud manufacturers.

9.25.4 Composite Construction in Highway Bridges

Shear connectors between a steel girder and a concrete slab in composite construction in a highway

Table 9.24 Shear Coefficient for Lightweight Concrete with Aggregates Conforming to ASTM C330

	Air-Dry Unit Weight, lb/ft^3 90	95	100	105	110	115	120
When $f_c' \le 4$ ksi	0.73	0.76	0.78	0.81	0.83	0.86	0.88
When $f_c' \ge 5$ ksi	0.82	0.85	0.87	0.91	0.93	0.96	0.99

bridge should be capable of resisting both horizontal and vertical movement between the concrete and steel. Maximum spacing for shear connectors generally is 24 in, but wider spacing may be used over interior supports, to avoid highly stressed portions of the tension flange (Fig. 9.9). Clear depth of concrete cover over shear connectors should be at least 2 in and they should extend at least 2 in above the bottom of the slab. In simple spans and positive-moment regions of continuous spans, composite sections generally should be designed to keep the neutral axis below the top of the steel girder. In negative-moment regions, the concrete is assumed to be incapable of resisting tensile stresses, but the longitudinal steel reinforcement may be considered to participate in the composite action if shear connectors are provided.

For composite action, stresses should be computed by the moment-of-inertia method for a transformed composite section, as for buildings, except that the AASHTO Specification (Art. 9.6) requires that the effect of creep be included in the computations. When shores are used and kept in place until the concrete has attained 75% of its specified 28-day strength, the stresses due to dead and live loads should be computed for the composite section.

Creep and Shrinkage ▪ AASHTO requires that the effects of creep be considered in the design of composite beams with dead loads acting on the composite section. For such beams, tension, compression, and horizontal shears produced by dead loads acting on the composite section should be computed for n (Table 9.25) or $3n$, whichever gives the higher stresses.

Shrinkage also should be considered. Resistance of a steel beam to longitudinal contraction of the concrete slab produces shear stresses along the contact surface. Associated with this shear are tensile stresses in the slab and compressive stresses in the steel top flange. These stresses also affect the beam deflection. The magnitude of the shrinkage effect varies within wide limits. It can be qualitatively reduced by appropriate casting sequences, for example, by placing concrete in a checkerboard pattern.

Table 9.25 Ratio of Moduli of Elasticity of Steel and Concrete for Bridges

f_c' for Concrete	$n = E_s/E_c$
2.0–2.3	11
2.4–2.8	10
2.9–3.5	9
3.6–4.5	8
4.6–5.9	7
6.0 and over	6

The steel beams or girders should be investigated for strength and stability for the loading applied during the time the concrete is in place and before it has hardened. The casting or placing sequence specified for the deck must be considered when calculating moments and shears in the beams or girders.

Span-Depth Ratios ▪ In bridges, for composite beams, preferably the ratio of span to steel beam depth should not exceed 30 and the ratio of span to depth of steel beam plus slab should not exceed 25.

Effective Width of Slabs ▪ For a composite interior girder, the effective width assumed for

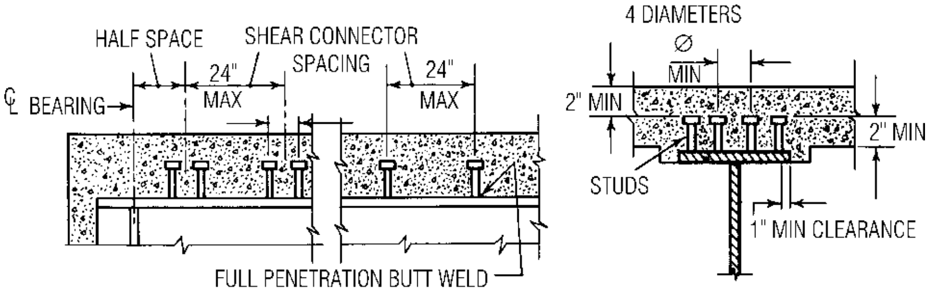

Fig. 9.9 Maximum pitch for stud shear connectors in composite beams.

the concrete flange should not exceed any of the following:

1. One-fourth the beam span between centers of supports
2. Distance between centerlines of adjacent girders
3. Twelve times the least thickness of the slab

For a girder with the slab on only one side, the effective width of slab should not exceed any of the following:

1. One-twelfth the beam span between centers of supports
2. Half the distance to the centerline of the adjacent girder
3. Six times the least thickness of the slab

Bending Stresses ▪ In composite beams in bridges, stresses depend on whether or not the members are shored; they are determined as for beams in buildings [see Eqs. (9.79) and (9.81)], except that the stresses in the steel may not exceed $0.55F_y$ [see Eqs. (9.87) and (9.88)].

Unshored:

$$f_s = \frac{M_D}{S_s} + \frac{M_L}{S_{tr}} \leq 0.55F_y \tag{9.87}$$

Shored:

$$f_s = \frac{M_D + M_L}{S_{tr}} \leq 0.55F_y \tag{9.88}$$

Shear Range ▪ Shear connectors in bridges are designed for fatigue and then are checked for ultimate strength. The horizontal-shear range for fatigue is computed from

$$S_r = \frac{V_r Q}{I} \tag{9.89}$$

where S_r = horizontal-shear range at juncture of slab and beam at point under consideration, kips/lin in

V_r = shear range (difference between minimum and maximum shears at the point) due to live load and impact, kips

Q = static moment of transformed compressive concrete area about neutral axis of transformed section, in^3

I = moment of inertia of transformed section, in^4

The transformed area is the actual concrete area divided by n (Table 9.25).

The allowable range of horizontal shear Z_r, kips, for an individual connector is given by Eq. (9.90) or (9.91), depending on the connector used.

For channels (with a minimum of $\frac{3}{16}$-in fillet welds along heel and toe):

$$Z_r = Bw \tag{9.90}$$

where w = channel length, in, in transverse direction on girder flange

B = cyclic variable = 4.0 for 100,000 cycles, 3.0 for 500,000 cycles, 2.4 for 2 million cycles, 2.1 for over 2 million cycles

For welded studs (with height-diameter ratio $H/d \geq 4$):

$$Z_r = \alpha d^2 \tag{9.91}$$

where d = stud diameter, in

α = cyclic variable = 13.0 for 100,000 cycles, 10.6 for 500,000 cycles, 7.85 for 2 million cycles, 5.5 for over 2 million cycles

Required pitch of shear connectors is determined by dividing the allowable range of horizontal shear of all connectors at one section Z_r, kips, by the horizontal range of shear S_r, kips/lin in.

Number of Connectors in Bridges ▪ The ultimate strength of the shear connectors is checked by computation of the number of connectors required from

$$N = \frac{P}{\phi S_u} \tag{9.92}$$

where N = number of shear connectors between maximum positive moment and end supports

S_u = ultimate shear connector strength, kips [see Eqs. (9.97) and (9.98) and Table 9.26]

ϕ = reduction factor = 0.85

P = force in slab, kips

At points of maximum positive moments, P is the smaller of P_1 and P_2, computed from Eqs. (9.93 and 9.94).

$$P_1 = A_s F_y \tag{9.93}$$

$$P_2 = 0.85 f'_c A_c \tag{9.94}$$

Table 9.26 Ultimate Horizontal-Shear Load for Connectors in Composite Beams in Bridges*

Type of Connector	Ultimate Shear, kips, for Concrete Compressive Strength f'_c, ksi		
	f'_c, ksi		
	3.0	3.5	4.0
Welded stud			
$\frac{3}{4}$-in dia × 3 in min high	21.8	24.4	27.0
$\frac{7}{8}$-in dia × 3.5 in min high	29.6	33.3	36.8
Rolled channel†			
3-in deep, 4.1 lb/ft	10.78w	11.65w	12.45w
4-in deep, 5.4 lb/ft	11.69w	12.62w	13.50w
5-in deep, 6.7 lb/ft	12.50w	13.50w	14.43w

* The values are based on the requirements of AASHTO and include no safety factor. Values are for concrete with unit weight of 144 lb/ft^3.
† w is channel length, in.

where A_c = effective concrete area, in^2

f'_c = 28-day compressive strength of concrete, ksi

A_s = total area of steel section, in^2

F_y = steel yield strength, ksi

The number of connectors required between points of maximum positive moment and points of adjacent maximum negative moment should equal or exceed N_2, given by

$$N_2 = \frac{P + P_3}{\phi S_u} \tag{9.95}$$

At points of maximum negative moments, the force in the slab P_3, is computed from

$$P_3 = A_{sr} F_{yr} \tag{9.96}$$

where A_{sr} = area of longitudinal reinforcing within effective flange, in^2

F_{yr} = reinforcing steel yield strength, ksi

Ultimate Shear Strength of Connectors, kips, in Bridges ▪ For channels:

$$S_u = 17.4\left(h + \frac{t}{2}\right) w \sqrt{f'_c} \tag{9.97}$$

where h = average channel-flange thickness, in

t = channel-web thickness, in

w = channel length, in

For welded studs ($H/d \geq 4$ in):

$$S_u = 0.4 d^2 \sqrt{f'_c E_c} \tag{9.98}$$

where E_c is the modulus of elasticity of the concrete, ksi, given by $E_c = 1.044 w^{3/2} \sqrt{f'_c}$ and w is the unit weight of concrete, lb/ft^3.

