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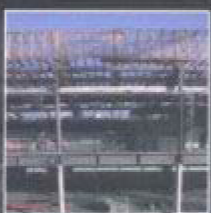
BUILDING DESIGN AND CONSTRUCTION HANDBOOK

SIXTH EDITION



STRUCTURAL STEEL CONSTRUCTION

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SECTION SEVEN
STRUCTURAL STEEL CONSTRUCTION

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Structural steel is an economical construction material for building applications. It offers high ratios of strength to weight and strength to volume. Thus, structural steel has the advantage of permitting long clear spans for horizontal members and requiring less floor space for columns than other common construction materials. It also can be used in combination with reinforced concrete to provide cost-effective building components. For large industrial buildings, where the structural frame can be exposed, it is often the material of choice.

The design of a structural building frame involves the following principal steps:

1. Select the general configuration and type of structure (Sec. 1).
2. Determine the service loads as required by the applicable building code (Art. 5.1.2).
3. Compute the internal forces and moments for the individual members (Sec. 5).
4. Proportion the members and connections.
5. Check performance characteristics, such as deflection, under service conditions.
6. Make a general overall review for economy of function.
7. Prepare complete design drawings delineating all structural steel requirements.

Designers, in addition to performing these steps, should also have an appreciation of the complete construction cycle to assure a practical and economical design. This includes understanding the needs of other disciplines and trades, types and availability of the materials used in steel of construction, applicable codes and specifications, the role and responsibilities of the fabricator and the erector, and a designer's own responsibilities in the area of quality assurance.

The other principal parties involved in structural steel construction are fabricators and erectors. Erectors frequently act as a subcontractor to the fabricator. Fabrication operations convert the mill materials into shipping pieces ready for erection at the jobsite. These operations are generally performed in a shop. The pieces are sized and shaped to the dimensions shown on detailed shop drawings that are prepared by the fabricator and approved by the structural designer. Shop attachment of detail pieces (stiffeners, connection materials, etc.) to the individual shipping pieces is most frequently done by welding. Generally, the fabricator is responsible for moving the fabricated material to the jobsite. The fabricator determines the size of shipping pieces, with the concurrence of the designer, at the time the shop drawings are prepared.

Erectors receive the material and the position and connect the steel into its final location at the project site. Erectors may have specific equipment on unique projects with which they are able to perform cost-effective operations. Such equipment may require attachment points or stiffening of the frame elements, in which case approval of the designer is requested.

Structural steel consists of hot-rolled steel shapes, steel plates of thickness of ⅛ in or greater, and such fittings as bolts, welds, bracing rods, and turnbuckles. The owner and the engineer should understand fully what will be furnished by the fabricator under a contract to furnish "structural steel." To promote uniformity in bidding practices, the American Institute of Steel Construction (AISC) has adopted a "Code of Standard Practice for Buildings and Bridges" (American Institute of Steel Construction, One East Wacker Drive, Suite 3100, Chicago, IL 60601-2001). Additional design guides are shown in Table 7.1.

7.1 CODES AND SPECIFICATIONS

Codes, specifications, and standards provide steel designers with sound design procedures and guidelines. These documents cover selection of service and design loads, criteria for proportioning members and their connections, procedures for fabrication and erection, requirements for inspections, and standards for protection against corrosion and fire. Use of these documents generally ensures safety, economical designs, and sound operational techniques.

The applicable building code defines the minimum legal requirements for a design. Most building authorities incorporate in their building code one of the model building codes (Art. 1.10), but some write their code requirements. Usually, the basis for the requirements for steel design and construction in building codes are the American Institute of Steel Construction specifications for structural steel buildings (Table 7.1). Note that two AISC specifications are available, one applicable to allowable stress design and plastic design (ASD) and the second to load and resistance factor design (LRFD).

Table 7.1 also lists other codes and specifications most frequently used by steel designers. Requirements for special-function buildings, needs of governmental agencies, and other unique requirements has led to promulgation of many other codes and specifications. Some of the organizations that publish these standards are the General Services Administration, U.S. Department of Commerce, Corps of Engineers, and U.S. Navy Bureau of Yards and Docks.

7.2 MILL MATERIALS

The steel shapes, plates, and bars that make up most of the materials used for structural steel are produced by mills as hot-rolled products. These products are made in a batch process; each production run of steel comes from a "heat." The

TABLE 7.1 Basic Steel Construction Codes and Specifications

Organization	Document	Scope
American Institute of Steel Construction (AISC) One East Wacker Drive Chicago, IL 60601-2001	Code of Standard Practice for Steel Buildings and Bridges	Defines structural steel Plans and specifications Fabrication Erection Quality control
	Specification for Structural Steel Buildings—Allowable Stress Design and Plastic Design (ASD)	Materials Loads Design criteria Serviceability Fabrication Erection Quality control
	Specifications for Structural Steel Buildings—Load and Resistance Factor Design (LRFD)	
American Iron and Steel Institute (AISI) 1101 17th St., N.W. Washington, DC 20036	Specification for the Design of Cold-Formed Steel Structural Members	Materials Design criteria
ASTM 100 Barr Harbor Drive West Conshohocken, PA 19428-2959	ASTM A6	Delivery-shapes/plates
	Various ASTM material specifications	Physical and chemical requirements
American Welding Society (AWS) 550 N.W. LeJeune Road Miami, FL 33126	Structural Welding Code—Steel (AWS D1.1)	Joint design Workmanship Procedures Inspection
Research Council on Structural Connections Engineering Foundation 345 E. 47th St. New York, NY 10017	Specifications for Structural Joints Using ASTM A325 or A490 Bolts	Materials Connection design Installation Inspection
Steel Joist Institute (SJI) 3127 10th Ave., North Ext. Myrtle Beach, SC 29577-6760	Standard Specifications and Load Tables, Open-Web Steel Joists	Materials Design
Steel Structures Painting Council (SSPC) 40 24th Street, Suite 600 Pittsburgh, PA 15213	Steel Structures Painting Manual, Vols. 1 and 2	Good practice Systems Specifications

specific grade of steel in all mill products is identified by reference to the heat number.

Through universal acceptance of ASTM specifications (Table 7.1), mill materials have uniform physical and quality characteristics. There is no significant metallurgical or physical difference between products ordered to a specific ASTM specification and rolled by any U.S. structural mill.

7.2.1 Grades of Steel

Structural steel grades are referred to by their corresponding ASTM designation. For example, the most commonly used grade of structural steel is A36, which is produced to meet the requirements of the ASTM A36 specification. This grade offers a good mix of strength, weldability, and cost. In many designs, this specification alone will satisfy designers' needs. Other specifications, such as A53 for pipe, provide an equivalent grade of steel for that type of product. However, as loads on the structural elements becomes larger, other grades of steel may become more economical because of dimensional limitations or simpler fabrication. These grades provide greater strength levels at somewhat higher costs per unit weight.

AISC recommends certain grades of steel, all of which have desirable characteristics, such as weldability and cost-effectiveness, for use where higher strength levels are required. The specifications covering these grades are listed in Table 7.2. Several steels have more than one level of tensile strength and yield stress, the

TABLE 7.2 Characteristics of Structural Steels

ASTM specification	Thickness, in	Minimum tensile strength, ksi	Minimum yield stress,* ksi
	Carbon Steels		
A36	To 8 in incl.	58–80†	36
A529	To ½ in incl.	60–85†	42
	High-strength, low-alloy steels		
A441	To ¾ incl.	70	50
	Over ¾ to 1½	67	46
	Over 1½ to 4 incl.	63	42
	Over 4 to 8 incl.	60	40
A572	Gr 42: to 4 incl.	60	42
	Gr 45: to 1½ incl.	60	45
	Gr 50: to 1½ incl.	65	50
	Gr 55: to 1½ incl.	70	55
	Gr 60: to 1 incl.	75	60
	Gr 65: to ½ incl.	80	65
A242	To ¾ incl.	70	50
	Over ¾ to 1½	67	46
	Over 1½ to 4 incl.	63	42
A588	To 4 incl.	70	50
	Over 4 to 5	67	46
	Over 5 to 8 incl.	63	42
A992	Shapes only	65	50
	Heat-treated low-alloy steels		
A514	To ¾ incl.	115–135	100
	Over ¾ to 2½	115–135	100
	Over 2½ to 4 incl.	105–135	90

* Yield stress or yield strength, whichever shows in the stress-strain curve.
† Minimum tensile strength may not exceed the higher value.

levels being dependent on thickness of material. The listed thicknesses are precise for plates and nearly correct for shapes. To obtain the precise value for shapes, refer to an AISC ''Manual of Steel Construction'' (ASD or LRFD) or to mill catalogs.

Weathering Steels. The A242 and A588 grades of steel offer enhanced corrosion resistance relative to A36 material. These steels, called weathering steels, form a thin oxidation film on the surfaces that inhibits further corrosion in ordinary atmospheric conditions. However, special treatment of construction details is required. Because of such constraints, and because these grades are more expensive, utilization of weathering steels in building construction is limited. These grades are more commonly used in bridge construction.

Steel Grade Identification. Because of the several grades of steel in use, ASTM specifications require that each piece of hot-rolled steel be properly identified with vital information, including the heat number. The AISC specifications for structural steel buildings require fabricators to be prepared to demonstrate, by written procedure and by actual practice, the visible identification of all main stress-carrying elements at least through shop assembly. Steel identification include ASTM designation, heat number (if required), and mill test reports when specifically ordered.

Availability. Because structural steel is produced in a batch process, the less commonly used shapes and the higher-strength grades are produced less frequently than commonly used A36 shapes. Furthermore, steel service centers stock the smaller A36 shapes. As a result, availability of steels can affect construction schedules. Consequently, steel designers should be aware of the impact of specifying less commonly used materials and shapes if the project has a tight schedule. Fabricator representatives can provide needed information.

7.2.2 Structural Shapes

Steel mills have a standard classification for the many products they make, one of which is *structural shapes* (*heavy*). By definition this classification takes in all shapes having at least one cross-sectional dimension of 3 in or more. Shapes of lesser size are classified as *structural shapes* (*light*) or, more specifically, bars.

Shapes are identified by their cross-sectional characteristics—angles, channels, beams, columns, tees, pipe, tubing, and piles. For convenience, structural shapes are simply identified by letter symbols as indicated in Table 7.3. The industry

TABLE 7.3 Symbols for Structural Shapes

Section	Symbol
Wide-flange shapes	W
Standard I shapes	S
Bearing-pile shapes	HP
Similar shapes that cannot be grouped in W, S, or HP	M
Structural tees cut from W, S, or M shapes	WT, ST, MT
American standard chemicals	C
All other channel shapes	MC
Angles	L

recommended standard (adopted 1970) for indicating a specific size of beam or column-type shape on designs, purchase orders, shop drawings, etc., specifies listing of symbol, depth, and weight, in that order. For example, W14 × 30 identifies a wide-flange shape with nominal depth of 14 in and weight of 30 lb/lin ft. The ×, read as "by," is merely a separation.

Each shape has its particular functional use, but the workhorse of building construction is the wide-flange W section. For all practical purposes, W shapes have parallel flange surfaces. The profile of a W shape of a given nominal depth and weight available from different producers is essentially the same, except for the size of fillets between web and flanges.

7.2.3 Tolerances for Structural Shapes and Plates

Mills are granted a tolerance because of variations peculiar to working of hot steel and wear of equipment. Limitations for such variations are established by ASTM specification A6.

Wide-flange beams or columns, for example, may vary in depth by as much as ½ in, i.e., ¼ in over and under the nominal depth. The designer should always keep this in mind. Fillers, shims, and extra weld metal installed during erection may not be desirable, but often they are the only practical solution to dimensional variations from nominal.

Cocked flanges on column members are particularly troublesome to the erector for it is not until the steel is erected in the field that the full extent of mill variations becomes evident. This is particularly true for a long series of spans or bays, where the accumulating effect of dimensional variation of many columns may require major adjustment. Fortunately, the average variation usually is negligible and nominal erection clearance allowed for by the fabricator will suffice.

Mill tolerances also apply to beams ordered from the mills cut to length. Where close tolerance may be desired, as sometimes required for welded connections, it may be necessary to order the beams long and then finish the ends in the fabricating shop to precise dimensions. This is primarily the concern of structural detailers.

7.2.4 Cambered Beams

Frequently, designers want long-span beams slightly arched (cambered) to offset deflection under load and to prevent a flat or saggy appearance. Such beams may be procured from the mills, the required camber being applied to cold steel. The AISC Manuals give the maximum cambers that mills can obtain and their prediction of the minimum cambers likely to remain permanent. Smaller cambers than these minimums may be specified, but their permanency cannot be guaranteed. Nearly all beams will have some camber as permitted by the tolerance for straightness, and advantage may be taken of such camber in shop fabrication.

A method of cambering, not dependent on mill facilities, is to employ heat. In welded construction, it is commonplace to flame-straighten members that have become distorted. By the same procedure, it is possible to distort or camber a beam to desired dimensions.

7.2.5 Steel Plates

Used by fabricators to manufacture built-up structural members, such as columns and girders, and for detail connection material, plates are identified by the symbol

PL. Cross-sectional dimensions are given in inches (or millimeters). A plate ½ in thick and 2 ft wide is billed as PL ½ × 24. Plates may also be specified by weight, although this is unusual in building construction work.

Mill tolerances for plate products for structural applications are also defined by ASTM specification A6. There are provisions for thickness, crown, camber, and length. Consideration of these characteristics are primarily the responsibility of fabricators. However, steel designers should be aware of how these tolerances affect the fabricator's work and permit the design to accommodate these characteristics.

7.2.6 Pipe and Tubular Sections

Pipe meeting the requirements of ASTM specification A53, Types E and S, Grade B, is comparable to A36 steel, with yield strength $F_y = 36$ ksi. It comes in three weight classification: standard, extra strong, and double extra strong, and in diameters ranging up to 26 in.

Several mills produce square and rectangular tubing, known as *hollow structural sections,* in sizes from 3 × 2 and 2 × 2 to 12 × 8 and 10 × 10 in, with wall thickness up to ⅝ in. These flat-sided shapes afford easier connections than pipes, not only for connecting beams but also for such items as window and door frames.

The main strength properties of several grades of steel used for pipe and tubular sections are summarized in Table 7.4.

Cautionary Note. Hollow structural sections are not produced to meet the requirements of ASTM specification A6. Because of this characteristic, the AISC and the Steel Tube Institute of North America recommended that the nominal wall thickness of such sections be reduced by 7% when calculating the section properties of these sections, (area, section modulus, and moment of inertia) so as to maintain a factor of safety equivalent to that present in other structural steel shapes.

TABLE 7.4 Characteristics of Pipe and Tubular Steels

ASTM spec.	Grade	Product	Min tensile strength, ksi	Min yield stress, ksi
A53	B	Pipe	60.0	35.0*
A500	A	Round	45.0	33.0
	A	Shaped	45.0	39.0
	B	Round	58.0	42.0
	B	Shaped	58.0	46.0
	C	Shaped	70.0	50.0
A501	. . .	All tubing	58.0	36.0
A618	I	All tubing	70.0	50.0
	II	All tubing	70.0	50.0
	III	All tubing	65.0	50.0

*Use 36.0 for purpose of design.

7.3 FASTENERS

Two basic types of fasteners are typically used in construction, bolts and welds. Both are used in the fabricating shop and on the job site in connections joining individual members. Welds are also used to fasten together components of built-up members. Bolts, however, are more commonly used for field connections, and welds, for shop work. Rivets, which were once widely used for main connections, both shop and field, are essentially obsolete.

Many variables affect selection of fasteners. Included among these are economy of fabrication and erection, availability of equipment, inspection criteria, labor supply, and such design considerations as fatigue, size and type of connections, continuity of framing, reuse, and maintenance. It is not uncommon for steel framing to be connected with such combinations as shop welds and field bolts or to be all-welded. It is usual to use field welds for column splices with bolted connections elsewhere. The variables affecting decisions on use of fasteners should be explored with engineers representing the fabricator and the erector.