Table 9.26 gives the ultimate shear for connectors as computed from Eqs. (9.97) and (9.98) for some commonly used concrete strengths.

9.26 Bracing

It usually is necessary to provide bracing for the main members or secondary members in most buildings and bridges.

9.26.1 In Buildings

There are two general classifications of bracing for building construction: sway bracing for lateral loads and lateral bracing to increase the capacity of individual beams and columns.

Both low- and high-rise buildings require sway bracing to provide stability to the structure and to resist lateral loads from wind or seismic forces. This bracing can take the form of diagonal members or X bracing, knee braces, moment connections, and shear walls.

X bracing is probably the most efficient and economical bracing method. Fenestration or architectural considerations, however, often preclude it. This is especially true for high-rise structures.

Knee braces are often used in low-rise industrial buildings. They can provide local support to the column as well as stability for the overall structure.

Moment connections are frequently used in high-rise buildings. They can be welded, riveted, or bolted, or a combination of welds and bolts can be used. End-plate connections, with shop welding and field bolting, are an economical alternative. Figure 9.10 shows examples of various end-plate moment connections.

In many cases, moment connections may be used in steel frames to provide continuity and thus reduce the overall steel weight. This type of framing is especially suitable for welded construction; full moment connections made with bolts may be cumbersome and expensive.

In low buildings and the top stories of high buildings, moment connections may be designed to resist lateral forces alone. Although the overall steel weight is larger with this type of design, the connections are light and usually inexpensive.

Shear walls are also used to provide lateral bracing in steel-framed buildings. For this purpose, it often is convenient to reinforce the walls needed for other purposes, such as fire walls, elevator shafts, and divisional walls. Sometimes shear walls are used in conjunction with other forms of bracing. Steel plate shear walls have also proven effective.

For plastic-design of multistory frames under factored gravity loads (service loads times 1.7) or under factored gravity plus wind loads (service loads times 1.3), unbraced frames may be used if designed to preclude instability, including the effects of axial deformation of columns. The factored column axial loads should not exceed $0.75AF_y$. Otherwise, frames should incorporate a vertical bracing system to maintain lateral stability. This vertical system may be used in selected braced bents that must carry not only horizontal loads directly applied to them but also the horizontal loads of unbraced bents. The latter loads may be transmitted through diaphragm action of the floor system.

Lateral bracing of columns, arches, beams, and trusses in building construction is used to reduce their critical or effective length, especially of those portions in compression. In floor or roof systems, for instance, it may be economical to provide a strut at midspan of long members to obtain an increase in the allowable stress for the load-carrying members. (See also Arts. 9.11 and 9.12 for effects on allowable stresses of locations of lateral supports.)

Usually, normal floor and roof decks can be relied on to provide sufficient lateral support to compression chords or flanges to warrant use of the full allowable compressive stress. Examples of cases where it might be prudent to provide supplementary support include purlins framed into beams well below the compression flange or precast-concrete planks inadequately secured to the beams.

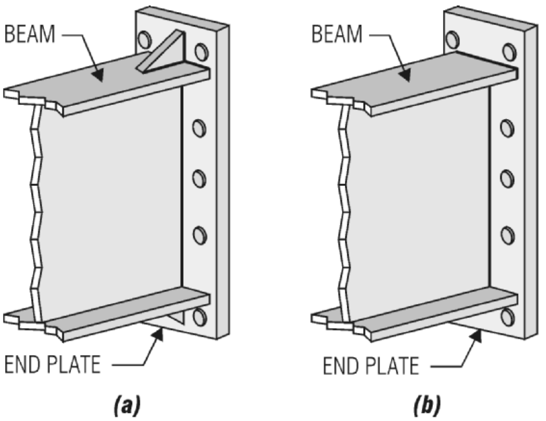

Fig. 9.10 End-plate connections for girders: (*a*) stiffened moment connection; (*b*) unstiffened moment connection.

9.26.2 In Bridges

Bracing requirements for highway bridges are given in detail in "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials.

Through trusses require top and bottom lateral bracing. Top lateral bracing should be at least as deep as the top chord. Portal bracing of the two-plane or box type is required at the end posts and should take the full end reaction of the top-chord lateral system. In addition, sway bracing at least 5 ft deep is required at each intermediate panel point.

Deck-truss spans and spandrel arches also require top and bottom lateral bracing. Sway bracing, extending the full depth of the trusses, is required in the plane of the end posts and at all intermediate panel points. The end sway bracing carries the entire upper lateral stress to the supports through the end posts of the truss.

A special case arises with a half-through truss because top lateral bracing is not possible. The main truss and the floor beams should be designed for a lateral force of 300 lb/lin ft, applied at the top-chord panel points. The top chord should be treated as a column with elastic lateral supports at each panel point. The critical buckling force should be at at least 50% greater than the maximum force from dead load, live load, and impact in any panel of the top chord.

Lateral bracing is not usually necessary for deck plate-girder or beam bridges. Most deck construction is adequate as top bracing, and substantial diaphragms (with depth preferably half the girder depth) or cross frames obviate the necessity of bottom lateral bracing. Cross frames are required at each end to resist lateral loads.The need for lateral bracing should be investigated with the use of equations and wind forces specified by AASHTO.

Through-plate girders should be stiffened against lateral deformation by gusset plates or knee braces attached to the floor beams. If the unsupported length of the inclined edge of a gusset plate exceeds $350/\sqrt{F_y}$ times the plate thickness, it should be stiffened with angles.

All highway bridges should be provided with cross frames or diaphragms spaced at a maximum of 25 ft.

("Detailing for Steel Construction," American Institute of Steel Construction, www.aisc.org.)

9.27 Mechanical Fasteners

Unfinished bolts are used mainly in building construction where slip and vibration are not a factor. Characterized by a square head and nut, they also are known as machine, common, ordinary, or rough bolts. They are covered by ASTM A307 and are available in diameters over a wide range (see also Art. 9.2).

A325 bolts are identified by the notation A325. Additionally, Type 1 A325 bolts may optionally be marked with three radial lines 120° apart; Type 2 A325 bolts, withdrawn in 1991, were marked with three radial lines 60° apart; and Type 3 A325 bolts must have the A325 notation underlined. Heavy hexagonal nuts of the grades designated in A325 are manufactured and marked according to specification A563.

A490 bolts are identified by the notation A490. Additionally, Type 2 A490 bolts must be marked with six radial lines 30° apart, and Type 3 A490 bolts must have the A490 notation underlined. Heavy hexagonal nuts of the grades designated in A490 are manufactured and marked according to specification A563.

9.27.1 Types of Bolted Connections

Two different types of bolted connections are recognized for bridges and buildings: bearing and slip critical. Bearing-type connections are allowed higher shear stresses and thus require fewer bolts. Slip-critical connections offer greater resistance to repeated loads and therefore are used when connections are subjected to stress reversal or where slippage would be undesirable. See Art. 9.24.

9.27.2 Symbols for Bolts and Rivets

These are used to denote the type and size of rivets and bolts on design drawings as well as on shop and erection drawings. The practice for buildings and bridges is similar.

Figure 9.11 shows the conventional signs for rivets and bolts.

9.27.3 Bolt Tightening

High-strength bolts for bearing-type connections can generally be installed in the *snug-tight* condition. This is the tightness that exists when all plies in the joint are in firm contact, and may be obtained by a few impacts of an impact wrench or by a full manual effort with a spud wrench. High-strength bolts in slip-critical connections and in connections that are subject to direct tension must be fully pretensioned. Such bolts can be tightened by a calibrated wrench or by the *turn-of-the-nut* method. Calibrated wrenches are powered and have an automatic cutoff set for a predetermined torque. With this method, a hardened washer must be used under the element turned.

The turn-of-the-nut method requires snugging the plies together and then turning the nut a specified amount. From one-third to one turn is specified; increasing amounts of turn are required for long bolts or for bolts connecting parts with slightly sloped surfaces. Alternatively, a direct tension indicator,

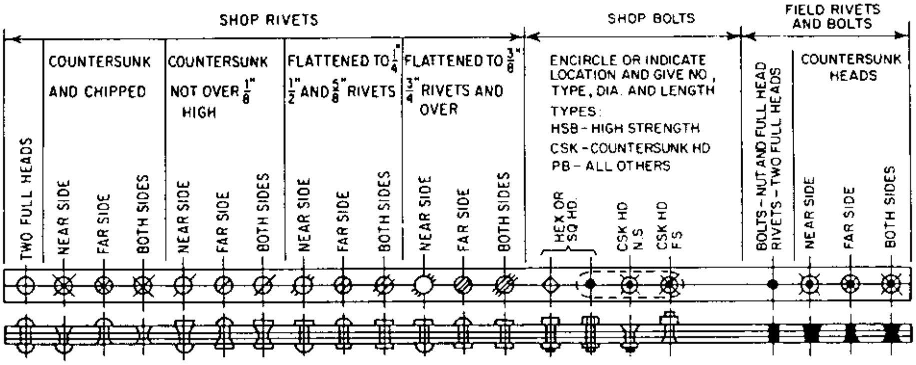

Fig. 9.11 Conventional symbols for bolts and rivets.

such as a load-indicating washer, may be used. This type of washer has on one side raised surfaces which when compressed to a predetermined height (0.005 in measured with a feeler gage) indicate attainment of required bolt tension. Another alternative is to use fasteners that automatically provide the required tension, such as by yielding of or twisting off of an element. The Research Council on Structural Connections "Specification for Structural Steel Joints Using A325 or A490 Bolts," gives detailed specifications for all tightening methods.

Table 9.27 Oversized- and Slotted-Hole Limitations for Structural Joints with A325 and A490 Bolts

Bolt Diameter, in	Maximum Hole Size, in*		
	Oversize Holes†	Short Slotted Holes‡	Long Slotted Holes‡
$\frac{1}{2}$	$\frac{5}{8}$	$\frac{9}{16} \times \frac{11}{16}$	$\frac{9}{16} \times 1\frac{1}{4}$
$\frac{5}{8}$	$\frac{13}{16}$	$\frac{11}{16} \times \frac{7}{8}$	$\frac{11}{16} \times 1\frac{9}{16}$
$\frac{3}{4}$	$\frac{15}{16}$	$\frac{13}{16} \times 1$	$\frac{13}{16} \times 1\frac{7}{8}$
$\frac{7}{8}$	$1\frac{1}{16}$	$\frac{15}{16} \times 1\frac{1}{8}$	$\frac{15}{16} \times 2\frac{3}{16}$
1	$1\frac{1}{4}$	$1\frac{1}{16} \times 1\frac{5}{16}$	$1\frac{1}{16} \times 2\frac{1}{2}$
$1\frac{1}{8}$	$1\frac{7}{16}$	$1\frac{3}{16} \times 1\frac{1}{2}$	$1\frac{3}{16} \times 2\frac{13}{16}$
$1\frac{1}{4}$	$1\frac{9}{16}$	$1\frac{5}{16} \times 1\frac{5}{8}$	$1\frac{5}{16} \times 3\frac{1}{8}$
$1\frac{3}{8}$	$1\frac{11}{16}$	$1\frac{7}{16} \times 1\frac{3}{4}$	$1\frac{7}{16} \times 3\frac{7}{16}$
$1\frac{1}{2}$	$1\frac{13}{16}$	$1\frac{9}{16} \times 1\frac{7}{8}$	$1\frac{9}{16} \times 3\frac{3}{4}$

* In slip-critical connections, a lower allowable shear stress, as given by AISC, should be used for the bolts.
† Not allowed in bearing-type connections.
‡ In bearing-type connections, slot must be perpendicular to direction of load application.

9.27.4 Holes

These generally should be $\frac{1}{16}$ in larger than the nominal fastener diameter. Oversize and slotted holes may be used subject to the limitations of Table 9.27.

("Detailing for Steel Construction," American Institute of Steel Construction.)

9.28 Welded Connections

Welding, a method of joining steel by fusion, is used extensively in both buildings and bridges. It usually requires less connection material than other methods. No general rules are possible regarding the economics of the various connection methods; each job must be individually analyzed.

Although there are many different welding processes, shielded-arc welding is used almost exclusively in construction. Shielding serves two purposes: It prevents the molten metal from oxidizing and it acts as a flux to cause impurities to float to the surface.

In manual arc welding, an operator maintains an electric arc between a coated electrode and the work. Its advantage lies in its versatility; a good operator can make almost any type of weld. It is used for fitting up as well as for finished work. The coating turns into a gaseous shield, protecting the weld and concentrating the arc for greater penetrative power.

Automatic welding, generally the submerged-arc process, is used in the shop, where long lengths of welds in the flat position are required. In this method, the electrode is a base wire (coiled) and the arc is protected by a mound of granular flux fed to the work area by a separate flux tube. Most welded bridge girders are fabricated by this method, including the welding of transverse stiffeners. Other processes, such as gas metal or flux-cored arc welding, are also used.

There are basically two types of welds: fillet and groove. Figure 9.12 shows conventional symbols for welds, and Figs. 9.13 to 9.15 illustrate typical fillet, complete-penetration groove, and partial-

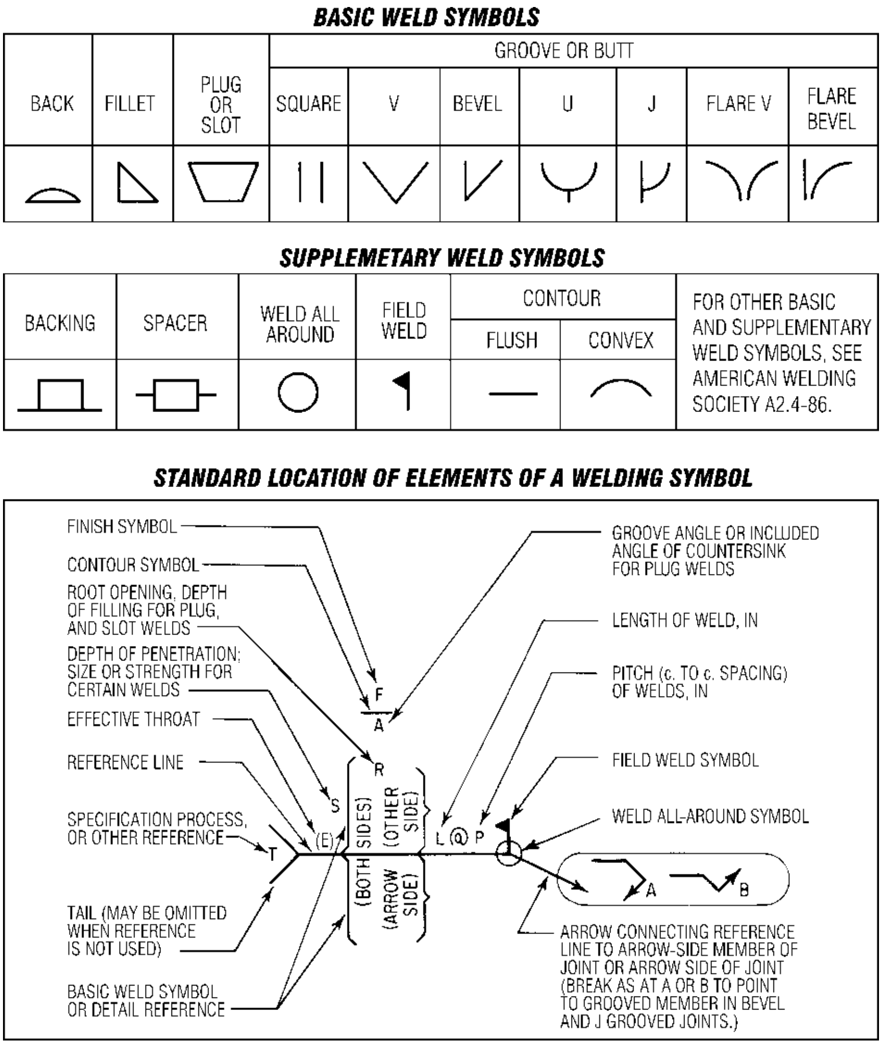

Fig. 9.12 Symbols recommended by the American Welding Society for welded joints. Size, weld symbol, length of weld, and spacing should read in that order from left to right along the reference line, regardless of its orientation or arrow location. The perpendicular leg of symbols for fillet, bevel, J, and flare-bevel-groove should be on the left. Arrow and Other Side welds should be the same size. Symbols apply between abrupt changes in direction of welding unless governed by the *all-around* symbol or otherwise dimensioned. When billing of detail material discloses the existence of a member on the far side (such as a stiffened web or a truss gusset), welding shown for the near side should also be duplicated on the far side.