7.3.1 High-Strength Bolts

Development of high-strength bolts is vested in the Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation. Its "Specification for Structural Steel Joints Using A325 or A490 Bolts" (Table 7.1) was adopted by the American Institute for Steel Construction. Bolts conforming to ASTM A449 are acceptable, but their usage is restricted to bearing-type connections (Fig. 7.1) re-

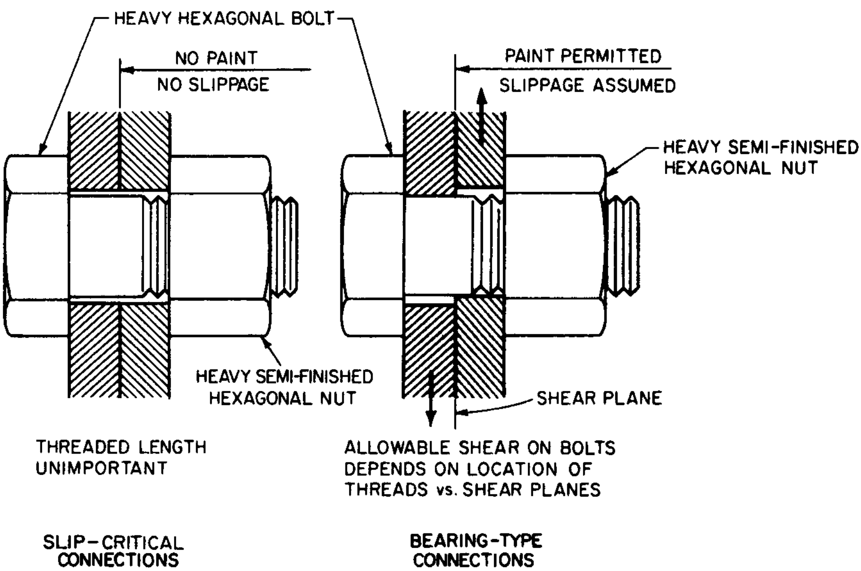


FIGURE 7.1 Two main types of construction with high-strength bolts. Although, in general, no paint is permitted on faying surfaces in slip-critical connections, the following are allowed: scored galvanized coatings, inorganic zinc-rich paint, and metallized zinc or aluminum coatings.

quiring bolt diameters greater than 1½ in. Furthermore, when they are required to be tightened to more than 50% of their specified minimum tensile strength, hardened steel washers should be installed under the heads.

When high-strength bolts are used in a connection, they are highly tensioned by tightening of the nuts and thus tightly clamp together the parts of the connection.

For convenient computation of load capacity, the clamping force and resulting friction are resolved as shear. Bearing between the bolt body and connected material is not a factor until loads become large enough to cause slippage between the parts of the connection. The bolts are assumed to function in shear following joint slippage into full bearing.

The clamping and bearing actions lead to the dual concept: slip-critical connections and bearing-type connections. For the latter, the allowable shear depends on the cross-sectional bolt area at the shear plane. Hence, two shear values are assigned, one for the full body area and one for the reduced area at the threads.

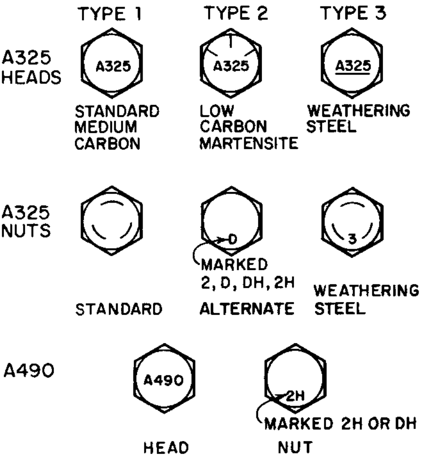


FIGURE 7.2 Identification markings on heads and nuts of high-strength bolts.

Identification. There is no difference in appearance of high-strength bolts intended for either slip-critical or bearing-type connections. To aid installers and inspectors in identifying the several available grades of steel, bolts and nuts are manufactured with permanent markings (Fig. 7.2).

7.3.2 High-Strength Bolt Installation

Washer requirements for high-strength bolted assemblies depend on the method of installation and type of bolt holes in the connected elements. These requirements are summarized in Table 7.5.

Bolt Tightening. Specifications require that all high-strength bolts be tightened to 70% of their specified minimum tensile strength, which is nearly equal to the proof load (specified lower bound to the proportional limit) for A325 bolts, and within 10% of the proof load for A490 bolts. Tightening above these minimum tensile values does not damage the bolts, but it is prudent to avoid excessive uncontrolled tightening. The required minimum tension, kips, for A325 and A490 bolts is given in Table 7.6.

There are three methods for tightening bolts to assure the prescribed tensioning:

Turn-of-Nut. By means of a manual or powered wrench, the head or nut is turned from an initial snug-tight position. The amount of rotation, varying from one-third to a full turn, depends on the ratio of bolt length (underside of heat to end of point) to bolt diameter and on the disposition of the outer surfaces of bolted parts (normal or sloped not more than 1:20 with respect to the bolt axis). Required rotations are

TABLE 7.5 Washer Requirements for High-Strength Bolts

Method of tensioning	A325 bolts	A490 bolts: Base material $F_y < 40.0$*	A490 bolts: Base material $F_y > 40.0$*
Calibrated wrench	One washer under turned element	Two washers	One washer under turned element
Turn-of-the-nut	None	Two washers	One washer under turned element
Both methods, slotted and oversized holes	Two washers	Two washers	Two washers

*F_y = specified minimum yield stress, ksi.

TABLE 7.6 Minimum Tightening Tension, kips, for High-Strength Bolts

Dia, in	A325	A490
⅝	19	24
¾	28	35
⅞	39	49
1	51	64
1⅛	56	80
1¼	71	102
1⅜	85	121
1½	103	148

tabulated in the "Specification for Structural Steel Joints Using A325 of A490 Bolts."

Calibrated Wrench. By means of a powered wrench with automatic cutoff and calibration on the job. Control and test are accomplished with a hydraulic device equipped with a gage that registers the tensile stress developed.

Direct Tension Indicator. Special indicators are permitted on satisfactory demonstration of performance. One example is a hardened steel washer with protrusions on one face. The flattening that occurs on bolt tightening is measured and correlated with the induced tension.

7.3.3 Unfinished Bolts

Known in construction circles by several names—ordinary, common, machine, or rough—unfinished bolts are characterized chiefly by the rough appearance of the shank. They are covered by ASTM A307. They fit into holes 1⁄16 in larger in diameter than the nominal bolt diameter.

Unfinished bolts have relatively low load-carrying capacity. This results from the possibility that threads might lie in shear planes. Thus, it is unnecessary to extend the bolt body by use of washers.

One advantage of unfinished bolts is the ease of making a connection; only a wrench is required. On large jobs, however, erectors find they can tighten bolts more economically with a pneumatic-powered impact wrench. Power tightening generally yields greater uniformity of tension in the bolts and makes for a better-balanced connection.

While some old building codes restrict unfinished bolts to minor applications, such as small, secondary (or intermediate) beams in floor panels and in certain parts of one-story, shed-type buildings, the AISC specifications for structural steel buildings, with a basis of many years of experience, permit A307 bolts for main connections on structures of substantial size. For example, these bolts may be used for beam and girder connections to columns in buildings up to 125 ft in height.

There is an economic relation between the strength of a fastener and that of the base material. So while A307 may be economical for connecting steel with a 36-ksi yield point, this type of bolt may not be economical with 50-ksi yield-point steel. The number of fasteners to develop the latter becomes excessive and perhaps impractical due to size of detail material.

A307 bolts should always be considered for use, even in an otherwise all-welded building, for minimum-type connections, such as for purlins, girts, and struts.

Locking Devices for Bolts. Unfinished bolts (ASTM A307) and interference-body-type bolts (Art 7.3.4) usually come with American Standard threads and nuts. Properly tightened, connections with these bolts give satisfactory service under static loads. But when the connections are subjected to vibration or heavy dynamic loads, a locking device is desirable to prevent the nut from loosening.

Locking devices may be classified according to the method employed: special threads, special nuts, special washers, and what may be described as field methods. Instead of conventional threads, bolt may be supplied with a patented self-locking thread called Dardelet. Sometimes, locking features are built into the nuts. Patented devices, the Automatic-Nut, Union-Nut, and Pal-Nut, are among the common ones. Washers may be split rings or specially touched. Field methods generally used include checking, or distorting, the threads by jamming them with a chisel or locking by tack welding the nuts.

7.3.4 Other Bolt-Type Fasteners

Interference body of bearing-type bolts are characterized by a ribbed or interrupted-ribbed shank and a button-shaped head; otherwise, including strength, they are similar to the regular A325 high-strength bolts. The extreme diameter of the shank is slightly larger than the diameter of the bolt hole. Consequently, the tips of the ribs or knurlings will groove the side of the hole, assuring a tight fit. One useful application has been in high television towers, where minimum-slippage joints are desired with no more installation effort than manual tightening with a spud wrench. Nuts may be secured with lock washers, self-locking nuts, or Dardelet self-locking threads. The main disadvantage of interference body bolts is the need for accurate matching of truly concentric holes in the members being joined; reaming sometimes is necessary.

Huckbolts are grooved (not threaded) and have an extension on the end of the shank. When the bolt is in the hole, a hydraulic machine, similar to a bolting or riveting gun, engages the extension. The machine pulls on the bolt to develop a high clamping force, then swages a collar into the grooved shank and snaps off the extension, all in one quick operation.

7.3.5 Welds

Welding is used to fasten together components of a built-up member, such as a plate girder, and to make connections between members. This technique, which uses fusion is a controlled atmosphere, requires more highly skilled labor than does bolting. However, because of cost advantages, welding is widely used in steel construction, especially in fabricating shops where conditions are more favorable to closely controlled procedures. When field welding is specified, the availability of skilled welders and inspection technicians and the use of more stringent quality-control criteria should be considered.

Any of several welding processes may be used: manual shielded metal arc, submerged arc, flux cored arc, gas metal arc, electrogas, and electroslag. They are not all interchangeable, however; each has its advantageous applications.

Many building codes accept the recommendations of the American Welding Society ‘‘Structural Welding Code’’ (AWS D1.1) (Table 7.1). The AISC specification incorporates many of this code's salient requirements.

Weld Types. Practically all welds used for connecting structural steel are of either of two types: fillet or groove.

Figure 7.3*a* and *b* illustrates a typical **fillet weld.** As stated in Art. 7.27, all stresses on fillet welds are resolved as shear on the effective throat. The normal throat dimension, as indicated in Fig. 7.3*a* and *b*, is the effective throat for all welding processes, except the submerged-arc method. The deep penetration characteristic of the latter process is recognized by increasing the effective throat dimension, as shown in Fig. 7.3*c*.

Groove welds (Fig. 7.3*d, e,* and *f*) are classified in accordance with depth of solid weld metal as either complete or partial penetration. Most groove welds, such as those in Fig. 7.3*d* and *e*, are made complete-penetration welds by the workmanship requirements: use backup strips or remove slag inclusions and imperfections (step called back-gouging) on the unshielded side of the root weld. The partial-penetration groove weld shown in Fig. 7.3*f* is typical of the type of weld used for box-type members and column splices. Effective throat depends on the welding process, welding position, and the chamfer angle α. The indicated effective throat (Fig. 7.3*f*) is proper for the shielded-metal-arc processes and for all welding positions. (See also Art. 7.27.)

Welding Electrodes. Specifications for all welding electrodes, promulgated by the American Welding Society (AWS), are identified as A5.1, A5.5, A5.17, etc., depending on the welding process. Electrodes for manual arc welding, often called stick electrodes, are designated by the letter E followed by four of five digits. The first two or three digits designate the strength level; thus, E70XX means electrodes having a minimum tensile strength of 70.0 ksi. Allowable shear stress on the depositied weld metal is taken as 0.30 times the electrode strength classification; thus, 0.30 times 70 to an E70 results in an allowable stress of 21.0 ksi. The remaining digits provide information on the intended usage, such as the particular welding positions and types of electrode coating.

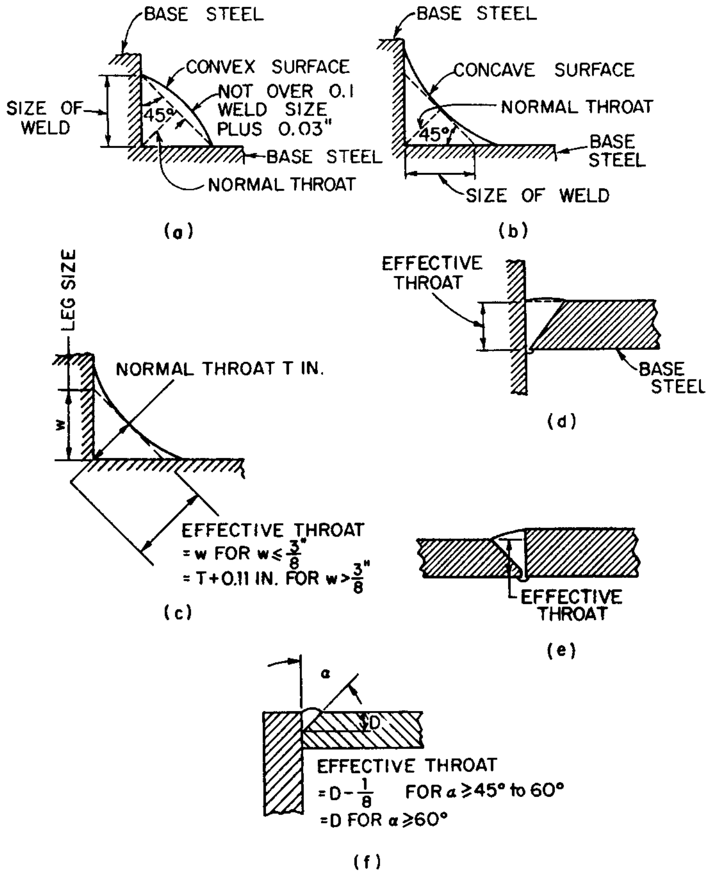


FIGURE 7.3 Effective throats of fillet and groove welds.

Welding Procedures. The variables that affect the quality of a weld are controlled by welding procedures that must be approved by the structural engineer. Specification AWS D1.1 contains several prequalified welding procedures, the use of which permits fabricators and erectors to avoid the need for obtaining approvals for specific routine work. Where unusual conditions exist, the specification requires that formal documentation be submitted for review and approval.

Base-Metal Temperatures. An important requirement in production of quality welds is the temperature of base metal. Minimum preheat and interpass temperature as specified by the AWS and AISC standards must be obtained within 3 inches of the welded joint before welding starts and then maintained until completion. Table 7.7 gives the temperature requirements based on thickness (thickest part of joint) and welding process for several structural steels. When base metal temperature is below 32°F, it must be preheated to at least 70° and maintained at that temperature during welding. No welding is permitted when ambient temperature is below 0°F.

TABLE 7.7 Minimum Preheat and Interpass Temperatures for Base Metal to Be Welded

Steel*	Shielded-metal-arc welding with other than low-hydrogen electrodes		Shielded-metal-arc welding with low-hydrogen electrodes, gas-metal-arc, and flux-cored arc welding	
	Thickness, in	Temp, °F	Thickness, in	Temp, °F
A36	To ¾ in incl.	32	To ¾ in incl.	32
	Over ¾ to 1½	150	Over ¾ to 1½	50
	Over 1½ to 2½	225	Over 1½ to 2½	150
	Over 2½	300	Over 2½	225
A242	Not permitted		To ¾ in incl.	32
A441			Over ¾ to 1½	70
A588			Over 1½ to 2½	150
A572 to $F_y = 50$			Over 2½	225
A529				

*For temperatures for other steels, see AWS D1.1, "Structural Welding Code," American Welding Society.

Additional information, including temperature requirements for other structural steels, is given AWS D1.1 and the AISC specifications for structural steel buildings (Table 7.1).

Another quality-oriented requirement applicable to fillet welds is minimum leg size, depending on thickness of steel (Table 7.8). The thicker part connected governs, except that the weld size need not exceed the thickness of the thinner part. This rule is intended to minimize the effects of restraint resulting from rapid cooling due to disproportionate mass relationships.

TABLE 7.8 Minimum Sizes* of Fillet and Partial-Penetration Welds

Base-metal thickness, in	Weld size, in
To ¼ incl.	⅛
Over ¼ to ½	1/16
Over ½ to ¾	¼
Over ¾ to 1½	5/16
Over 1½ to 2½	⅛
Over 2½ to 6	½
Over 6	⅝

*Leg dimension for fillet welds; minimum effective throat for partial-penetration groove welds.

7.3.6 Inspection of Welds

The quality of welded work is highly dependent upon the close adherence to applicable welding process and procedural requirements. This, plus attention to dimensional requirements, will generally result in serviceable welds. As a result, most welding work incorporated in building construction, other than for major structures, is inspected using visual inspection techniques. The fabricator's quality personnel are responsible for adherence to approved procedures. The owner's inspector observed the erector's operations and may perform any necessary visual inspection of the finished work.

Four nondestructive testing methods are commonly used to evaluate welded work. These are (1) magnetic-particle inspection, (2) liquid penetrant inspection, (3) radiographic inspection, and (4) ultrasonic inspection. The latter two methods are the most common today. Each of these nondestructive testing methods add to the cost of construction and should be used where some special service requirement justifies this added feature. Any such testing must be identified on the drawings or in the specifications.

7.3.7 Fastener Symbols

Fasteners are indicated on design, shop, and field erection drawings by notes and symbols. A simple note may suffice for bolts; for example: "⅞-in A325 bolts, except as noted." Welds require more explicit information, since their location is not so obvious as that of holes for bolts.

Symbols are standard throughout the industry. Figure 7.4 shows the symbols for bolts, Fig. 7.5 the symbols for welds. The welding symbols (Fig. 7.5*a*) together with the information key (Fig. 7.5*b*) are from the American Welding Society "Symbols for Welding and Nondestructive Testing, AWS A2.4.