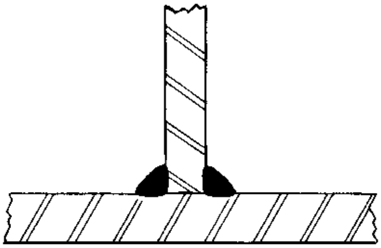

Fig. 9.13 Typical fillet welds.

Fig. 9.14 Typical complete-penetration groove weld.

Fig. 9.15 Typical partial-penetration groove weld.

penetration groove welds. AISC (Art. 9.6) permits partial-penetration groove welds with a reduction in allowable stress. AASHTO (Art. 9.6) does not allow partial-penetration groove welds for bridges where tension may be applied normal to the axis of the weld. Allowable stresses for welds in buildings and bridges are presented in Art. 9.19.

("Detailing for Steel Construction," American Institute of Steel Construction.)

9.29 Combinations of Fasteners

In new construction, different types of fasteners (bolts, or welds) are generally not combined to share the same load because varying amounts of deformation are required to load the different fasteners properly. AISC (Art. 9.6) permits one exception to this rule: Slip-critical bolted connections may be used with welds if the bolts are tightened prior to welding. When welding is used in alteration of existing building framing, existing rivets and existing high-strength bolts in slip-critical connections may be assumed to resist stresses from loads present at the time of alteration, and the welding may be designed to carry only the additional stresses.

9.30 Column Splices

Connections between lengths of a compression member are often designed more as an erection device than as stress-carrying elements.

Building columns usually are spliced at every second or third story, about 2 ft above the floor. AISC (Art. 9.6) requires that the connectors and splice material be designed for 50% of the stress in the columns. In addition, they must be proportioned to resist tension that would be developed by lateral forces acting in conjunction with 75% of the calculated dead-load stress and without live load.

The AISC "Manual of Steel Construction" (ASD and LRFD) illustrates typical column splices for riveted, bolted, and welded buildings. Where joints depend on contact bearing as part of the splice capacity, the bearing surfaces may be prepared by milling, sawing or suitable means.

Bridge Splices ▪ AASHTO (Art. 9.6) requires splices (tension, compression, bending, or shear) to be designed for the average of the stress at the point of splice and the strength of the member but not less than 75% of the strength of the member. Splices in truss chords should be located as close as possible to panel points.

In bridges, if the ends of columns to be spliced are milled, the splice bolts can be designed for 50 percent of the lower allowable stress of the section spliced. In buildings, AISC permits other means of surfacing the end, such as sawing, if the end is accurately finished to a true plane.

("Detailing for Steel Construction," American Institute of Steel Construction.)

9.31 Beam Splices

Connections between lengths of a beam or girder are designed as either shear or moment connections (Fig. 9.16) depending on their location and function in the structure. In cantilever or hung-span construction in buildings, where beams are extended over the tops of columns and spliced, or connected by another beam, it is sometimes

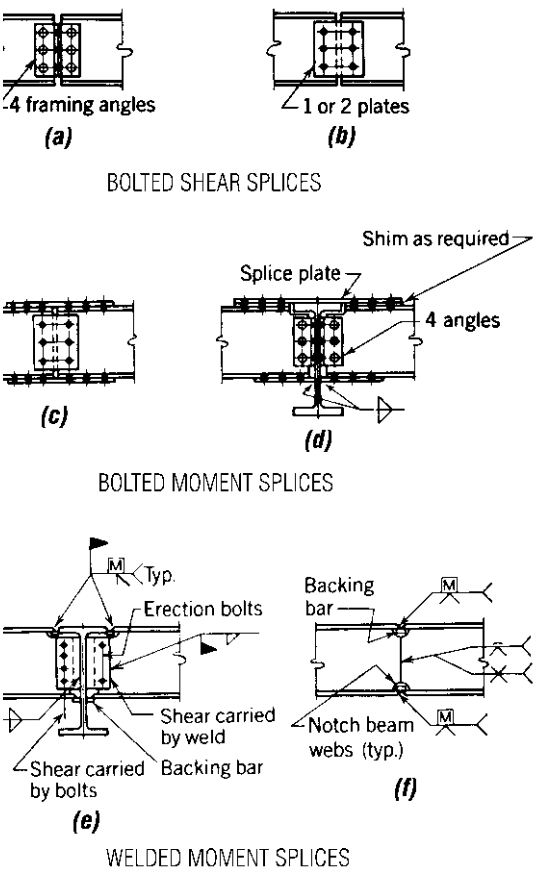

Fig. 9.16 Examples of beam splices used in building construction.

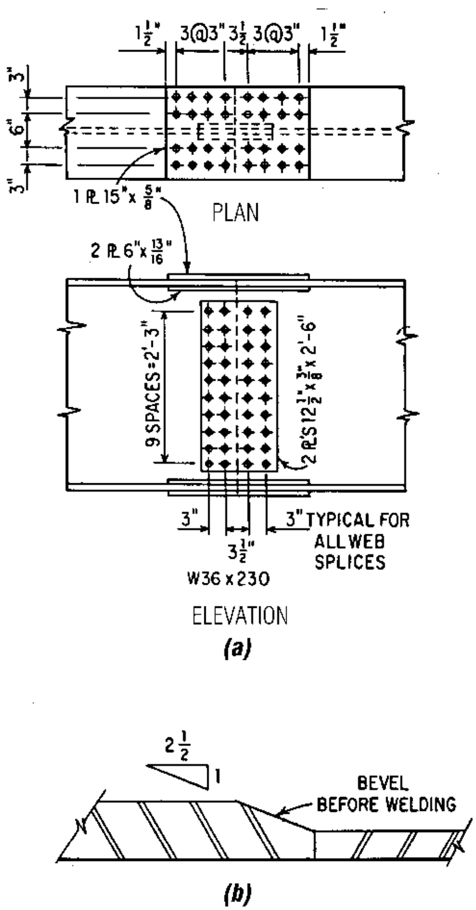

Fig. 9.17 Bridge-beam splices: (*a*) Bolted moment splice. (*b*) Welded flange splice.

possible to use only a shear splice (Fig. 9.16*a* and *b*) if no advantage is taken of continuity and alternate span loading is not likely. Otherwise, at least a partial moment splice is required, depending on the loading and span conditions. Splices may be welded or bolted. The AISC "Manual of Steel Construction" (ASD and LRFD) illustrates typical beam splices.

For continuous bridges, beam splices are designed for the full moment capacity of the beam or girder and are usually bolted (Fig. 9.17*a*). Field-welded splices, although not so common as field-bolted splices, may be an economical alternative.

Special flange splices are always required on welded girders where the flange thickness changes. Care must be taken to ensure that the stress flow is uniform. Figure 9.17*b* shows a typical detail.

("Detailing for Steel Construction," American Institute of Steel Construction.)

9.32 Erecting Structural Steel

Structural steel is erected by either hand-hoisting or power-hoisting devices.

The simplest hand device is the **gin pole** (Fig. 9.18). The pole is usually a sound, straight-grained timber, although metal poles can also be used. The guys, made of steel strands, generally are set at an angle of 45° or less with the pole. The

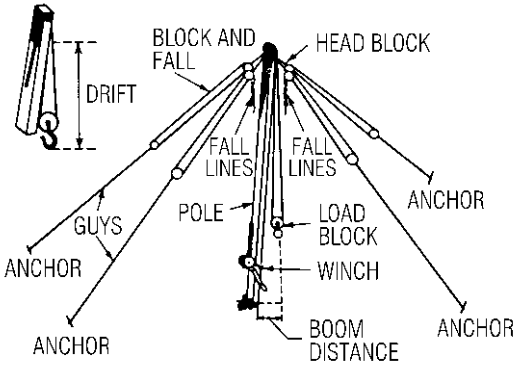

Fig. 9.18 Gin pole.

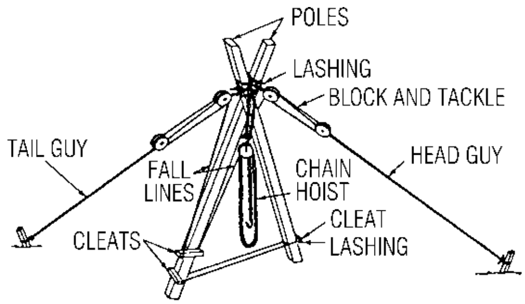

Fig. 9.19 A or shear-leg frame.

hoisting line may be manila or wire rope. The capacity of a gin pole is determined by the strength of the guys, hoist line, winch, hook, supporting structure, and the pole itself.

There are several variations of gin poles, such as the **A frame** (Fig. 9.19) and the **Dutchman** (Fig. 9.20).

A **stiffleg derrick** consists of a boom, vertical mast, and two inclined braces, or stifflegs (Fig. 9.21). It is provided with a special winch, which is furnished with hoisting drums to provide separate load and boom lines. After the structural frame of a high building has been completed, a stiffleg may be installed on the roof to hoist building materials, mechanical equipment, and so forth to various floors.

Guy derricks (Fig. 9.22) are advantageous in erecting multistory buildings. These derricks can jump themselves from one story to another. The boom temporarily serves as a gin pole to hoist the mast to a higher level. The mast is then secured in place and, acting as a gin pole, hoists the boom into its next position. Slewing (rotating) the derrick may be handled manually or by power.

A **Chicago boom** is a lifting device that uses the structure being erected to support the boom (Fig. 9.23).

Cranes are powered erection equipment consisting primarily of a rotating cab with a counterweight and a movable boom (Fig. 9.24). Sections of boom may be inserted and removed, and jibs may be added to increase the reach.

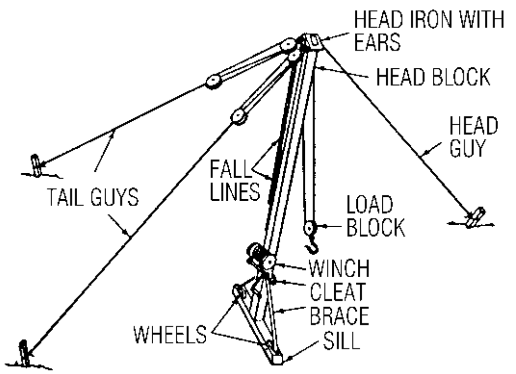

Fig. 9.20 Dutchman.

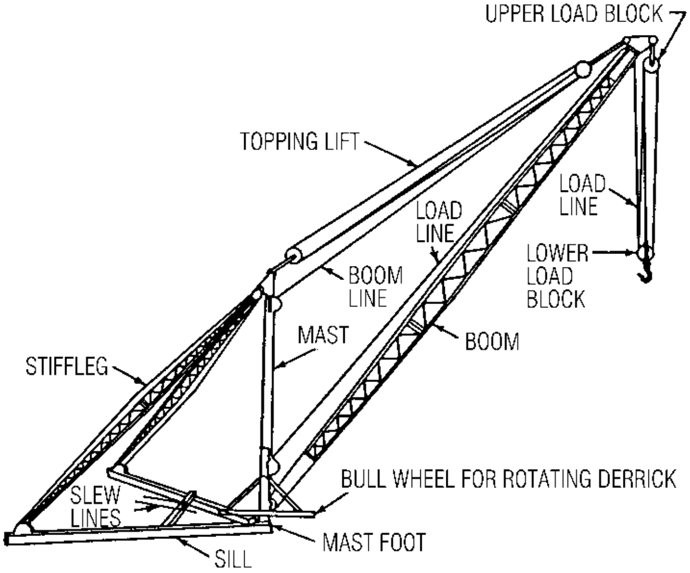

Fig. 9.21 Stiffleg derrick.

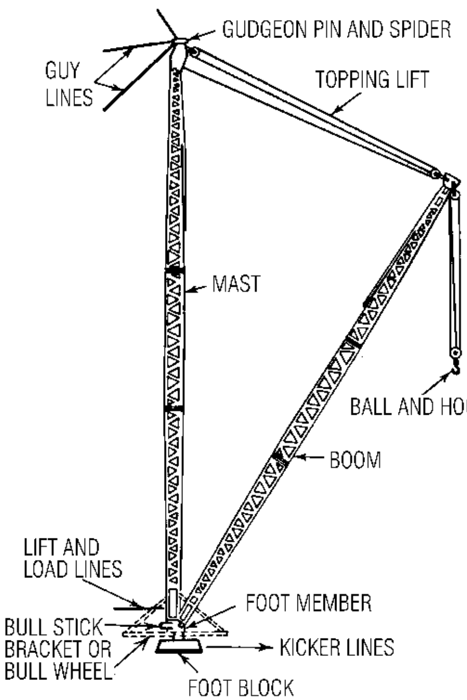

Fig. 9.22 Guy derrick.

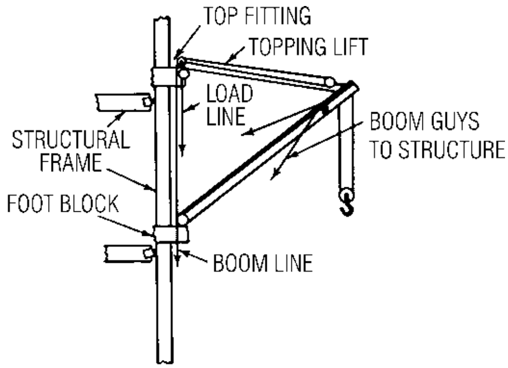

Fig. 9.23 Chicago boom.

Cranes may be mounted on a truck, crawler, or locomotive frame. The truck-mounted crane requires firm, level ground. It is useful on small jobs, where maneuverability and reach are required. Crawler cranes are more adaptable for use on soggy soil or where an irregular or pitched surface exists. Locomotive cranes are used for bridge erection or for jobs where railroad track exists or when it is economical to lay track.

The **tower crane** (Fig. 9.25) has important advantages. The control station can be located on

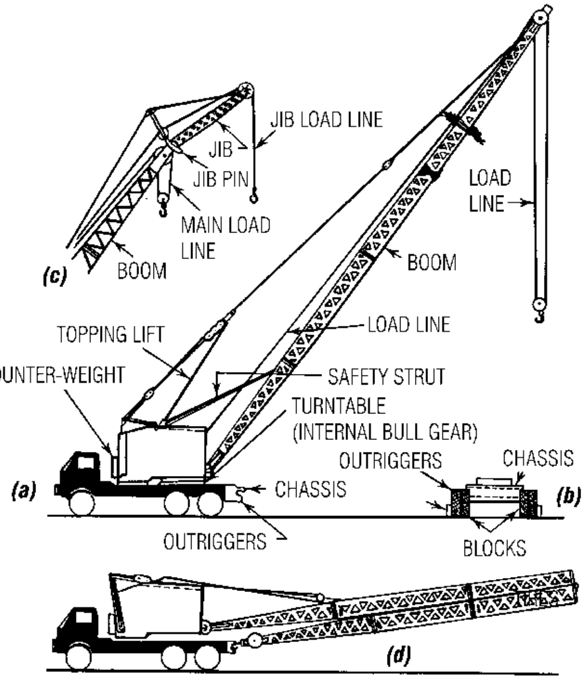

Fig. 9.24 Truck crane.

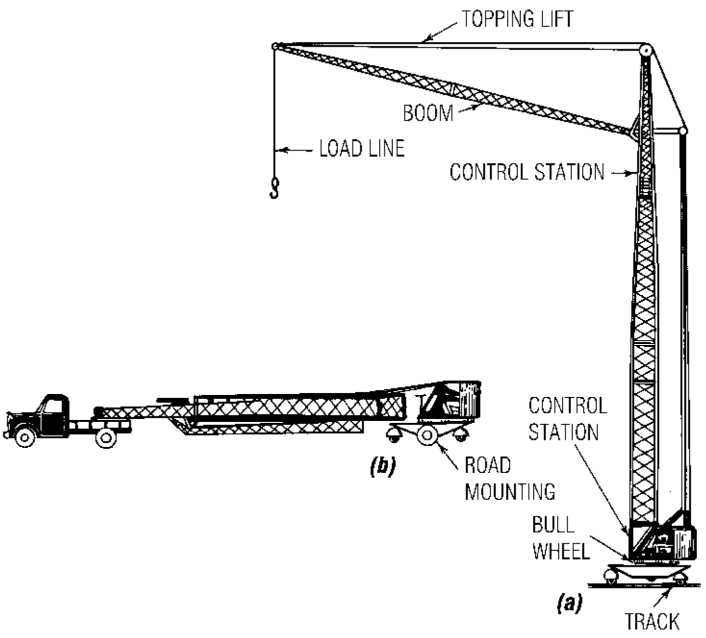

Fig. 9.25 Tower or slewing crane.

the crane or at a distant position that enables the operator to see the load at all times. Also, the equipment can be used to place concrete directly in the forms for floors and roofs, eliminating chutes, hoppers, and barrows.