7.3.8 Erection Clearance for Fasteners

All types of fasteners require clearances for proper installation in both shop and field. Shop connections seldom are a problem, since each member can be easily manipulated for access. Field connections, however, require careful planning, because connections can be made only after all members to be connected are aligned

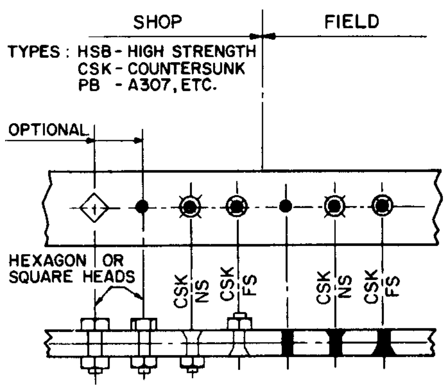
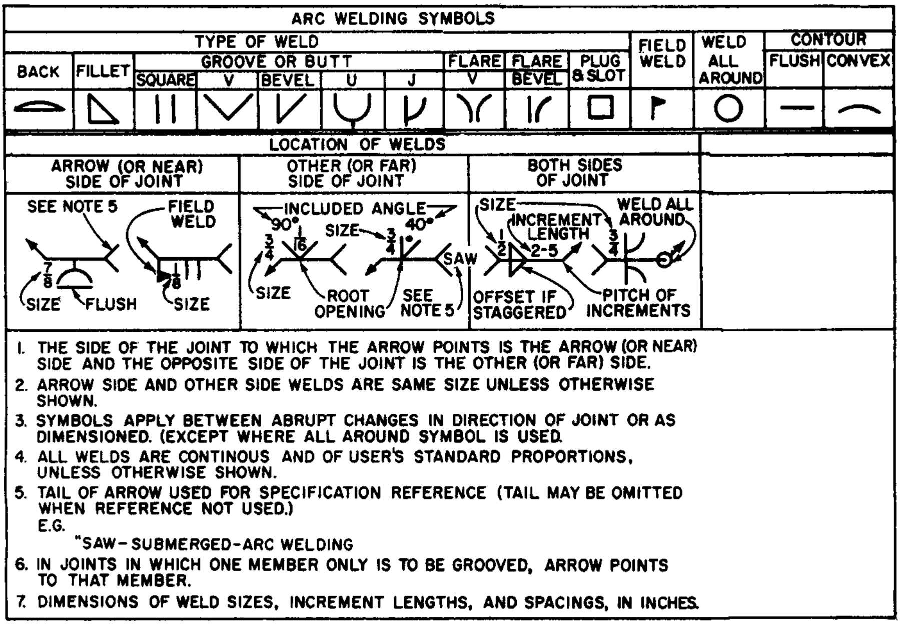
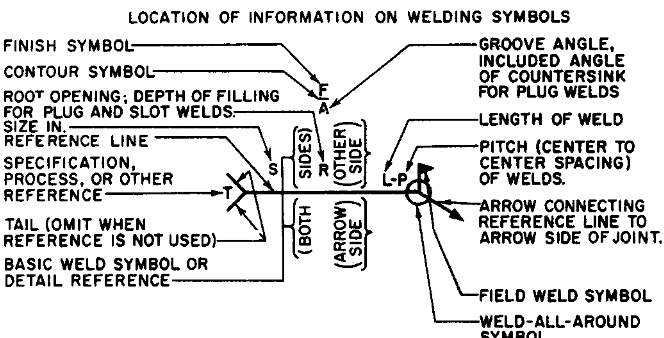


FIGURE 7.4 Symbols for shop and field bolts.



(a)



(b)

FIGURE 7.5 Symbols for shop and field welds.

in final position. This is the responsibility of the fabricator's engineering staff and is discharged during the making of shop drawings. However, the basic design configuration must permit the necessary clearances to be developed.

Clearances are required for two reasons: to permit entry, as in the case of bolts entering holes, and to provide access to the connected elements either to allow the tightening of bolts with field tools or to permit the movement of manual electrodes or semiautomatic welding tools in depositing weld metal.

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

7.4 FABRICATION

When considering fabrication, as well as erection of the fabricated product, the designer must taken into account contractual matters, work by others on the construction team, schedule implications of the design, and quality assurance matters. Fortunately, there are well established aids for these considerations. Contractual questions such as what constitutes structural steel, procedures for preparing and approving the shop detail drawings, and standard fabrication procedures and tolerances are all addressed in the AISC's Code of Standard Practice (Table 7.1). Insights on economical connection details and the impact of material selection on mill material deliveries are generally available from the fabricator's engineering staff. These engineers are also able to comment on unique erection questions.

Quality assurance questions fall into two categories, fabrication operations and field operations. Today, sound quality control procedures are in place in most fabrication shops through an AISC program which prequalifies fabricators. There are three levels of qualification: I, II and III, with Level III being the most demanding. Fabricators with either a Level I or Level II certification are suitable for almost all building work.

Most engineers incorporate the AISC's Code of Standard Practice in their project specification.

7.4.1 Shop Detail Drawings

Detail drawings are prepared by the fabricator to delineate to his work force the fabrication requirements. Because each shop has certain differences in equipment and/or procedures, the fabricator develops details which, when matched with his processes, are the most economical. To accomplish this end, the design drawings need to be complete, showing all structural steel requirements, and should include design information on the forces acting at connections. Designers should avoid specifying deck openings and beam penetrations through notes on the drawings. This is a frequent cause of extra costs on fabrication contracts.

7.4.2 Fabrication Processes

Mill material is cut to length by sawing, shearing, or flame cutting. Columns may also be milled to their final length. Holes for fasteners are drilled or punched. Punched and reamed holes are seldom used in building construction. Cuts for weld preparation, web openings, and dimensional clearances are flame cut. AISC guidelines for each of these processes are associated with the AISC's fabricator prequalification program. Welding for building construction is performed in accordance with the provisions of the AWS Structural Welding Code, D1.1. Most requirements can be satisfied using pre-qualified welding procedures.

7.5 QUALITY ASSURANCE

Concepts for improving and maintaining quality in the constructed project stress the participation of the design professional in the project team consisting of the owner, design professional, and general contractor. While the structural engineer plays a varying role in the major phases of a project—that is, conceptual, preliminary, and final design; bidding; and construction—his or her participation is vital to achieving the appropriate level of quality.

Those activities of the structural engineer that have the greatest impact on quality are materials selection, determination of workmanship quality levels, quality control (QC) requirements, preparation of clear and complete contract documents, and review of the contractor's work. One aspect of the last item that is particularly important in steel construction is the review and approval of the fabricator's shop drawings. Because the fabricator's engineers design connections to meet the criteria provided by the design professional, the review and approval process must assure that connection designs and details are compatible with the intent and requirements of the basic design.

(''Quality in the Constructed Project,'' American Society of Civil Engineers.)

STRUCTURAL FRAMING SYSTEMS

Steel construction may be classified into three board categories: wall-bearing, skeleton, and long-span framing. Depending on the needs of the building, one or more of these categories may be incorporated.

In addition to the main building elements—floors, roofs, walls—the structural system must include bracing members that provide lateral support for main members as well as for other bracing members, resistance to lateral loads on the building, redundant load paths, and stiffness to the structure limit deflections. An economical and safe design properly integrates these systems into a completed structure.

7.6 WALL-BEARING FRAMING

Probably the oldest and commonest type of framing, wall-bearing (not to be confused with bearing-wall construction), occurs whenever a wall of a building, interior or exterior, is used to support ends of main structural elements carrying roof or floor loads. The walls must be strong enough to carry the reaction from the supported members and thick enough to ensure stability against any horizontal forces that may be imposed. Such construction often is limited to relatively low structures, because load-bearing walls become massive in tall structures. Nevertheless, a wall-bearing system may be advantageous for tall buildings when designed with reinforcing steel.

A common application of wall-bearing construction may be found in many single-family homes. A steel beam, usually 8 or 10 in deep, is used to carry the interior walls and floor loads across the basement with no intermediate supports, the ends of the beam being supported on the foundation walls. The relatively shallow beam depth affords maximum headroom for the span. In some cases, the spans may be so large that an intermediate support becomes necessary to minimize deflection. Usually a steel pipe column serves this purpose.

Another example of wall-bearing framing is the member used to support masonry over windows, doors, and other openings in a wall. Such members, called **lintels,** may be a steel angle section (commonly used for brick walls in residences)

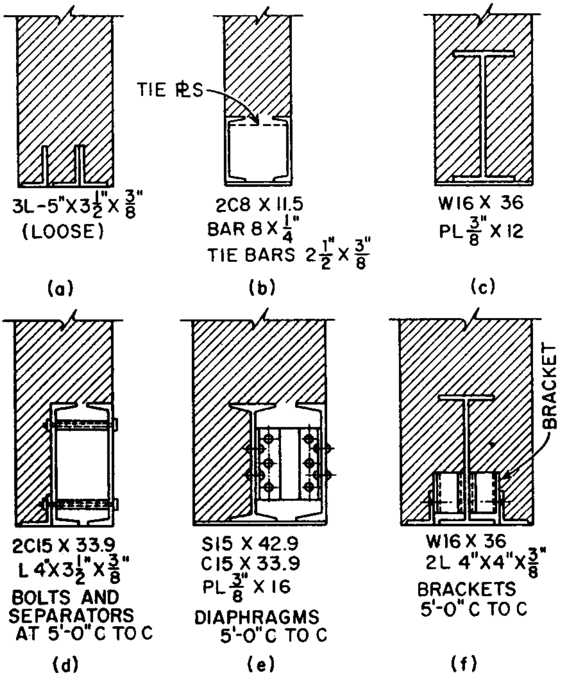


FIGURE 7.6 Lintels supporting masonry.

or, on longer spans and for heavier walls, a fabricated assembly. A variety of frequently used types is shown in Fig. 7.6. In types *b*, *c*, and *e*, a continuous plate is used to close the bottom, or soffit, of the lintel, and to join the load-carrying beams and channels into a single shipping unit. The gap between the toes of the channel flanges in type *d* may be covered by a door frame or window trim, to be installed later. Pipe and bolt separators are used to hold the two channels together to form a single member for handling.

Bearing Plates. Because of low allowable pressures on masonry, bearing plates (sometimes called masonry plates) are usually required under the ends of all beams that rest on masonry walls, as illustrated in Fig. 7.7. Even when the pressure on the wall under a member is such that an area no greater than the contact portion of the member itself is required, wall plates are sometimes prescribed, if the member is of such weight that it must be set by the steel erector. The plates, shipped loose and in advance of steel erection, are then set by the mason to provide a satisfactory seat at the proper elevation.

Anchors. The beams are usually anchored to the masonry. **Government anchors,** as illustrated in Fig. 7.7, are generally preferred.

Nonresidential Uses. Another common application for the wall-bearing system is in one-story commercial and light industrial-type construction. The masonry side walls support the roof system, which may be rolled beams, open-web joists, or light

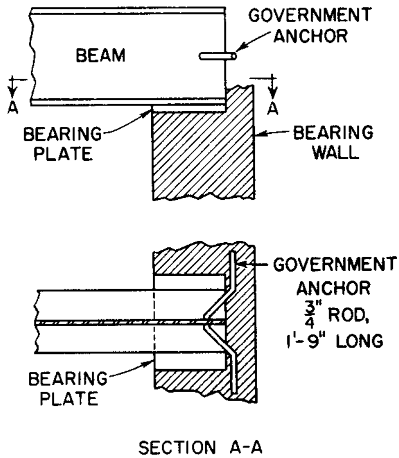


FIGURE 7.7 Wall-bearing beam.

trusses. Clear spans of moderate size are usually economical, but for longer spans (probably over 40 ft), wall thickness and size of buttresses (pilasters) must be built to certain specified minimum proportions commensurate with the span—a requirement of building codes to assure stability. Therefore, the economical aspect should be carefully investigated. It may cost less to introduce steel columns and keep wall size to the minimum permissable. On the other hand, it may be feasible to reduce the span by introducing intermediate columns and still retain the wall-bearing system for the outer end reactions.

Planning for Erection. One disadvantage of wall-bearing construction needs emphasizing: Before steel can be set by the ironworkers, the masonry must be built up to the proper elevation to receive it. When these elevations vary, as is the case at the end of a pitched or arched roof, then it may be necessary to proceed in alternate stages, progress of erection being interrupted by the work that must be performed by the masons, and vice versa. The necessary timing to avoid delays is seldom obtained. A few columns or an additional rigid frame at the end of a building may cost less than using trades to fit an intermittent and expensive schedule. Remember, too, that labor-union regulations may prevent the trades from handling any material other than that belonging to their own craft. An economical rule may well be: Lay out the work so that the erector and ironworkers can place and connect all the steelwork in one continuous operation.

(F. S. Merritt and R. Brockenbrough, "Structural Steel Designers Handbook," 2d ed., McGraw-Hill Publishing Company, New York.)

7.7 SKELETON FRAMING

In skeleton framing all the gravity loadings of the structure, including the walls are supported by the steel framework. Such walls are termed **nonbearing** or **curtain walls.** This system made the skyscraper possible. Steel, being so much stronger

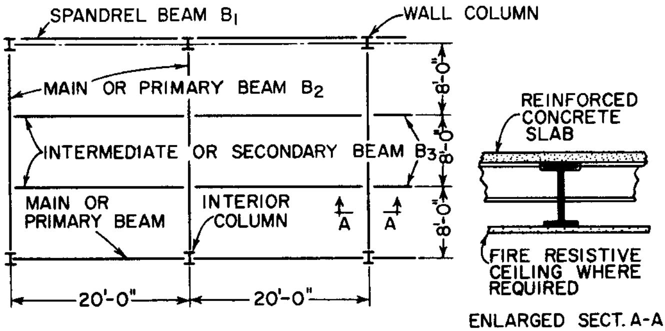


FIGURE 7.8 Typical beam-and-column steel framing, shown in plan.

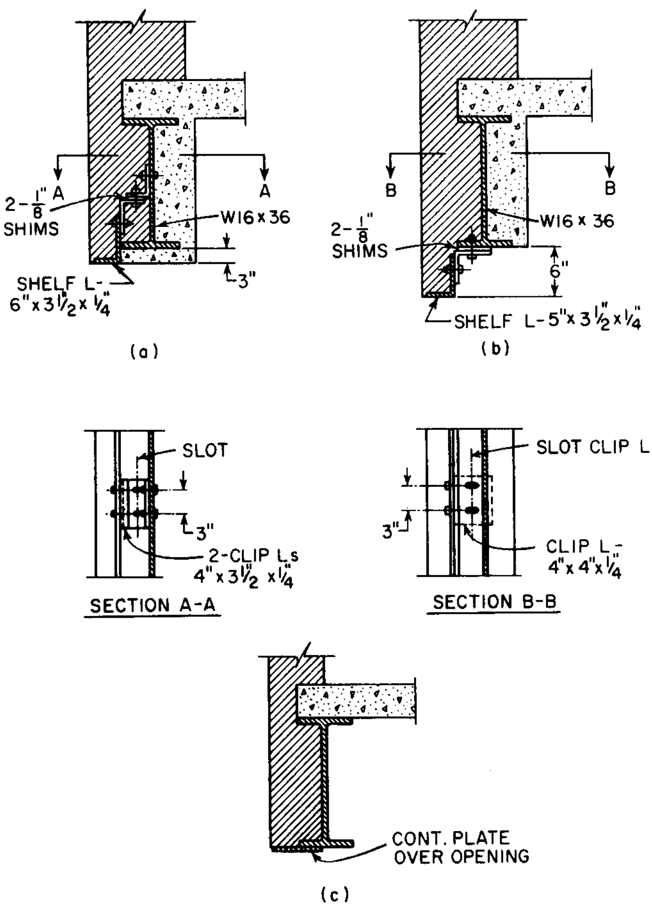


FIGURE 7.9 Typical steel spandrel beams.

than all forms of masonry, is capable of sustaining far greater load in a given space, thus obstructing less of the floor area in performing its function.

With columns properly spaced to provide support for the beams spanning between them, there is no limit to the floor and roof area that can be constructed with this type of framing, merely by duplicating the details for a single bay. Erected tier upon tier, this type of framing can be built to any desired height. Fabricators refer to this type of construction as ''beam and column.'' A typical arrangement is illustrated in Fig. 7.8.

The spandrel beams, marked B1 in Fig. 7.8, are located in or under the wall so as to reduce eccentricity caused by wall loads. Figure 7.9 shows two methods for connecting to the spandrel beam the shelf angle that supports the outer course of masonry over window openings 6 ft or more in width. In order that the masonry contractor may proceed expeditiously with the work, these shelf angles must be in alignment with the face of the building and at the proper elevation to match a masonry joint. The connection of the angles to the spandrel beams is made by bolting; shims are provided to make the adjustments for line and elevation.

Figure 7.9*a* illustrates a typical connection arrangement when the outstanding leg of the shelf angle is about 3 in or less below the bottom flange of the spandrel beam; Fig. 7.9*b* illustrates the corresponding arrangement when the outstanding leg of the shelf angle is more than about 3 in below the bottom flange of the spandrel beam. In the cases represented by Fig. 7.9*b*, the shelf angles are usually shipped attached to the spandrel beam. If the distance from the bottom flange to the horizontal leg of the shelf angle is greater than 10 in, a hanger may be required.

In some cases, as over door openings, the accurate adjustment features provided by Fig. 7.9*a* and *b* may not be needed. It may then be more economical to simplify the detail, as shown in Fig. 7.9*c*. The elevation and alignment will then conform to the permissible tolerances associated with the steel framework.

(E. H. Gaylord, Jr., et al., ''Design of Steel Structures,'' 3rd ed.; R. L. Brockenbrough and F. S. Merritt, ''Structural Steel Designers Handbook,'' 2d ed., McGraw-Hill Publishing Company, New York.)

7.8 LONG-SPAN FRAMING

Large industrial buildings, auditoriums, gymnasiums, theaters, hangars, and exposition buildings require much greater clear distance between supports than can be supplied by beam and column framing. When the clear distance is greater than can be spanned with rolled beams, several alternatives are available. These may be classified as *girders, simple trusses, arches, rigid frames, cantilever-suspension spans,* and various types of space frames, such as *folded plates, curvilinear grids, thin-shell domes, two-way trusses,* and *cable networks.*

Girders are the usual choice where depths are limited, as over large unobstructed areas in the lower floors of tall buildings, where column loads from floors above must be carried across the clear area. Sometimes, when greater strength is required than is available in rolled beams, cover plates are added to the flanges (Fig. 7.10*a*) to provide the additional strength.

When depths exceed the limit for rolled beams, i.e., for spans exceeding about 67 ft (based on the assumption of a depth-span ratio of 1:22 with 36-in-deep Ws), the girder must be built up from plates and shapes. Welded girders are used instead of the old-type conventional riveted girds (Fig. 7.10*b*), composed of web plate, angles, and cover plates.

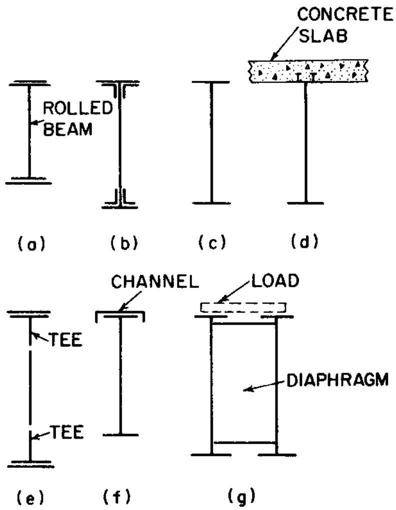


FIGURE 7.10 Typical built-up girders.

Welded girders generally are composed of three plates (Fig. 7.10*c*). This type offers the most opportunity for simple fabrication, efficient use of material, and least weight. Top and bottom flange plates may be of different size (Fig. 7.10*d*), an arrangement advantageous in composite construction, which integrates a concrete floor slab with the girder flange, to function together.

Heavy girders may use cover-plated tee sections (Fig. 7.10*e*). Where lateral loads are a factor, as in the case of girders supporting cranes, a channel may be fastened to the top flange (Fig. 7.10*f*). In exceptionally heavy construction, it is not unusual to use a pair of girders diaphragmed together to share the load (Fig. 7.10*g*).

The availability of high-strength, weldable steels resulted in development of **hybrid girders.** For example, a high-strength steel, say A572 Grade 50, whose yield stress is 50 ksi, may be used in a girder for the most highly stressed flanges, and the lower-priced A36 steel, whose yield stress is 36 ksi, may be used for lightly stressed flanges and web plate and detail material. The AISC specification for allowable-stress design requires that the top and bottom flanges at any cross section have the same cross-sectional area, and that the steel in these flanges be of the same grade. The allowable bending stress may be slightly less than that for conventional homogeneous girders of the high-strength steel, to compensate for possible overstress in the web at the junction with the flanges. Hybrid girders are efficient and economical for heavy loading and long spans and, consequently, are frequently employed in bridgework.

Trusses. When depth limits permit, a more economical way of spanning long distances is with trusses, for both floor and roof construction. Because of their greater depth, trusses usually provide greater stiffness against deflection when compared pound for pound with the corresponding rolled beam or plate girder that otherwise would be required. Six general types of trusses frequently used in building frames are shown in Fig. 7.11 together with modifications that can be made to suit particular conditions.

Trusses in Fig. 7.11*a* to *d* and *k* may be used as the principal supporting members in floor and roof framing. Types *e* to *j* serve a similar function in the framing of symmetrical roofs having a pronounced pitch. As shown, types *a* to *d* have a top chord that is not quite parallel to the bottom chord. Such an arrangement is used to provide for drainage of flat roofs. Most of the connections of the roof beams **(purlins),** which these trusses support, can be identical, which would not be the case if the top chord were dead level and the elevation of the purlins varied. When used in floors, truss types *a* to *d* have parallel chords.

Properly proportioned, bow string trusses (Fig. 7.11*j*) have the unique characteristic that the stress in their web members is relatively small. The top chord, which usually is formed in the arc of a circle, is stressed in compression, and the bottom chord is stressed in tension. In spite of the relatively expensive operation

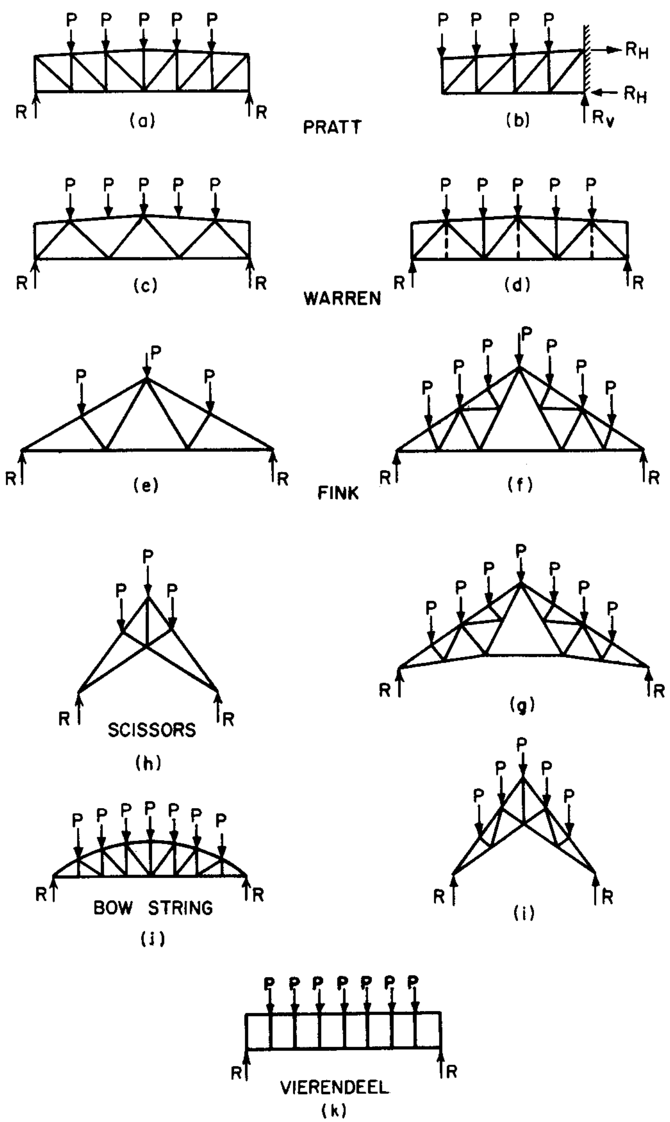


FIGURE 7.11 Types of steel trusses.

of forming the top chord, this type of truss has proved very popular in roof framing on spans of moderate lengths up to about 100 ft.

The Vierendeel truss (Fig. 7.11*k*) generally is shop welded to the extent possible to develop full rigidity of connections between the verticals and chords. It is useful where absence of diagonals is desirable to permit passage between the verticals.

Trusses also may be used for long spans, as three-dimensional trusses (space frames) or as grids. In two-way girds, one set of parallel lines of trusses is intersected at 90° by another set of trusses so that the verticals are common to both sets. Because of the rigid connections at the intersections, loads are distributed nearly equally to all trusses. Reduced truss depth and weight savings are among the apparent advantages of such grids.

Long-span joists are light trusses closely spaced to support floors and flat roofs. They conform to standard specifications (Table 7.1) and to standard loading. Both Pratt and Warren types are used, the shape of chords and webs varying with the fabricator. Yet, all joists with the same designation have the same guaranteed load-supporting capacity. The standard loading tables list allowable loads for joists up to 72 in deep and with clear span up to 144 ft. The joists may have parallel or sloping chords or other configuration.

Truss Applications. Cross sections through a number of buildings having roof trusses of the general type just discussed are shown diagrammatically in Fig. 7.12. Cross section *a* might be that of a storage building or a light industrial building. A Fink truss provides a substantial roof slope. Roofs of this type are often designed

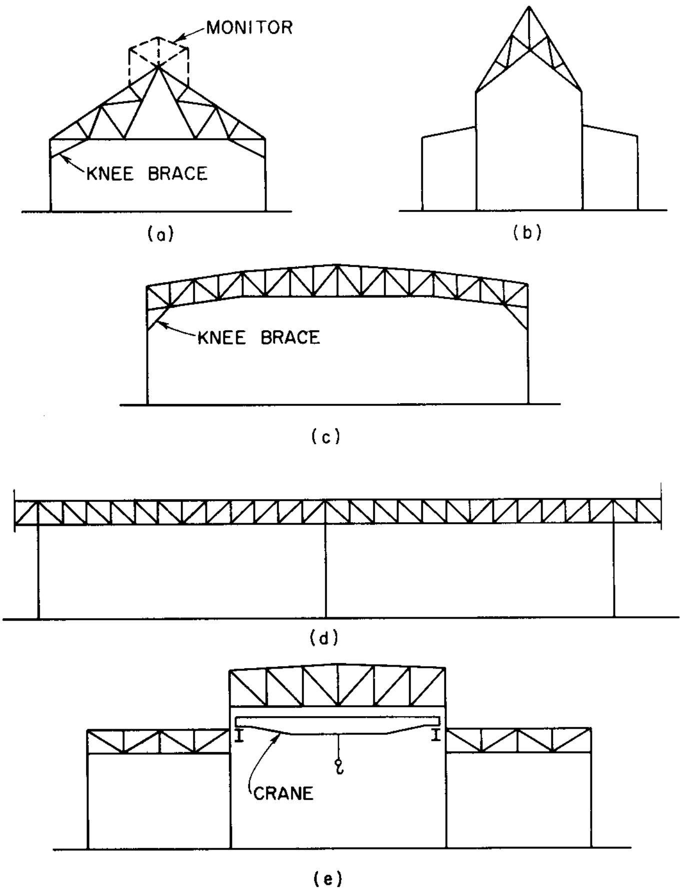


FIGURE 7.12 Some examples of structures with truss roofs.

to carry little loading, if any, except that produced by wind and snow, since the contents of the building are supported on the ground floor. For light construction, the roof and exterior wall covering may consist of thin, cold-formed metal panels. Lighting and ventilation, in addition to that provided by windows in the vertical side walls, frequently are furnished by means of sash installed in the vertical side of a continuous monitor, framing for which is indicated by the dotted lines in the sketch.

Cross section *b* shows a scissors truss supporting the high roof over the nave of a church. This type of truss is used only when the roof pitch is steep, as in ecclesiastical architecture.

A modified Warren truss, shown in cross section *c*, might be one of the main supporting roof members over an auditorium, gymnasium, theater, or other assembly-type building where large, unobstructed floor space is required. Similar trusses, including modified Pratt, are used in the roofs of large garages, terminal buildings, and airplane hangars, for spans ranging from about 80 up to 500 ft.

The Pratt truss (Fig. 7.12*d*) is frequently used in industrial buildings, while *e* depicts a type of framing often used where overhead traveling cranes handle heavy loads from one point on the ground to another.

Arches. When very large clear spans are needed, the bent framing required to support walls and roof may take the form of solid or open-web arches, of the kind shown in Fig. 7.13. A notable feature of bents *a* and *b* is the heavy steel pins at points *A, B,* and *C,* connecting the two halves of the arch together at the crown and supporting them at the foundation. These pines are designed to carry all the reaction from each half arch, and to function in shear and bearing much as a single bolt is assumed to perform when loaded in double shear.

Use of hinge pins offers two advantages in long-span frames of the type shown in Fig. 7.13. In the first place, they simplify design calculations. Second, they simplify erection. All the careful fitting can be done and strong connections required to develop the needed strength at the ends of the arch can be made in the shop, instead of high above ground in the field. When these heavy members have been raised in the field about in their final position, the upper end of each arch is adjusted, upward or downward, by means of jacks near the free end of the arch. When the holes in the pin plates line up exactly, the crown pins is slipped in place and secured against falling out by the attachment of keeper plates. The arch is then ready to carry its loading. Bents of the type shown in Fig. 7.13*a* and *b* are referred to as **three-hinged arches.**

When ground conditions are favorable and foundations are properly designed, and if the loads to be carried **are** relatively light, as, for example, for a large gymnasium, a **hingeless arch** similar to the one shown diagrammatically in Fig. 7.13*c* may offer advantage in overall economy.

In many cases, the arches shown in Fig. 7.13*a* and *b* are designed without the pins at *B* **(two-hinged arch).** Then, the section at *B* must be capable of carrying the moment and shear present. Therefore, the section at *B* may be heavier than for the three-hinged arch, and erection will be more exacting for correct closure.

Rigid Frames. These are another type of long-span bent. In design, the stiffness afforded by beam-to-column connections is carefully evaluated and counted on in the design to relieve some of the bending moment that otherwise would be assumed as occurring with maximum intensity at midspan. Typical examples of rigid frame bents are shown in Fig. 7.14. When complete assembled in place in the field, the

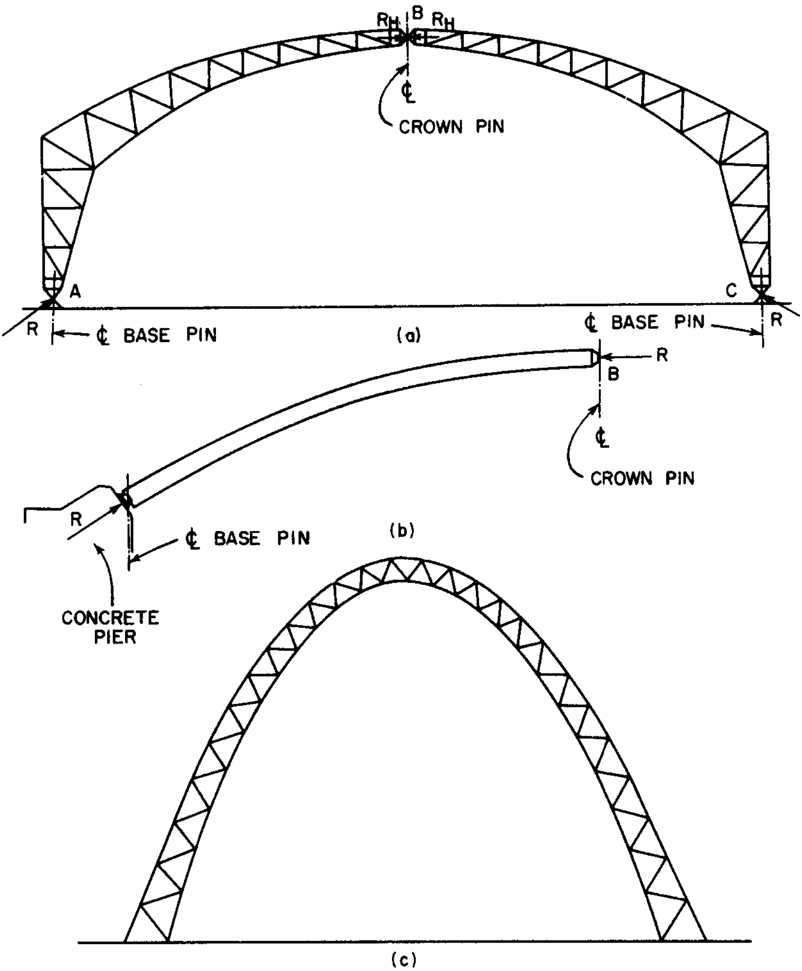


FIGURE 7.13 Steel arches: (*a*) and (*b*) three-hinged; (*c*) fixed.

frames are fully continuous throughout their entire length and height. A distinguishing characteristic of rigid frames is the absence of pins or hinges at the crown, or midspan.