Variations of the tower crane include the *kangaroo* (Fig. 9.26*a*) and the *hammerhead types* (Fig. 9.26*b*). The control station is located at the top of the tower and gives the operator a clear view of erection from above. A hydraulic jacking system is

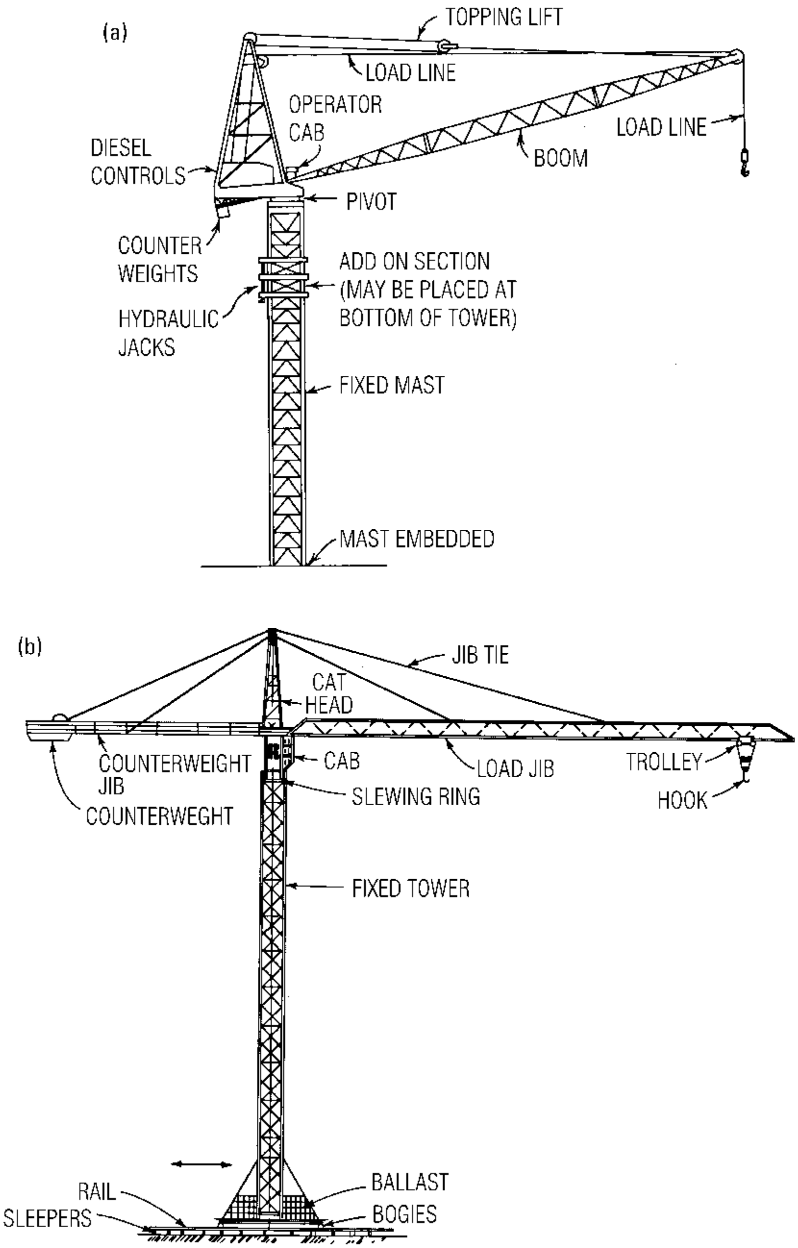

Fig. 9.26 Variations of the tower crane: (*a*) Kangaroo. (*b*) Hammerhead.

built into the fixed mast, and new mast sections are added to increase the height. As the tower gets higher, the mast must be tied into the structural framework for stability.

No general rules can be given regarding the choice of an erection device for a particular job. The main requirement is usually speed of erection, but other factors must be considered, such as the cost of the machine, labor, insurance, and cost of the power. Also, it is important to follow safety regulations set forth by the U.S. Office of Safety and Health Administration (OSHA).

9.33 Tolerances and Clearances for Erecting Beams

It is the duty of the structural-shop drafter to detail the steel so that each member may be swung into position without shifting members already in place.

Over the years, experience has resulted in "standard" practices in building work. The following are some examples:

In a framed connection, the total out-to-out distance of beam framing angles is usually ⅛ in shorter than the face-to-face distance between the columns or other members to which the beam will be connected. Once the beam is in place, it is an easy matter to bend the outstanding legs of the angle, if necessary, to complete the connection. With a relatively short beam, the drafter may determine that it is impossible to swing the beam into place with only the ⅛-in clearance. In such cases, it may be necessary to ship the connection angles "loose" for one end of the beam. Alternatively, it may be advantageous to connect one angle of each end connection to the supporting member and complete the connection after the beam is in place.

The common case of a beam framing into webs of columns must also be carefully considered. The usual practice is to place the beam in the "bosom" of the column by tilting it in the sling as shown in Fig. 9.27. It must, of course, clear any obstacle above. Also, the greatest diagonal distance *G* must be about ⅛ in less than the distance between column webs. After the beam is seated, the top angle may be attached.

It is standard detailing practice to compensate for anticipated mill variations. The limits for mill tolerances are prescribed in ASTM A6, "General Requirements for Delivery of Rolled Steel Plates, Shapes, Sheet Piling, and Bars for Structural Use." For example, wide-flange beams are considered straight, vertically or laterally, if they are within ⅛ in for each 10 ft of length. Similarly, columns are straight if the deviation is within ⅛ in/10 ft, with a maximum deviation of ⅜ in.

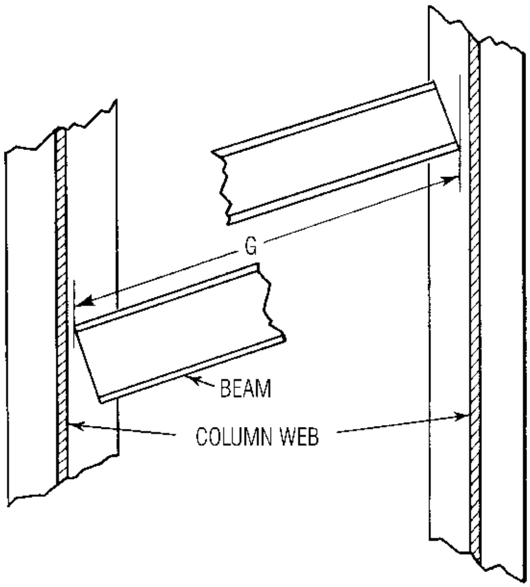

Fig. 9.27 Diagonal distance *G* for beam should be less than the clear distance between column webs, to provide erection clearance.

The "Code of Standard Practice" of the American Institute of Steel Construction gives permissible tolerances for the completed frame; Fig. 9.2 summarizes these. As shown, beams are considered level and aligned if the deviation does not exceed 1 : 500. With columns, the 1 : 500 limitation applies to individual pieces between splices. The total or cumulative displacement for multistory buildings is also given. The control is placed on exterior columns or those in elevator shafts.

There are no rules covering tolerances for milled ends of columns. It is seldom possible to achieve tight bearing over the cross section, and there is little reason for such a requirement. As the column receives its load, portions of the bearing area may quite possibly become plastic, which tends to redistribute stresses. Within practical limits, no harm is done to the load-carrying capacity of the member.

9.34 Fire Protection of Steel

Although structural steel does not support combustion and retains significant strength at elevated temperatures as subsequently discussed, the threat of sustained high-temperature fire, in certain types of construction and occupancies, requires that a steel frame be protected with fire-resistive materials.