In principle, single-span rigid-frame bents are either two-hinged or hingeless arches. For hingeless arches, the column bases are fully restrained by large rigid foundations, to which they are attached by a connection capable of transmitting moment as well as shear. Since such foundations may not be economical or even possible when soil conditions are not favorable, the usual practice is to consider the bents hinged at each reaction. However, this does not imply the necessity of expensive pin details; in most cases, sufficient rotation of the column base can be obtained with the ordinary flat-ended base detail and a single line of anchor bolts

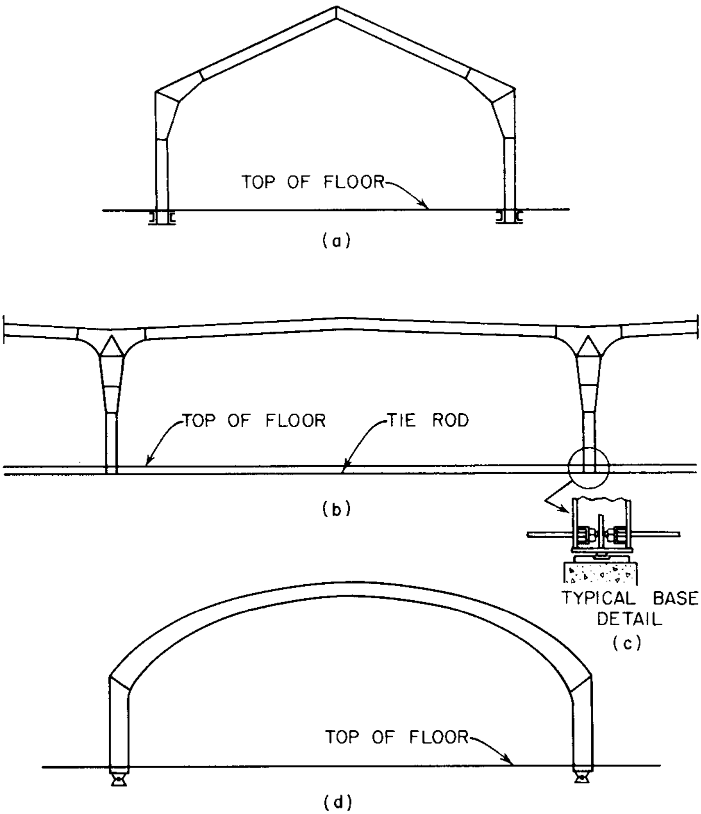


FIGURE 7.14 Steel rigid frames: (*a*) single bent; (*b*) continuous frame with underfloor tie; (*c*) connection of tie to a column; (*d*) with two-hinged.

placed perpendicular to the span on the column center line. Many designers prefer to obtain a hinge effect by concentrating the column load on a narrow bar, as shown in Fig. 7.14*c*; this refinement is worthwhile in larger spans.

Regardless of how the frame is hinged, there is a problem in resisting the horizontal shear that the rigid frame imparts to the foundation. For small spans and light thrusts, it may be feasible to depend on the foundation to resist lateral displacement. However, more positive performance and also reduction in costs are usually obtained by connecting opposite columns of a frame with tie rods, as illustrated in Fig. 7.14*b*, thus eliminating these horizontal forces from the foundation.

For ties on small spans, it may be possible to utilize the reinforcing bars in the floor slab or floor beams, by simply connecting them to the column bases. On larger spans, it is advisable to use tie rods and turnbuckles, the latter affording the opportunity to prestress the ties and thus compensate for elastic elongation of the rods when stressed. Prestressing the rod during erection to 50% of its value has been recommended for some major installations; but the foundations should be checked for resisting some portion of the thrust.

Single-story, welded rigid frames often are chosen where exposed steelwork is desired for such structures as churches, gymnasiums, auditoriums, bowling alleys, and shopping centers, because of attractive appearance and economy. Columns may be tapered, girders may vary in depth linearly or parabolically, haunches (knees) may be curved, field joints may be made inconspicuous, and stiffness may simply be plates.

Field Splices. One problem associated with long-span construction is that of locating field splices compatible with the maximum sizes of members that can be shipped and erected. Field splices in frames are generally located at or near the point of counterflexure, thus reducing the splicing material to a minimum. In general, the maximum height for shipping by truck is 8 ft, by rail 10 ft. Greater overall depths are possible, but these should always be checked with the carrier; they vary with clearances under bridges and through tunnels.

Individual shipping pieces must be stiff enough to be handled without buckling or other injury, light enough to be lifted by the raising equipment, and capable of erection without interference from other parts of the framework. This suggests a study of the entire frame to ensure orderly erection, and to make provisions for temporary bracing of the members, to prevent jackknifing, and for temporary guying of the frame, to obtain proper alignment.

Hung-Span Beams. In some large one-story buildings, an arrangement of cantilever-suspension (hung) spans (Fig. 7.15) has proved economical and highly efficient. This layout was made so as to obtain equal maximum moments, both negative and positive, for the condition of uniform load on all spans. A minimum of three spans is required; that is, a combination of two end spans (*A*) and one intermediate span (*C*). The connection at the end of the cantilever (point *D*) must be designed as a shear connection only. If the connection is capable of transmitting moment as well as shear, it will change the design to one of continuity and the dimensions in Fig. 7.15 will not apply. This scheme of cantilever and suspended spans is not necessarily limited to one-story buildings.

As a rule, interior columns are separate elements in each story. Therefore, horizontal forces on the building must be taken solely by the exterior columns.

(E. H. Gaylord, Jr., et al., "Design of Steel Structures," 3d ed.; and F. S. Merritt and R. L. Brockenbrough, "Structural Steel Designer's Handbook," 2d ed., McGraw-Hill Publishing Company, New York.)

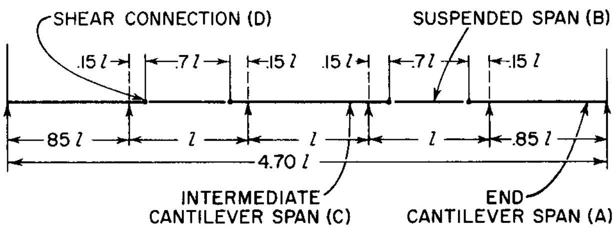


FIGURE 7.15 Hung- or suspended-span steel construction.

7.9 STEEL AND CONCRETE FRAMING

In another type of framing system, different from those described in Arts. 7.7 and 7.8, a partial use of structural steel has an important role, namely, **composite framing** of reinforced concrete and structural steel.

Composite construction actually occurs whenever concrete is made to assist steel framing in carrying loads. The term composite, however, often is used for the specific cases in which concrete slabs act together with flexural members.

Reinforced-concrete columns of conventional materials when employed in tall buildings and for large spans become excessively large. One method of avoiding this objectionable condition is to use high-strength concrete and high-strength reinforcing bars. Another is to use a structural-steel column core. In principle, the column load is carried by both the steel column and the concrete that surrounds the steel shape. Building codes usually contain an appropriate formula for this condition.

A number of systems employ a combination of concrete and steel in various ways. One method features steel columns supporting a concrete floor system by means of a steel shearhead connected to the columns at each floor level. The shallow grillage is embedded in the floor slab, thus obtaining a smooth ceiling without drops or capitals.

Another combination system is the **lift-slab** method. In this system, the floor slabs are cast one on top of another at ground level. Jacks, placed on the permanent steel columns, raise the slabs, one by one, to their final elevation, where they are made secure to the columns. When fireproofing is required, the columns may be boxed in with any one of many noncombustible materials available for that purpose. The merit of this system is the elimination of formwork and shoring that are essential in conventional reinforced-concrete construction.

For high-rise buildings, structural-steel framing often is used around a central, load-bearing, concrete core, which contains elevators, stairways, and services. The thick walls of the core, whose tubular configuration may be circular, square, or rectangular, are designed as shear walls to resist all the wind forces as well as gravity loads. Sometimes, the surrounding steel framing is cantilevered from the core, or the perimeter members are hung from trusses or girders atop the core and possibly also, in very tall buildings, at midheight of the core.

FRAME AND MEMBER BRACING SYSTEMS

7.10 BRACING DESIGN CONSIDERATIONS

Bracing as it applies to steel structures includes secondary members incorporated into the system of main members to serve these principal functions:

1. Slender compression members, such as columns, beams, and truss elements are braced, or laterally supported, so as to restrain the tendency to buckle in a direction normal to the stress path. The rigidity, or resistance to buckling, of an individual member is determined from its length and certain physical properties of its cross section. Economy and size usually determine whether bracing is to be employed.

2. Since most structures are assemblies of vertical and horizontal members forming rectangular (or square) panels, they possess little inherent rigidity. Consequently, additional rigidity must be supplied by a secondary system of members or by rigid or semi-rigid joints between members. This is particularly necessary when the framework is subject to lateral loads, such as wind, earthquakes, and moving loads. Exempt from this second functional need for bracing are trusses, which are basically an arrangement of triangles possessing in their planes an inherent ideal rigidity both individually and collectively.

3. There frequently is a need for bracing to resist erection loads and to align or prevent overturning, in a direction normal to their planes, of trusses, bents, or frames during erection. Such bracing may be temporary; however, usually bracing needed for erection is also useful in supplying rigidity to the structure and therefore is permanently incorporated into the building. For example, braces that tie together adjoining trusses and prevent their overturning during erection are useful to prevent sway—even though the swaying forces may not be calculable.

7.11 FRAME BRACING

Design of bracing to resist forces induced by wind, seismic disturbances, and moving loads, such as those caused by cranes, is not unlike, in principle, design of members that support vertical dead and live loads. These lateral forces are readily calculable. They are collected at points of application and then distributed through the structural system and delivered to the ground. Wind loads, for example, are collected at each floor level and distributed to the columns that are selected to participate in the system. Such loads are cumulative; that is, columns resisting wind shears must support at any floor level all the wind loads on the floors above the one in consideration.

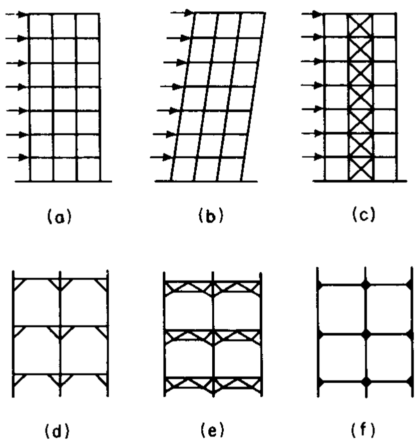


FIGURE 7.16 Wind bracing for multistory buildings.

7.11.1 Bracing Tall Buildings

If the steel frame of the multistory building in Fig. 7.16*a* is subjected to lateral wind load, it will distort as shown in Fig. 7.16*b*, if the connections of columns and beams are of the standard type, for which rigidity (resistance to rotation) is nil. One can visualize this readily by assuming each joint is connected with a single pin. Naturally, the simplest method to prevent this distortion is to insert diagonal members—triangles being inherently rigid, even if all the members forming the triangles are pin-connected.

Braced Bents. Bracing of the type in Fig. 7.16*c*, called X bracing, is both efficient and economical. Unfortunately, X bracing is usually impracticable because of interference with doors, windows, and clearance between floor and ceiling. Usually, for office buildings large column-free areas are required. This offers flexibility of space use, with movable partitions. But about the only place for X bracing in this type of building is in the elevator shaft, fire tower, or wherever a windowless wall is required. As a result, additional bracing must be supplied by other methods. On the other hand, X bracing is used extensively for bracing industrial buildings of the shed or mill type.

Moment-Resisting Frames. Designers have a choice of several alternatives to X bracing. Knee braces, shown in Fig. 7.16*d*, or portal frames, shown in Fig. 7.16*e*, may be used in outer walls, where they are likely to interfere only with windows. For buildings with window walls, the bracing often used is the bracket type (Fig. 7.16*f*). It simply develops the end connection for the calculated wind moment. Connections vary in type, depending on size of members, magnitude of wind moment, and compactness needed to comply with floor-to-ceiling clearances.

Figure 7.17 illustrates a number of bracket-type wind-braced connections. The minimum type, represented in Fig. 7.17*e*, consists of angles top and bottom: They are ample for moderate-height buildings. Usually the outstanding leg (against the column) is of a size that permits only one gage line. A second line of fasteners would not be effective because of the eccentricity. When greater moment resistance is needed, the type shown in Fig. 7.17*b* should be considered. This is the type that has become rather conventional in field-bolted construction. Figure 7.17*c* illustrates the maximum size with beam stubs having flange widths that permit additional gage lines, as shown. It is thus possible on larger wide-flange columns to obtain 16 fasteners in the stub-to-column connection.

The resisting moment of a given connection varies with the distance between centroids of the top and bottom connection piece. To increase this distance, thus increasing the moment, an auxiliary beam may be introduced as shown in Fig. 7.17*d*, if it does not create an interference.

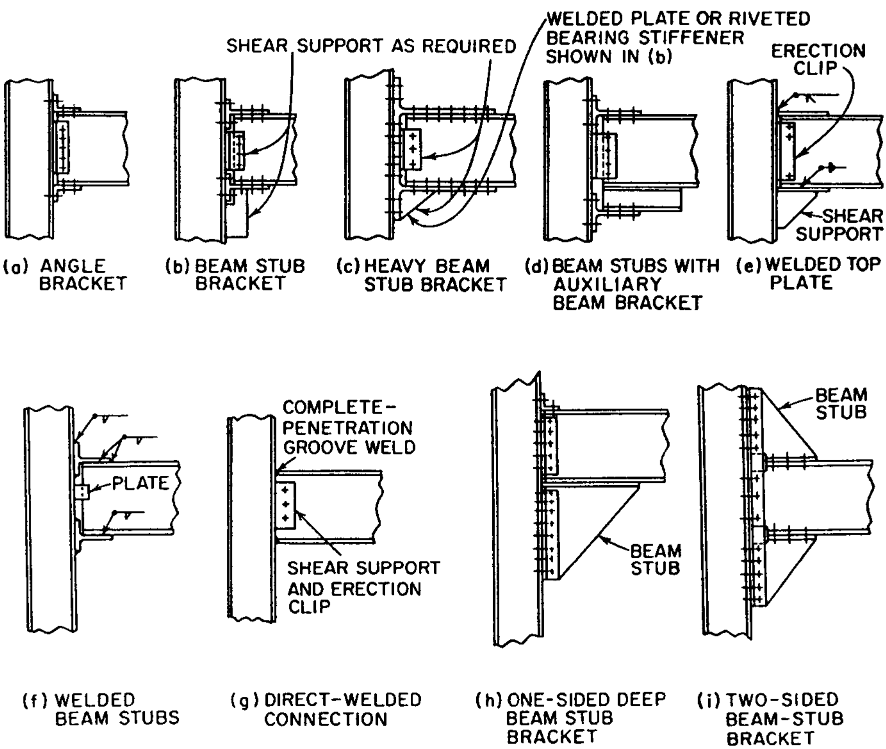


FIGURE 7.17 Typical wind connections for beams to columns.

All the foregoing types may be of welded construction, rather than bolted. In fact, it is not unusual to find mixtures of both because of the fabricator's decision to shop-bolt and field-weld, or vice versa. Welding, however, has much to offer in simplifying details and saving weight, as illustrated in Fig. 7.17*e*, *f*, and *g*. The last represents the ultimate efficiency with respect to weight saving, and furthermore, it eliminates interfering details.

Deep wing brackets (Fig. 7.17*h* and *i*) are sometimes used for wall beams and spandrels designed to take wind stresses. Such deep brackets are, of course, acceptable for interior beam bracing whenever the brackets do not interfere with required clearances.

Not all beams need to wind-braced in tall buildings. Usually the wind load is concentrated on certain column lines, called **bents,** and the forces are carried through the bents to the ground. For example, in a wing of a building, it is possible to concentrate the wind load on the outermost bent. To do so may require a stiff floor or diaphragm-like system capable of distributing the wind loads laterally. One-half these loads may be transmitted to the outer bent, and one-half to the main building to which the wing connects.

Braced bents are invariably necessary across the narrow dimension of a building. The question arises as to the amount of bracing required in the long dimension, since wind of equal unit intensity is assumed to act on all exposed faces of structures. In buildings of square or near square proportions, it is likely that braced bents will be provided in both directions. In buildings having a relatively long dimension, as compared with width, the need for bracing diminishes. In fact, in many instances, wind loads are distributed over so many columns that the inherent rigidity of the whole system is sufficient to preclude the necessity of additional bracing.

Column-to-column joints are treated differently for wind loads. Columns are compression members and transmit their loads, from section above to section below, by direct bearing between finished ends. It is not likely, in the average building, for the tensile stresses induced by wind loads ever to exceed the compressive pressure due to dead loads. Consequently, there is no theoretical need for bracing a column joint. Actually, however, column joints are connected together with nominal splice plates for practical considerations—to tie the columns during erection and to obtain vertical alignment.

This does not mean that designers may always ignore the adequacy of column splices. In lightly loaded structures, or in exceptionally tall but narrow buildings, it is possible for the horizontal wind forces to cause a net uplift in the windward column because of the overturning action. The commonly used column splices should then be checked for their capacity to resist the maximum net tensile stresses caused in the column flanges. This computation and possible heaving up of the splice material may not be thought of as bracing; yet, in principle, the column joint is being "wind-braced" in a manner similar to the wind-braced floor-beam connections.

7.11.2 Shear Walls

Masonary walls enveloping a steel frame, interior masonry walls, and perhaps some stiff partitions can resist a substantial amount of lateral load. Rigid floor systems participate in lateral-force distribution by distributing the shears induced at each floor level to the columns and walls. Yet, it is common design practice to carry wind loads on the steel frame, little or no credit being given to the substantial resistance rendered by the floors and walls. In the past, some engineers deviated from this conversatism by assigning a portion of the wind loads to the floors and walls; nevertheless, the steel frame carried the major share. When walls of glass or thin metallic curtain walls, lightweight floors, and removable partitions are used, this construction imposes on the steel frame almost complete responsibility for transmittal of wind loads to the ground. Consequently, windbracing is critical for tall steel structures.

In tall, slender buildings, such as hotels and apartments with partitions, the cracking of rigid-type partitions is related to the wracking action of the frame caused by excessive deflection. One remedy that may be used for exceptionally slender frames (those most likely to deflect excessively) is to supplement the normal bracing of the steel frame with shear walls. Acting as vertical cantilevers in resisting lateral forces, these walls, often constructed of reinforced concrete, may be arranged much like structural shapes, such as plates, channels, Ts, Is, or Hs. (See also Arts. 3.2.4 and 5.12.) Walls needed for fire towers, elevator shafts, divisional walls, etc., may be extended and reinforced to serve as shear walls, and may relieve the steel frame of cumbersome bracing or avoid uneconomical proportions.

7.11.3 Bracing Industrial-Type Buildings

Bracing of low industrial buildings for horizontal forces presents fewer difficulties than bracing of multistory buildings, because the designer usually is virtually free to select the most efficient bracing without regard to architectural considerations or interferences. For this reason, conventional X bracing is widely used—but not exclusively. Knee braces, struts, and sway frames are used where needed.

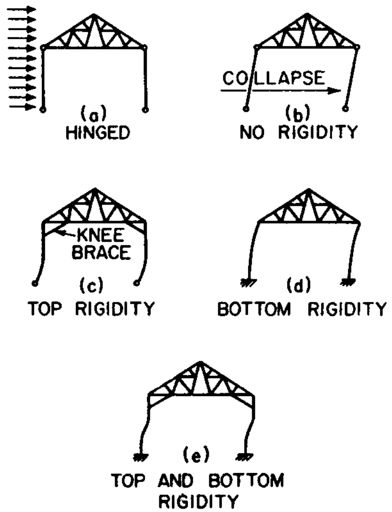


FIGURE 7.18 Relative stiffness of bents depends on restraints on columns.

Wind forces acting on the frame shown in Fig. 7.18*a*, with hinged joints at the top and bottom of supporting columns, would cause collapse as indicated in Fig. 7.18*b*. In practice, the joints would not be hinged. However, a minimum-type connection at the truss connection and a conventional column base with anchor bolts located on the axis transverse to the frame would approximate this theoretical consideration of hinged joints. Therefore, the structure requires bracing capable of preventing collapse or unacceptable deflection.

In the usual case, the connection between truss and columns will be stiffened by means of knee braces (Fig. 7.18*c*). The rigidity so obtained may be supplemented by providing partial rigidity at the column base by simply locating the anchor bolts in the plane of the bent.

In buildings containing overhead cranes, the knee braced may interfere with crane operation. Then, the interference may be eliminated by fully anchoring the column base so that the column may function as a vertical cantilever (Fig. 7.18*d*).

The method often used for very heavy industrial buildings is to obtain substantial rigidity at both ends of the column so that the behavior under lateral load will resemble the condition illustrated in Fig. 7.18*e*. In both (*d*) and (*e*), the footings must be designed for such moments.

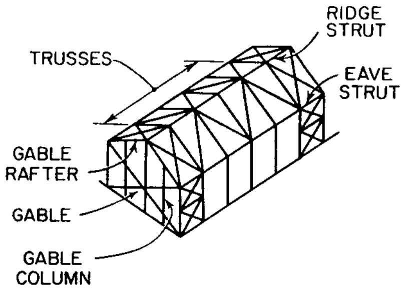


FIGURE 7.19 Braced bays in framing for an industrial building.

A common assumption in wind distribution for the type of light mill building shown in Fig. 7.19 is that the windward columns take a large share of the load acting on the side of the building and deliver the load directly to the ground. The remaining wind load on the side is delivered by the same columns to the roof systems, where the load joins with the wind forces imposed directly on the roof surface. Then, by means of diagonal X bracing, working in conjunction with the struts and top chords of the trusses, the load is carried to the eave struts, thence to the gables and, through diagonal bracing, to the foundations.

Because wind may blow from any direction, the building also must be braced for the wind load on the gables. This bracing becomes less important as the building increases in length and conceivably could be omitted in exceptionally long structures. The stress path is not unlike that assumed for the transverse wind forces. The load generated on the ends is picked up by the roof system and side framing, delivered to the eave struts, and then transmitted by the diagonals in the end sidewall bays to the foundation.

No distribution rule for bracing is intended in this discussion; bracing can be designed many different ways. Whereas the foregoing method would be sufficient for a small building, a more elaborate treatment may be required for larger structures.

Braced bays, or towers, are usually favored for structures such as that shown in Fig. 7.20. There, a pair of transverse bents are connected together with X bracing in the plane of the columns, plane of truss bottom chords, plane of truss top chords, and by means of struts and sway frames. It is assumed that each such tower can carry the wind load from adjacent bents, the number depending on assumed rigid-

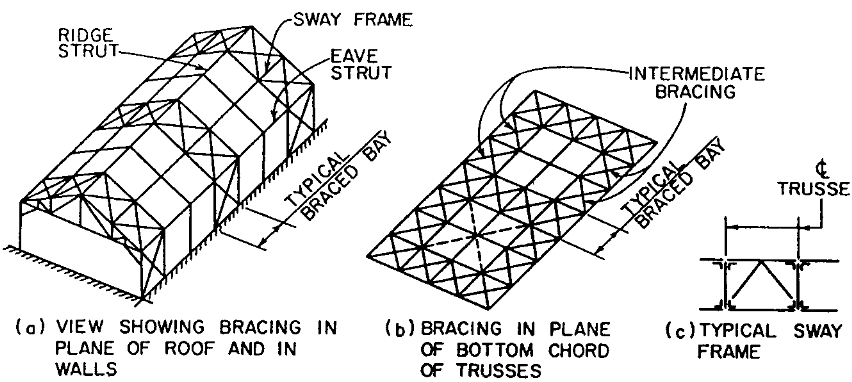


FIGURE 7.20 Braced bays in a one-story building transmit wind loads to the ground.

ities, size, span, and also on sound judgment. Usually every third or fourth bent should become a braced bay. Participation of bents adjoining the braced bay can be assured by insertion of bracing designated "intermediate" in Fig. 7.20*b*. This bracing is of greater importance when knee braces between trusses and columns cannot be used. When maximum lateral stiffness of intermediate bents is desired, it can be obtained by extending the X bracing across the span; this is shown with broken lines in Fig. 7.20*b*.

Buildings with flat or low-pitched roofs, shown in Fig. 7.12*d* and *e*, require little bracing because the trusses are framed into the columns. These columns are designed for the heavy moments induced by wind pressure against the building side. The bracing that would be provided, at most, would consist of X bracing in the plane of the bottom chords for purpose of alignment during erection and a line or two of sway frames for longitudinal rigidity. Alignment bracing is left in the structure since it affords a secondary system for distributing wind loads.

7.11.4 Bracing Craneway Structures

All building framing affected by overhead cranes should be braced for the thrusts induced by sidesway and longitudinal motions of the cranes. Bracing used for wind or erection may be assumed to sustain the lateral crane loadings. These forces are usually concentrated on one bent. Therefore, normal good practice dictates that adjoining bents share in the distribution. Most effective is a system of X bracing located in the plane of the bottom chords of the roof trusses.

In addition, the bottom chords should be investigated for possible compression, although the chords normally are tension members. A heavily loaded crane is apt to draw the columns together, conceivably exerting a greater compression stress than the tension stress obtainable under dead load alone. This may indicate the need for intermediate bracing of the bottom chord.

7.11.5 Bracing Rigid Frames

Rigid frames of the type shown in Fig. 7.14 have enjoyed popular usage for gymnasiums, auditoriums, mess halls, and with increasing frequency, industrial buildings. The stiff knees at the junction of the column with the rafter imparts excellent transverse rigidity. Each bent is capable of delivering its share of wind load directly to the footings. Nevertheless, some bracing is advisable, particularly for resisting wind loads against the end of the building. Most designers emphasize the importance of an adequate eave strut; it usually is arranged so as to brace the inside flange (compression) of the frame knee, the connection being located at the midpoint of the transition between column and rafter segments of the frame. Intermediate X bracing in the plane of the rafters usually is omitted.

7.12 BRACING FOR INDIVIDUAL MEMBERS

For an ideally straight, exactly concentrically loaded beam or column, only a small force may be needed from an intermediate brace to reduce the unbraced length of a column or the unsupported length of the compression flange of a beam. But there is no generally accepted method of calculating that force.

The principal function of a brace is to provide a node in the buckled configuration. Hence, rigidity is the main requirement for the brace. But actual members do contain nonuniform residual stresses and slight initial crookedness and may be slightly misaligned, and these eccentricities create deformations that must be resisted by the brace.

A rule used by some designers that has proved satisfactory is to design the brace for 2% of the axial load of columns, or 2% of the total compressive stress in beam flanges. Studies and experimental evidence indicate that this rule is conservative.

7.12.1 Column Bracing

Interior columns of a multistory building are seldom braced between floor connections. Bracing of any kind generally interferes with occupancy requirements and architectural considerations. Since the slenderness ratio l/r in the weak direction usually controls column size, greatest economy is achieved by using only wide-flange column sections or similar built-up sections.

It is frequently possible to reduce the size of wall columns by introducing knee braces or struts in the plane of the wall, or by taking advantage of deep spandrels or girts that may be otherwise required. Thus the slenderness ratio of the weak and strong axis can be brought into approximate balance. The saving in column weight may not always be justified; one must take into account the weight of additional bracing and cost of extra details.

Column bracing is prevalent in industrial buildings because greater vertical clearances necessitate longer columns. Tall slender columns may be braced about both axes to obtain an efficient design.

Undoubtedly, heavy masonry walls afford substantial lateral support to steel columns embedded wholly or partly in the wall. The general practice, however, is to disregard this assistance.

An important factor in determining column bracing is the allowable stress or load for the column section (Art. 7.19). Column formulas for obtaining this stress are based on the ratio of two variables, effective length Kl and the physical property called radius of gyration r.

The question of when to brace (to reduce the unsupported length and thus slenderness ratio) is largely a matter of economics and architectural arrangements; thus no general answer can be given.

7.12.2 Beam Bracing

Economy in size of member dictates whether laterally unsupported beams should have additional lateral support between end supports. Lateral support at intermediate points should be considered whenever the allowable stress obtained from the reduction formulas for large l/r_t falls below some margin, say 25%, of the stress allowed for the fully braced condition. There are cases, however, where stresses as low as 4.0 ksi have been justified, because intermediate lateral support was impractical.

The question often arises: When is a steel beam laterally supported? There is no fixed rule in specifications (nor any intended in this discussion) because the answer requires application of sound judgment based on experiences. Tests and studies that have been made indicate that it takes rather small forces to balance the lateral thrusts of initial buckling.

Figure 7.21 illustrates some of the common situations encountered in present-day practice. In general, **positive lateral support is provided by:**

(*a*) and (*b*) All types of cast-in-place concrete slabs (questionable for vibrating loads and loads hung on bottom flange).

(*c*) Metal and steel plate decks, with welded connections.

(*d*) Wood decks nailed securely to nailers bolted to the beam.

(*e*) and (*f*) Beam flange tied or braced to strut system, either as shown in (*e*) or by means of cantilever tees, as shown in (*f*); however, struts should be adequate to resist rotation.

(*g*) Purlins used as struts, with tees acting as cantilevers (common in rigid frames and arches). If plate stiffness are used, purlins should be connected to them with high-strength bolts to ensure rigidity.

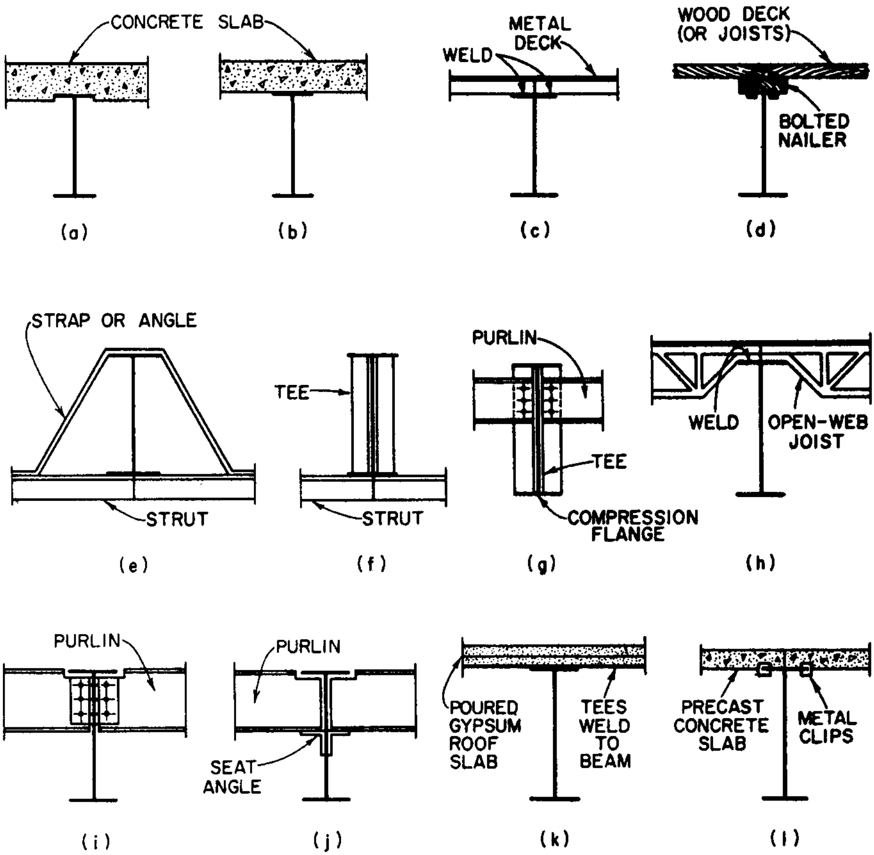


FIGURE 7.21 Methods of providing lateral support for beams.

(*h*) Open-web joists tack-welded (or the equivalent) to the beams, but the joists themselves must be braced together (bridging), and the flooring so engaged with the flanges that the joists, in turn, are adequately supported laterally.

(*i*) Purlins connected close to the compression flange.

(*k*) Tees (part of cast-in-place gypsum construction) welded to the beams.

Doubtful lateral support is provided by:

(*j*) Purlins seated on beam webs, where the seats are distant from the critical flange

(*l*) Precast slabs not adequately fastened to the compression flange.

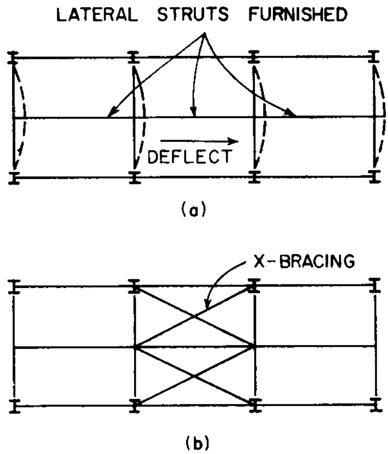


FIGURE 7.22 Lateral bracing systems; (*a*) without and (*b*) with X bracing.

The reduction formulas for large l/r, given in Fig. 7–31 do not apply to steel beams fully encased in concrete, even though no other lateral support is provided.

Introducing a secondary member to cut down the unsupported length does not necessarily result in adequate lateral support. The resistive capacity of the member and its supports must be traced through the system to ascertain effectiveness. For example the system in Fig. 7.22*a* may be free to deflect laterally as shown. This can be prevented by a rigid floor system that acts as a diaphragm, or in the absence of a floor, it may be necessary to X-brace the system as shown in Fig. 7.22*b*.

FLOOR AND ROOF SYSTEMS

7.13 FLOOR-FRAMING DESIGN CONSIDERATIONS

Selection of a suitable and economical floor system for a steel-frame building involves many considerations: load-carrying capacity, durability, fire resistance, dead weight, overall depth, facility for installing power, light, and telephones, facility for installing aid conditioning, sound transmission, appearance, maintenance, and construction time.

Building codes specify minimum design live loads for floor and roof systems. In the absence of a code regulation, one may use "Minimum Design Loads in Buildings and Other Structures," ASCE 7-93, American Society of Civil Engineers. See also Art. 5.1.2. Floors should be designed to support the actual loading or these minimum loads, whichever is larger. Most floors can be designed to carry any given load. However, in some instances, a building code may place a maximum load limit on particular floor systems without regard to calculated capacity.

Resistance to lateral forces should not be disregarded, especially in areas of seismic disturbances or for perimeter windbents. In designs for such conditions, floors may be employed as horizontal diaphragms to distribute lateral forces to walls or vertical framing; those elements then transmit the lateral forces to the foundations. When using lightweight floor systems, special reinforcement in the floor slab may be necessary at those points where the floor diaphragm transfers the horizontal forces to the frame elements.