In many buildings, no protection at all is required because they house little combustible material or they incorporate sprinkler systems. Therefore, "exposed" steel is often used for industrial-type buildings, hangars, auditoriums, stadiums, warehouses, parking garages, billboards, towers, and low stores, schools, and hospitals. Bridges require no fire protection.

The factors that determine fire-protection requirements, if any, are height, floor area, type of occupancy (a measure of combustible contents), availability of fire-fighting apparatus, sprinkler systems, and location in a community (fire zone), which is a measure of hazard to adjoining properties.

Fire Ratings ▪ Based on the above factors, building codes specify minimum fire-resistance requirements. The degree of fire resistance required for any structural component is expressed in terms of its ability to withstand fire exposure in accordance with the requirements of the ASTM standard time-temperature fire test, as shown in Fig. 9.28.

Under the standard fire-test ASTM Specification (E119), each tested assembly is subjected to the standard fire of controlled extent and severity. The fire-resistance rating is expressed as the time, in hours, that the assembly is able to withstand exposure to the standard fire before the criterion of failure is reached. These tests indicate the period of time during which the structural members, such as columns and beams, are capable of maintaining their strength and rigidity when subjected to the standard fire. They also establish the period of time during which floors, roofs, walls, or partitions will prevent fire spread by protecting against the passage of flame, hot gases, and excessive heat.

Strength Changes ▪ When evaluating fire-protection requirements for structural steel, it is useful to consider the effect of heat on its strength. In general, the yield point decreases linearly from its value at 70 °F to about 80% of that value at 800°F. At 1000 °F, the yield point is about 70% of its value at 70 °F and approaches the working stress of the structural members. Tension and compression members, therefore, are permitted to carry their maximum working stresses if the average temperature in the member does not exceed 1000 °F or the maximum at any one point does not exceed 1200 °F. (For steels other than carbon or low-alloy, other temperature limits may be necessary.)

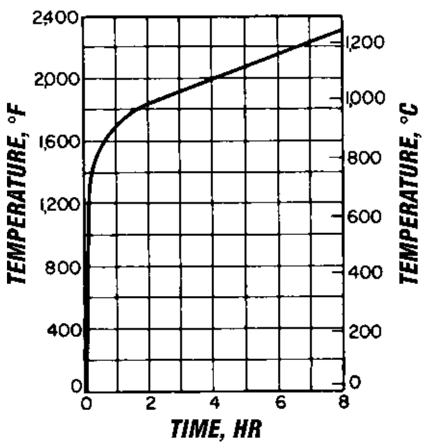

Fig. 9.28 ASTM time-temperature curve for fire test. Air temperature reaches 1000 °F in 5 min, 1700 °F in 1 h, and 2000 °F in 4 h.

Coefficient of Expansion ▪ The average coefficient of expansion for structural steel between temperatures of 100 and 1200 °F is given by the formula

$$c = (6.1 + 0.0019t) \times 10^{-6} \qquad (9.99)$$

where c = coefficient of expansion per °F

t = temperature, °F

Change in Modulus ▪ The modulus of elasticity is about 29,000 ksi at room temperature and decreases linearly to about 25,000 ksi at 900 °F. Above that, it decreases more rapidly.

Fire-Protection Methods ▪ Once the required rating has been established for a structural component, there are many ways in which the steel frame may be protected. For columns, one popular-fire-protection material is lightweight plaster (Fig. 9.29). Generally, a vermiculite or perlite

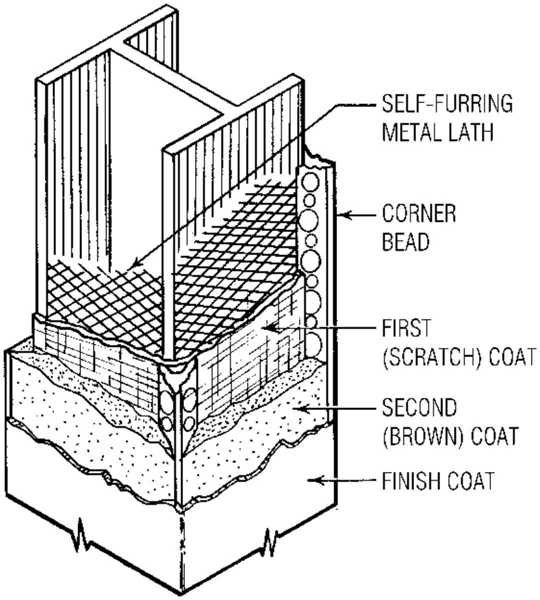

Fig. 9.29 Column fireproofing with plaster on metal lath.

plaster thickness of 1 to $1\frac{3}{4}$ in affords protection of 3 to 4 h, depending on construction details.

Concrete, brick, or tile is sometimes used on columns where rough usage is expected. Ordinarily, however, these materials are inefficient because of the large dead weight they add to the structure. Lightweight aggregates would, of course, reduce this inefficiency.

Beams, girders, and trusses may be fireproofed individually or by a membrane ceiling. Lath and plaster, sprayed mineral fibers, or concrete encasement may be used. As with columns, concrete adds considerably to the weight. The sprayed systems usually require some type of finish for architectural reasons.

The membrane ceiling is used quite often to fireproof the entire structural floor system, including beams, girders, and floor deck. For many buildings, a finished ceiling is required for architectural reasons. It is therefore logical and economical to employ the ceiling for fire protection also. Figure 9.30 illustrates typical installations. As can be seen, the rating depends on the thickness and type of material.

Two alternative methods of fire protection are flame shielding and water-filled columns. These methods are usually used together and are employed where the exposed steel frame is used architecturally.

Another method of fire protection is by separation from a probable source of heat. If a structural member is placed far enough from the source of heat, its temperature will not exceed the critical limit. Mathematical procedures for determining the temperature of such members are available. (See, for example, “Fire-Safe Structural Steel—A Design Guide,” American Iron and Steel Institute, 1001 17th St., Washington, D.C. 20036, www.aisc.org.)

Figure 9.31 illustrates the principle of flame shielding. The spandrel web is exposed on the exterior side and sprayed with fireproofing material on the inside. The shield in this case is the insulated bottom flange, and its extension protects the web from direct contact with the flame. The web is heated by radiation only and will achieve a maximum

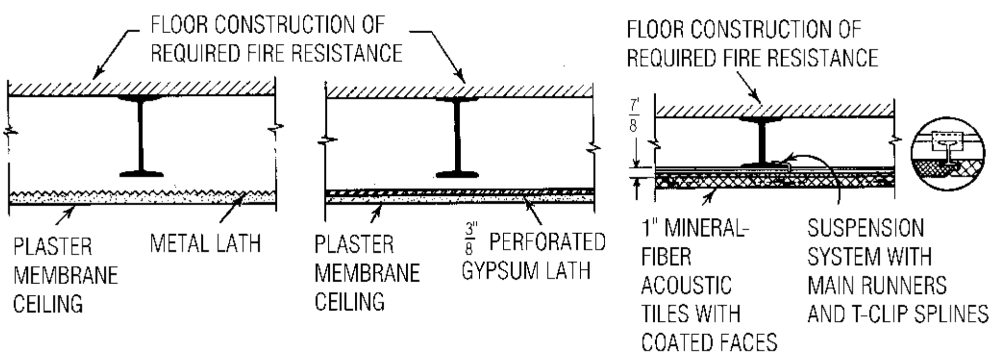

Fig. 9.30 Ceiling-membrane fireproofing applied below floor beams and girders.

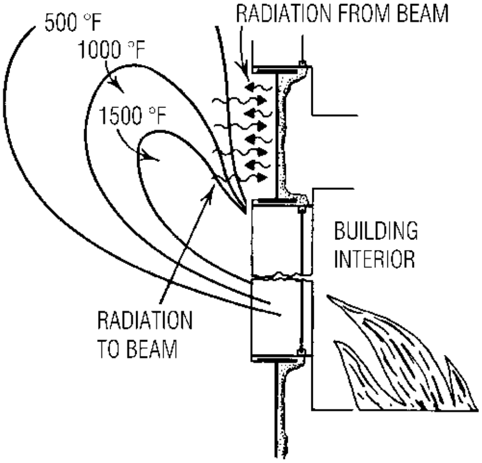

Fig. 9.31 Flame-shielded spandrel girder. (*From "Fire-Resistant Steel-Frame Construction," American Iron and Steel Institute, with permission.*)

temperature well below the critical temperature associated with structural failure.