Durability becomes a major consideration when a floor is subject to loads other than static or moderately kinetic types of forces. For example, a light joist system may be just the floor for an apartment or an office building but may be questionable for a manufacturing establishment where a floor must resist heavy impact and severe vibrations. Shallow floor systems deflect more than deep floors; the system selected should not permit excessive or objectionable deflections.

Fire resistance and fire rating are very important factors, because building codes in the interest of public safety, specify the degree of resistance that must be provided. Many floor systems are rated by the codes or by fire underwriters for purposes of satisfying code requirements or basing insurance rates.

The dead weight of the floor system, including the framing, is an important factor affecting economy of construction. For one thing, substantial saving in the weight and cost of a steel frame may result with lightweight floor systems. In addition, low dead weight may also reduce foundation costs.

Joist systems, either steel or concrete, require no immediate support, since they are obtainable in lengths to meet normal bay dimensions in tier building construction. On the other hand, concrete arch and cellular-steel floors are usually designed with one or two intermediate beams within the panel. The elimination of secondary beams does not necessarily mean overall economy just because the structural-steel contract is less. These beams are simple to fabricate and erect and allow much duplication. An analysis of contract price shows that the cost per ton of secondary beams will average 20% under the cost per ton for the whole steel structure; or viewed another way, the omission of secondary beams increases the price per ton on the balance of the steelwork by 3½% on the average. This fact should be taken into account when making a cost analysis of several systems.

Sometimes, the depth of a floor system is important. For example, the height of a building may be limited for a particular type of fire-resistant construction or by zoning laws. The thickness of the floor system may well be the determining factor limiting the number of stories that can be built. Also, the economy of a deep floor is partly offset by the increase in height of walls, columns, pipes, etc.

Another important consideration, particularly for office buildings and similar-type occupancies, is the need for furnishing an economical and flexible electrical wiring system. With the accent on movable partitions and ever-changing office arrangements, the readiness and ease with which telephones, desk lights, computers, and other electric-powered business machines can be relocated are of major importance. Therefore, the floor system that by its makeup provides large void spaces or cells for concealing wiring possesses a distinct advantage over competitive types of solid construction. Likewise, accommodation of recessed lighting in ceilings may disclose an advantage for one system over another. Furthermore, for economical air conditioning and ventilation, location of ducts and method of support warrant study of several floor systems.

Sound transmission and acoustical treatments are other factors that need to be evaluated. A wealth of data are available in reports of the National Institute of Standards and Technology. In general, floor systems of sandwich type with air spaces between layers afford better resistance to sound transmission than solid systems, which do not interrupt sound waves. Although the ideal soundproof floor is impractical, because of cost, several reasonably satisfactory systems are available. Much depends on type of occupancy, floor coverings, and ceiling finish—acoustical plaster or tile.

Appearance and maintenance also should be weighed by the designer and the owner. A smooth, neat ceiling is usually a prerequisite for residential occupancy; a less expensive finish may be deemed satisfactory for an institutional building.

Speed of construction is essential. Contractors prefer systems that enable the follow-up trades to work immediately behind the erector and with unimpeded efficiency.

In general, either rolled beams or open-web joists are used to support the floor elements. The most common types of flooring are (a) concrete fill on metal deck, (b) pre-cast concrete plank, and (c) cast-in-place concrete floors with integral joist. Metal decks may be cellular or plain and are usually stud-welded to the supporting elements to provide composite action. Cast-in-place concrete floors, or concrete-pan floors, are becoming less common than in the past. In addition to the systems described, there are several adaptations of these as well as other proprietary systems.

7.13.1 Steel Joist Floors

The lightest floor system in common use is the open-web steel joist construction shown in Fig. 7.23. It is popular for all types of light occupancies, principally because of initial low cost.

Many types of open-web joists are available. Some employ bars in their makeup, while others are entirely of rolled shapes; they all conform to standards and good-practice specifications promulgated by the Steel Joist Institute and the American Institute of Steel Construction (see Table 7.1). All joists conform to the standard loading tables and carry the same size designation so that designers need only indicate on project drawings the standard marking without reference to manufacturer, just as for a steel beam or column section.

Satisfactory joists construction is assured by adhering to SJI and AISC recommendations. Joists generally are spaced 2 ft c to c. They should be adequately braced (with bridging) during construction to prevent rotation or buckling, and to avoid ‘‘springy’’ floors, they should be carefully selected to provide sufficient depth.

This system has many advantages: Falsework is eliminated. Joists are easily handled, erected, and connected to supporting beams—usually by tack welding.

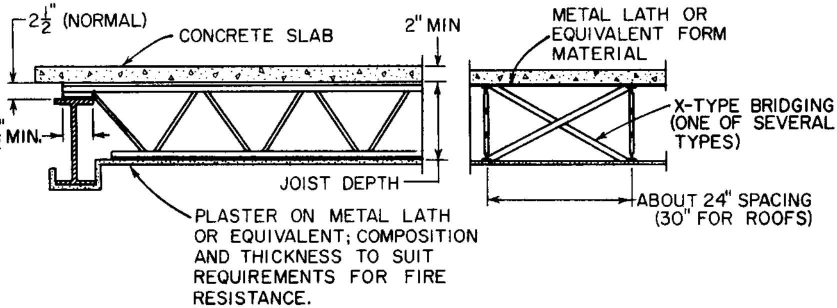


FIGURE 7.23 Open-web steel joist construction.

Temporary coverage and working platforms are quickly placed. The open space between joists, and through the webs, may be utilized for ducts, cables, light fixtures, and piping. A thin floor slab may be cast on steel lath, corrugated-steel sheets, or wire-reinforced paper lath laid on top of the joists. A plaster ceiling may be suspended or attached directly to the bottom flange of the joists.

Lightweight beams, or so-called "junior" beams, are also used in the same manner as open-web joists, and with the same advantages and economy, except that the solid webs do not allow as much freedom in installation of utilities. Beams may be spaced according to their safe load capacity; 3- and 4-ft spacings are common. As a type, therefore, the lightweight-steel-beam floor is intermediate between concrete arches and open-web joists.

7.13.2 Cellular-Steel Floors

Cold-formed steel decking is frequently used in office buildings. One type is illustrated in Fig. 7.24. Other manufacturers make similar cellular metal decks, the primary difference being in the shape of the cells. Often, decking with half cells is used. These are open ended on the bottom, but flat sheets close those cells that incorporate services. Sometimes, cells are enlarged laterally to transmit air for air conditioning.

Two outstanding advantages of cellular floors are rapidity of erection and ease with which present and future connections can be made to telephone, computer, light, and power wiring, each cell serving as a conduit. Each deck unit becomes a working platform immediately on erection, thus enabling the several finishing trades to follow right behind the steel erector.

Although the cost of the steel deck system may be larger than that of other floor systems, the cost differential can be narrowed to competitive position when equal consideration for electrical facility is imposed on the other systems; e.g., the addition of 4 in of concrete fill to cover embedded electrical conduit on top of a concrete flat-slab floor.

In earlier floors of this type, the steel decking was assumed to be structurally independent. In that case, the concrete fill served only to provide fire resistance and a level floor. Most modern deckings, however, are bonded or locked to the concrete, so that the two materials act as a unit in composite construction. Usually, only top-quality stone concrete (ASTM C33 aggregates) is used, although lightweight concrete made with ASTM C330 aggregates is an acceptable alternative.

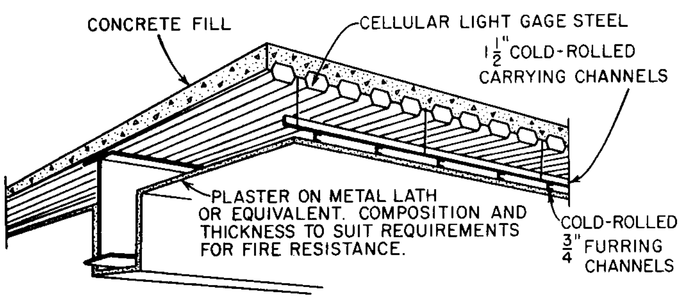


FIGURE 7.24 Cellular-steel floor construction.

Usage of cellular deck in composite construction is facilitated by economical attachment of shear connectors to both the decking and underlying beams. For example, when welded studs are used, a welding gun automatically fastens the studs through two layers of hot-dipped galvanized decking to the unpainted top flanges of the steel beams. This construction is similar to composite concrete-steel beams (Art. 7.13.3)

The total floor weight of cellular steel construction is low, comparable to open-web steel joists. Weight savings of about 50% are obtained in comparison with all-concrete floors; 30% savings in overall weight of the building. However, a big cost saving in a high-labor-rate area results from elimination of costly formwork needed for concrete slabs, since the steel decking serves as the form.

Fire resistance for any required rating is contributed by the fill on top of the cells and by the ceiling below (Fig. 7.24). Generally, removable panels for which no fire rating is claimed are preferred for suspended ceilings. In this case, fireproofing materials are applied directly to the underside of the metal deck and all exposed surfaces of steel floor beams, a technique often called spray-on fireproofing.

7.13.3 Composite Concrete-Steel Beams

In composite construction, the structural concrete slab is made to assist the steel beams in supporting loads. Hence, the concrete must be bonded to the steel to ensure shear transfer. When the steel beams are completely encased in the concrete, the natural bond is considered capable of resisting horizontal shear. But that bond generally is disregarded when only the top flange is in contact with the concrete. Consequently, shear connectors are used to resist the horizontal shear. Commonly used connectors are welded studs, hooked or headed, and short lengths of channels.

Usually, composite construction is most efficient for heavy loading, long spans, large beam spacing, and restricted depths. Because the concrete serves much like a cover plate, lighter steel beams may be used for given loads, and deflections are smaller than for noncomposite construction.

7.13.4 Concrete-Pan Floors

Concrete floors cast on removable metal forms or pans, which form the joists, are frequently used with steel girders. Since the joists span the distance between columns, intermediate steel beams are not needed (Fig. 7.25). This floor generally weighs less than the arch system (reinforced concrete slabs on widely spaced beams), but still considerably more than the lightest types.

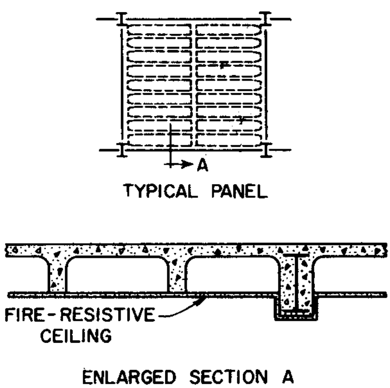


FIGURE 7.25 Concrete joist floor.

There are a number of variations of the concrete-joist system, such as the "gird" or "waffle" system, where the floor is cast on small, square, removable pans, or domes, so that the finished product becomes a two-way joist system. Other systems employ permanent filler blocks—usually a lightweight tile. Some of these variations fall in the heaviest floor classification; also the majority require substantial forms and shoring.

7.14 ROOF FRAMING SYSTEMS

These are similar in many respects to the floor types, discussed in Arts. 7.13 and 7.13.1. In fact, for flat-top tier buildings, the roof may be just another floor. However, when roof loads are smaller than floor loads, as is usually the case, it may be economical to lighten the roof construction. For example, steel joists may be spaced farther apart. Where roof decking is used, the spacing of the joists is determined by the load-carrying ability of the applied decking and of the joists.

Most of the considerations discussed for floors in Art. 7.13 also are applicable to roof systems. In addition, however, due thought should be given to weather resistance, heat conductance and insulation, moisture absorption and vapor barriers, and especially to maintenance.

Many roof systems are distinctive as compared with the floor types; for example, the corrugated sheet-metal roofing commonly employed on many types of industrial or mill buildings. The sheets rest on small beams, channels, or joists, called purlins, which in turn are supported by trusses. Similar members on the sidewalls are called girts.

DESIGN OF MEMBERS

In proportioning of members, designers should investigate one or more or a combination of five basic stress or strength conditions: axial tension, axial compression, bending, shearing, and member element crippling. Other conditions that should be investigated under special conditions are local buckling, excessive deflection, torsion and fatigue. Until the early 1990s, such analyses were based on allowable stress design (ASD). More recently, a method known as load and resistance factor design (LRFD) has come into use because it permits a more rational design. It takes into account the probability of loading conditions and statistical variations in the strength, or resistance capability, of members and connection materials.

The use of LRFD design procedures will result in a savings of material, generally in the range of 15 to 20%, and on major structures, some elements may show a savings of up to 25%. Such weight savings generally means a lesser cost for the structural steel. However, except for major structures, when serviceability factors such as deflection and vibration are considered in the proportioning of the individual members, the nominal savings of LRFD procedures versus ASD procedures is more likely to be approximately 5%.

7.15 BASES FOR ASD AND LRFD

ASD is based on elastic theory. Design limits the maximum unit stress a member is permitted to bear under service loads to a level determined by a judgmental, but experience-based, safety factor. Building codes establish allowable unit stresses, which are normally related to the minimum yield stress for each grade of steel.

Plastic design is based on the ultimate strength of members. A safety factor, comparable to that established for elastic design, is applied to the design load to determine the ultimate-load capacity required of a member.

LRFD is based on the concept that no applicable limit state should be exceeded when the structure, or any member or element, is subject to appropriate combinations of factored loads.

A **limit state** is defined as a condition in which a structure or structural component becomes unfit for further structural service. A structural member can have several limit states.

Strength limit states relate to maximum load-carrying capacity.

Serviceability limit states relate to performance under normal service conditions with respect to such factors as deflection and vibration.

Design specifications establish **load factors** to be applied to each type of service load, such as dead, live, and wind loads, the values of the factors depending on the specific combination of loads to be imposed on a structure (Art. 5.1.3).

The AISC ''Load and Resistance Factor Design Specification for Structural Steel Buildings'' requires that structures be designed so that, under the most critical combination of factored loads, the design strength of the structures or their individual elements is not exceeded. For each strength limit state, the design strength is the product of the nominal strength and a resistance factor ϕ, given in the specification. Derived with the use of probability theory, ϕ provides an extra margin of safety for the limit state being investigated. Nominal strength of a member depends on its geometric properties, yield or ultimate strength, and type of loading to be resisted, such as tension, compression, or flexure.

The AISC LRFD specification permits structural analysis based on either elastic or plastic behavior. Elastic theory is most commonly used. Where plastic theory is used for complex structures, all possible mechanisms that may form in the structure should be investigated. The collapse mechanism is the one that requires the lightest load for collapse to occur.

Numerous computer programs for analysis and design of members or structures are available. If data input describing the structure and loading are accurate, most of these programs yield a quick and accurate design. For complex structures, care should be taken in use of computer programs to check the results to ensure that they are logical, since a critical input error may not be easily found. If a program can produce a plot of the configuration of the loaded structure based on the data input, the plot should be used as a check, inasmuch as omission of a member or other errors in connectivity data can be readily discerned from the plot.

(''Plastic Design in Steel—A Guide and Commentary,'' M & R No. 41, American Society of Civil Engineers.)

7.16 DESIGN AIDS AND REFERENCES

Design procedures using either the ASD or the LRFD specifications require the use of many numerical values which represent the section properties of the individual shapes or plates under consideration. Several publications in the form of handbooks have been developed by the industry to provide the designer with this and other useful information. In addition, many steel producers publish handbooks which

TABLE 7.9 Handbooks and Design Guides

Publisher	Title	Content
American Institute of Steel Construction (AISC) One East Wacker Drive Chicago, IL 60601-2001	ASD Manual of Steel Construction	Design specification Section properties Dimensional data Design aids
	LRFD Manual of Steel Construction—Vol. I	Design specification Section properties Dimensional data Design aids
	LRFD Manual of Steel Construction—Vol. I	Design aids Suggested design details Dimensional data
	Design Guide No. 1 Column Base Plates	Theory and examples of base plate and anchor bolt design
	Design Guide No. 2 Steel and Composite Beams with Web Openings	Theory and examples of web penetration design
	Design Guide No. 3 Serviceability Design Considerations for Low-Rise Buildings	Design criteria
	Design Guide No. 5 Design of Low- and Medium-Rise Steel Buildings	Synopsis of design criteria and design details
	Design Guide No. 7 Industrial Buildings: Roof to Column Anchorage	Industrial building design
	Seismic Provisions for Structural Steel Buildings	Design criteria Design details
American Institute of Steel Construction (address above) or Steel Tube Institute of North America 8500 Station Street Suite 270 Mentor, OH 44060	Hollow Structural Sections Connections Manual	Section properties Dimensional data Fabrication Detail design criteria
Steel Joist Institute (SJI) 3127 10th Ave. No. Ext. No. Myrtle Beach, SC 29577-6760	Standard Specifications and Load Tables, Open-Web Steel Joist	Dimensional data Load capacity

provide section property values for the products they market. Table 7.9 lists several handbooks widely used by design professionals, as well as other design guides which address specific design features.

7.17 SERVICEABILITY CRITERIA

Experienced designers are aware of certain practical limitations on the size of individual members. Flexural members which have marginal or too shallow a depth can cause deflections that can damage other building elements, as well as cause vibrations under moving loads that disturb a building's occupants. Almost all building code leave stiffness design criteria to the designer. Experienced designers have found that to specify limits for all possible variations loads, occupancies, and types of construction is impracticable.