Water-filled columns can be used with flame-shielded spandrels and are an effective fire-resistance system. The hollow columns are filled with water plus antifreeze (in northern climates). The water is stationary until the columns are exposed to fire. Once exposed, heat that penetrates the column walls is absorbed by the water. The heated water rises, causing water in the entire system to circulate. This takes heated water away from the fire and brings cooler water to the fire-affected columns (Fig. 9.32).

Another alternative in fire protection is intumescent paint. Applied by spray or trowel, this material has achieved a 1-h rating and is very close to a 2-h rating. When subjected to heat, it puffs up to form an insulating blanket. It can be processed in many colors and has an excellent architectural finish.

In building construction, it is often necessary to pierce the ceiling for electrical fixtures and air-conditioning ducts. Tests have provided data for the effect of these openings. The rule that has resulted is that ceilings should be continuous, except that openings for noncombustible pipes, ducts, and electrical outlets are permissible if they do not exceed 100 in^2 in each 100 ft^2 of ceiling

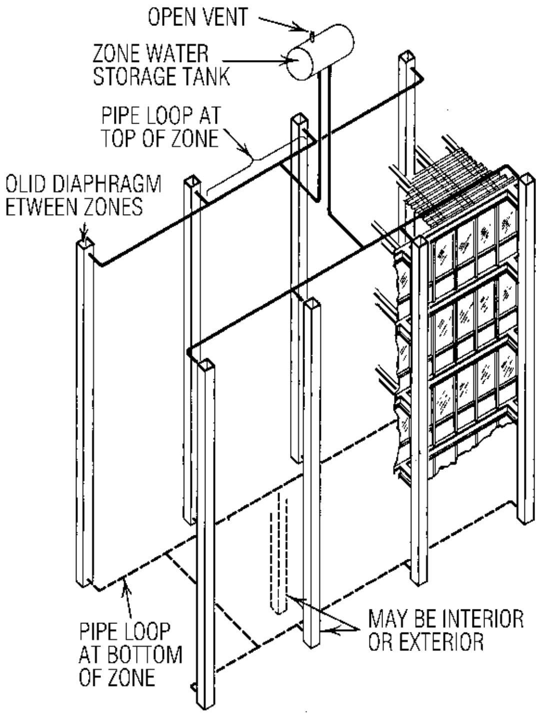

Fig. 9.32 Piping arrangement for liquid-filled-column fire-protection system. (*From "Fire-Resistant Steel-Frame Construction," American Iron and Steel Institute, with permission.*)

area. All duct openings must be protected with approved fusible-link dampers.

Summaries of established fire-resistance ratings are available from the following organizations:

American Insurance Association, 1130 Connecticut Ave NW, Washington, DC 20036.

National Institute of Standards and Technology, Washington, DC 20234

Gypsum Association, 810 First St., Washington, DC 20002.

Metal Lath/Steel Framing Association, 8 S. Michigan Ave, Chicago, IL 60603

Perlite Institute, 88 New Dorp Plaza, Staten Island, NY, 10306-2994

Vermiculite Association, Whitegate Acre, Metheringham, Fen, Lincoln, LN43AL, UK

American Iron and Steel Institute, 1140 Connecticut Ave., N.W., Washington, DC 20036

American Institute of Steel Construction, One East Wacker Dr., Chicago, IL 60601-2001

9.35 Corrosion Protection of Steel

The following discussion applies to all steels used in applications for which a coating is required for protection against atmospheric corrosion. As previously indicated (Art. 9.3), some high-strength, low-alloy steels can, with suitable precautions (including those in Art. 9.36), be used in the bare, uncoated condition for some applications in which a coating is otherwise required for protection against atmospheric corrosion.

Steel does not rust except when exposed to atmospheres above a critical relative humidity of about 70%. Serious corrosion occurs at normal temperature only in the presence of both oxygen and water, both of which must be replenished continually. In a hermetically sealed container, corrosion of steel will continue only until either the oxygen or water, or both, are exhausted.

To select a paint system for corrosion prevention, therefore, it is necessary to begin with the function of the structure, its environment, maintenance practices, and appearance requirements. For instance, painting steel that will be concealed by an interior building finish is usually not required. On the other hand, a bridge exposed to severe weather conditions would require a paint system specifically designed for that purpose.

The Society for Protective Coatings, SPC (Forty 24th St., Pittsburgh, PA 15222, www.sspc.org) issues specifications covering practical and economical methods of surface preparation and painting steel structures. The SPC also engages in research aimed at reducing or preventing steel corrosion. This material is published in two volumes: I, "Good Painting Practice," and II, "Systems and Specifications."

The SPC Specifications include numerous paint systems. By reference to a specific specification number, it is possible to designate an entire proved paint system, including a specific surface preparation, pretreatment, paint-application method, primer, and intermediate and top coat. Each specification includes a "scope" clause recommending the type of usage for which the system is intended.

In addition to the overall system specification, the SPC publishes individual specifications for surface preparation and paints. Surface preparations included are solvent, hand tool, power tool, pickling, flame, and several blast techniques.

When developing a paint system, it is extremely important to relate properly the type of paint to the surface preparation. For instance, a slow-drying paint containing oil and rust-inhibitive pigments and one possessing good wetting ability could be applied on steel nominally cleaned. On the other hand, a fast-drying paint with poor wetting characteristics requires exceptionally good surface cleaning, usually entailing complete removal of mill scale.

"Standard Specifications for Highway Bridges," (American Association of State Highway and Transportation Officials), gives detailed specifications and procedures for the various painting operations and for paint systems. AASHTO Specifications for surface preparation include hand cleaning, blast cleaning, and steam cleaning. Application procedures are given for brush, spray, or roller, as well as general requirements.

Concrete Protection ▪ In bridge and building construction, steel may be in contact with concrete. According to SPC vol. I, "Good Painting Practice":

1. Steel that is embedded in concrete for reinforcing should not be painted. Design considerations require strong bond between the reinforcing and the concrete so that the stress is distributed; painting of such steel does not supply sufficient bond. If the concrete is properly made and of sufficient thickness over the metal, the steel will not corrode.
2. Steel encased with exposed lightweight concrete that is porous should be painted with at least one coat of good-quality rust-inhibitive primer. When conditions are severe or humidity is high, two or more coats of paint should be applied since the concrete may accelerate corrosion.
3. When steel is enclosed in concrete of high density or low porosity and the concrete is at least 2 to 3 in thick, painting is not necessary since the concrete will protect the steel.

4. Steel in partial contact with concrete is generally not painted. This creates an undesirable condition, for water may seep into the crack between the steel and the concrete, causing corrosion. A sufficient volume of rust may build up, spalling the concrete. The only remedy is to chip or leave a groove in the concrete at the edge next to the steel and seal the crack with an alkali-resistant calking compound (such as bituminous cement).
5. Steel should not be encased in concrete that contains cinders since the acidic condition will cause corrosion of the steel.

9.36 Bolted Joints in Bare-Steel Structures

Special considerations are required for the design of joints in bare weathering steels. Atmospheric-corrosion-resistant, high-strength, low-alloy steels are used in the unpainted (bare) condition for such diverse applications as buildings, railroad hopper cars, bridges, light standards, transmission towers, plant structures, conveyor-belt systems, and hoppers because these steels are relatively inexpensive and require little maintenance. Under alternate wetting and drying conditions, a protective oxide coating that is resistant to further corrosion forms. But if such atmospheric-corrosion-resistant steels remain wet for prolonged periods, their corrosion resistance will not be any better than that of carbon steel. Thus, the design of the structure should minimize ledges, crevices, and other areas that can hold water or collect debris.

Experience with bolted joints in exposed frameworks of bare weathering steel indicates that if the stiffness of the joint is adequate and the joint is tight, the space between two faying surfaces of weathering-type steel seals itself with the formation of corrosion products around the periphery of the joint. However, if the joint design does not provide sufficient stiffness, continuing formation of corrosion products within the joint leads to expansive forces that can (1) deform the connected elements such as cover plates and (2) cause large tensile loads on the bolts.

Consequently, in the design of bolted joints in bare weathering steel, it is important to adhere to the following guidelines:

1. Limit pitch to 14 times the thickness of the thinnest part (7-in maximum).
2. Limit edge distance to 8 times the thickness of the thinnest part (5-in maximum).
3. Use fasteners such as ASTM A325, Type 3, installed in accordance with specifications approved by the Research Council on Structural Connections. (Nuts should also be of weathering steel; galvanized nuts may not provide adequate service if used with weathering steel.)