This section outlines various criteria, originally based on experience but up-dated on the basis of testing, which the designer can incorporate to develop a serviceable design. The ASD specification (Table 7.1) restricts the maximum live-load deflection of beams and girders supporting plaster ceilings to $1/360$ of the span. This requirement is not applicable to less rigid construction details. The AISC LRFD specification contains no numerical limits for serviceability criteria. Table 7.10 may be used to set limits on deflections of flexural members frequently encountered in building design.

Minimum Depth-Span Ratios. Also, as a guide, Table 7.10 lists suggested minimum depth-span ratios for various loading conditions and yield strengths of steel up to $F_y = 50.0$ ksi. These may be useful for estimating or making an initial design selection. Since maximum deflection is a straight-line function of maximum bend-

TABLE 7.10 Guide to Selection of Beam Depths and Deflection Limits

	Yield stress F_y, ksi				Maximum stress, ksi	
	36.0	42.0	45.0	50.0	$0.60F_y$	$0.66F_y$
Specific beam condition	Minimum depth-span ratio				Maximum ratio of deflection to span	
Heavy shock or vibration	$\frac{1}{18}$	$\frac{1}{15.5}$	$\frac{1}{14.5}$	$\frac{1}{13}$	$\frac{1}{357}$	$\frac{1}{324}$
Heavy pedestrian traffic	$\frac{1}{20}$	$\frac{1}{17}$	$\frac{1}{16}$	$\frac{1}{14.5}$	$\frac{1}{320}$	$\frac{1}{291}$
Normal loading	$\frac{1}{22}$	$\frac{1}{19}$	$\frac{1}{18}$	$\frac{1}{16}$	$\frac{1}{290}$	$\frac{1}{264}$
Beams for flat roofs*	$\frac{1}{25}$	$\frac{1}{21.5}$	$\frac{1}{20}$	$\frac{1}{18}$	$\frac{1}{258}$	$\frac{1}{232}$
Roof purlins, except for flat roofs*	$\frac{1}{28}$	$\frac{1}{24}$	$\frac{1}{22}$	$\frac{1}{20}$	$\frac{1}{232}$	$\frac{1}{210}$

*Investigate for stability against ponding.

ing stress f_b and therefore is nearly proportional to F_y, a beam of steel with F_y = 100.0 ksi would have to be twice the depth of a beam of steel with F_y = 50.0 ksi when each is stressed to allowable values and has the same maximum deflection.

Vibration of large floor areas that are usually free of physical dampeners, such as partitions, may occur in buildings such as shopping centers and department stores, where pedestrian traffic is heavy. The minimum depth-span ratios in Table 7.10 suggested for "heavy pedestrian traffic" are intended to provide an acceptable solution.

One rule of thumb that may be used to determine beam depth quickly is to choose a depth, in, not less than 1.5% of F_y times the span, ft. Thus, for A36 steel depth, in, should be at least half the span, ft.

Ponding. Beams for flat roofs may require a special investigation to assure stability against water accumulation, commonly called ponding, unless there is adequate provision for drainage during heavy rainfall. The AISC specification gives these criteria for stable roofs:

$$C_p + 0.9C_s \leq 0.25 \tag{7.1}$$

$$I_d \geq \frac{25S^4}{10^6} \tag{7.2}$$

where $C_p = 32L_sL_p^4/10^7\ I_p$
$C_s = 32SL_s^4/10^7\ I_s$
L_p = column spacing in direction of girder, ft (length of primary members)
L_s = column spacing perpendicular to direction of girder, ft (length of secondary member)
S = spacing of secondary members, ft
I_p = moment of inertia for primary members, in^4
I_s = moment of inertia for secondary members, in^4. Where a steel deck is supported on primary members, it is considered the secondary member. Use $0.85I_s$ for joists and trusses
I_d = moment of inertia of a steel deck supported on secondary members, in^4/ft

Uniform-Load Deflections. For the common case of a uniformly loaded simple beam loaded to the maximum allowable bending stress, the deflection in inches may be computed from

$$\delta = \frac{5}{24}\,\frac{F_b l}{Ed/l} \tag{7.3}$$

where F_b = the allowable bending stress, ksi
l = the span, in
E = 29,000 ksi
d/l = the depth-span ratio

Drift. AISC Design Guide No. 3 (Table 7.9) suggests that the lateral deflection of a building frame (drift) be limited to a value which does not damage other structural or architectural components when subject to a 10-year recurrence interval wind pressure. The 10-year wind pressure can be reasonably estimated at 75% of the 50-year wind pressure.

Camber. Trusses of 80-ft or greater span should be cambered to offset dead-load deflections. Crane girders 75 ft or more in span should be cambered for deflection under dead load plus one-half live load.

7.18 TENSION MEMBERS

These are proportioned so that their gross and net areas are large enough to resist imposed loads. The criteria for determining the net area of a tension member with bolt holes is the same for allowable stress design and load-and-resistance-factor design. In determination of net area, the width of a bolt hole should be taken $1/16$ in larger than the nominal dimension of the hole normal to the direction of applied stress. Although the gross section for a tension member without holes should be taken normal to the direction of applied stress, the net section for a tension member with holes should be chosen as the one with the smallest area that passes through any chain of holes across the width of the member. Thus, the net section may pass through a chain of holes lying in a plane normal to the direction of applied stress or through holes along a diagonal of zigzag line.

Net section for a member with a chain of holes extending along a diagonal or zigzag line is the product of the net width and thickness. To determine net width, deduct from the gross width the sum of the diameters of all the holes in the chain, then add, for each gage space in the chain, the quantity

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

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 member is obtained from that chain with the least net width.

When a member axially stressed in tension is subjected to nonuniform transfer of load because of connections through bolts to only some of the elements of the cross section, as in the case of a W, M, or S shape connected solely by bolts through the flanges, the net area should be reduced as follows: 10% if the flange width is at least two-thirds the beam depth and at least three fasteners lie along the line of stress; 10% also for structural tees cut from such shapes; 15% for any of the preceding shapes that do not meet those criteria and for other shapes that have at least three fasteners in line of stress; and 25% for all members with only two fasteners in the line of stress.

7.18.1 ASD of Tension Members

Unit tensile stress F_t on the gross area should not exceed $0.60F_y$, where F_y is the minimum yield stress of the steel member (see Table 7.11). Nor should F_t exceed $0.50F_u$, where F_u is the minimum tensile strength of the steel member, when the allowable stress is applied to the net area of a member connected with fasteners requiring holes. However, if the fastener is a large pin, as used to connect eyebars, pin plates, etc., F_t is limited to $0.45F_y$ on the net area. Therefore, for the popular

TABLE 7.11 Tension on Gross Area

Allowable tensile stress (ASD)		Unit design tensile strength (LRFD)	
F_y, ksi	F_t, ksi	F_y, ksi	$\phi P_n / A_g$
36	21.6	36	32.4
42	25.2	42	37.8
45	27.0	45	30.5
50	30.0	50	45.0
55	33.0	55	49.5
60	36.0	60	54.0

A36 steel, the allowable tension stresses for gross and net areas are 22.0 and 29.0 ksi, respectively, and in the case of pin plates, 16.2 ksi.

7.18.2 LRFD of Tension Members

Design tensile strength ϕP_n, kips, of the gross area A_g, in^2, should not exceed $0.90F_y$, where F_y is the minimum yield stress of the steel (Table 7.9) and $P_n = A_g F_y$. Nor should the design tensile strength ϕP_n, kips, exceed $0.75F_u$ on the net area A_e, in^2, of the member. Other criteria control the design tensile strength of pin-connected members. (Refer to the AISC specification for LRFD.)

7.19 COLUMNS AND OTHER COMPRESSION MEMBERS

The principal factors governing the proportioning of members carrying compressive forces are overall column buckling, local buckling, and gross section area. The effect of overall column buckling depends on the slenderness ratio Kl/r, where Kl is the effective length, in, of the column, l is the unbraced length, and r is the least radius of gyration, in, of the cross section. The effect of local buckling depends on the width-thickness ratios of the individual elements of the member cross section.

W shapes with depths of 8, 10, 12, and 14 in are most commonly used for building columns and other compression members. For unbraced compression members, the most efficient shape is one where the value of r_y with respect to the minor axis approaches the value of r_x with respect to the major axis.

When built-up sections are used as compression members, the element joining the principal load-carrying elements, such as lacing bars, should have a shear capacity of at least 2% of the axial load.

7.19.1 Effective Column Length

Proper application of the column capacity formulas for ASD or LRFD depends on judicious selection of K. This term is defined as the ratio of effective column length to actual unbraced length.

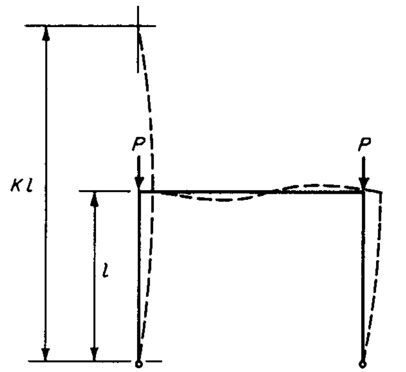
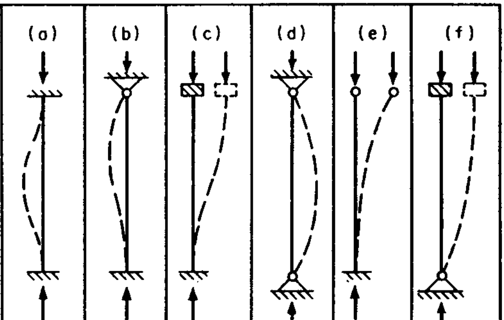


FIGURE 7.26 Configurations of members of a rigid frame caused by sidesway.

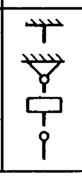
For a pin-ended column with translation of the ends prevented, $K = 1$. But in general, K may be greater or less than unity. For example, consider the columns in the frame in Fig. 7.26. They are dependent entirely on their own stiffness for stability against sidesway. If enough axial load is applied to them, their effective length will exceed their actual length. But if the frame were braced to prevent sidesway, the effective length would be less than the actual length because of the resistance to end rotation provided by the girder.

Theoretical values of K for six idealized conditions in which joint rotation and translation are either fully realized or nonexistent are given in Fig. 7.27. Also noted are values recommended by the Column Research Council for use in design when these conditions are approximated. Since joint fixity is seldom fully achieved, slightly higher design values than theoretical are given for fixed-end columns.

Specifications do not provide criteria for sidesway resistance under vertical loading, because it is impossible to evaluate accurately the contribution to stiffness of the various components of a building. Instead, specifications cite the general conditions that have proven to be adequate.



	(a)	(b)	(c)	(d)	(e)	(f)
BUCKLED SHAPE OF COLUMN IS SHOWN BY DASHED LINE						
THEORETICAL K VALUE	0.5	0.7	1.0	1.0	2.0	2.0
RECOMMENDED DESIGN VALUE WHEN IDEAL CONDITIONS ARE APPROXIMATED	0.65	0.80	1.2	1.0	2.10	2.0



END CONDITION CODE	
	ROTATION FIXED AND TRANSLATION FIXED
	ROTATION FREE AND TRANSLATION FIXED
	ROTATION FIXED AND TRANSLATION FREE
	ROTATION FREE AND TRANSLATION FREE

FIGURE 7.27 Values of effective column length K for idealized conditions.

Constructions that inhibit sidesway in building frames include substantial masonry walls, interior shear walls; braced towers and shafts; floors and roofs providing diaphragm action—that is, stiff enough to brace the columns to shear walls or bracing systems; frames designed primarily to resist large side loadings or to limit horizontal deflection; and diagonal X bracing in the planes of the frames. Compression members in trusses are considered to be restrained against translation at connections. Generally, for all these constructions, K may be taken as unity, but a value less than one is permitted if proven by analysis.

When resistance to sidesway depends solely on the stiffness of the frames; for example, in tier buildings with light curtain walls or with wide column spacing, and with no diagonal bracing systems or shear walls, the designer may use any of several proposed rational methods for determining K. A quick estimate, however, can be made by using the alignment chart in an AISC "Manual of Steel Construction." The effective length Kl of compression members, in such cases, should not be less than the actual unbraced length.

7.19.2 ASD of Compression Members

The allowable compressive stress on the gross section of axially loaded members is given by formulas determined by the effective slenderness ratios Kl/r of the members. A critical value, designated C_c, occurs at the slenderness ratio corresponding to the maximum stress for elastic buckling failure (Table 7.12). This is illustrated in Fig. 7.28. An important fact to note: when Kl/r exceeds $C_c = 126.1$, the allowable compressive stress is the same for A36 and all higher-strength steels.

$$C_c = \sqrt{2\pi;s2E/F_y} \tag{7.4}$$

where E = modulus of elasticity of the steel = 29,000 ksi and F_y = specified minimum yield stress, ksi.

When Kl/r for any unbraced segments is less than C_c, the allowable compressive stress, ksi is

$$F_a = \frac{[1 - (Kl/r)^2/2C_c^2]F_y}{\text{FS}} \tag{7.5}$$

where FS is the safety factor, which varies from 1.67 when $Kl/r = 0$ to 1.92 when $Kl/r = C_c$.

TABLE 7.12 Slenderness Ratio at Maximum Stress for Elastic Buckling Failure

F_y, ksi	C_c	F_y, ksi	C_c
36.0	126.1	60.0	97.7
42.0	166.7	65.0	93.8
45.0	112.8	90.0	79.8
50.0	107.0	100.0	75.7
55.0	102.0		

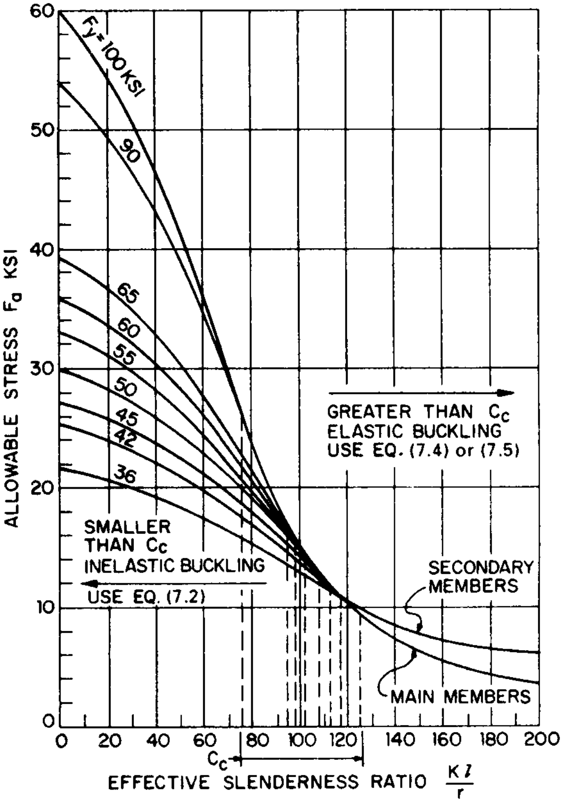


FIGURE 7.28 Allowable stresses for axial compression.

$$\text{FS} = \frac{5}{3} + \frac{3Kl/r}{8C_c} - \frac{(Kl/r)^3}{8C_c^3} \tag{7.6}$$

When Kl/r is greater than C_c:

$$F_a = \frac{12\pi^2 E}{23(Kl/r)^2} = \frac{149{,}000}{(Kl/r)^2} \tag{7.7}$$

This is the Euler column formula for elastic buckling with a constant safety factor of 1.92 applied.

Increased stresses are permitted for bracing and secondary members with l/r greater than 120. (K is taken as unity.) For such members, the allowable compressive stress is

$$F_{as} = \frac{F_a}{1.6 - l/200r} \tag{7.8}$$

where F_a is given by Eq. (7.5) or (7.6). The higher stress is justified by the relative

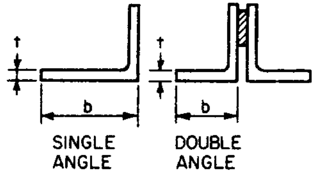
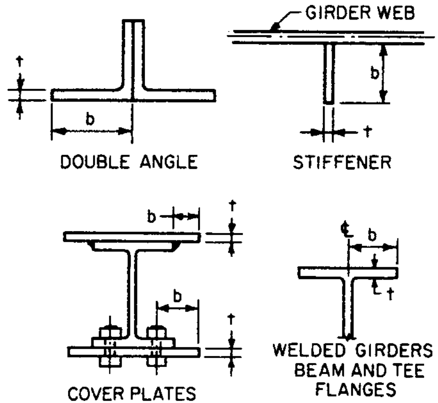
	MAXIMUM $\frac{b}{t}$									
	$F_y =$	36.0	42.0	45.0	50.0	55.0	60.0	65.0	90.0	100.0
	$\frac{76}{\sqrt{F_y}}$	12.7	11.7	11.3	10.7	10.2	9.8	9.4	8.0	7.6
	$\frac{95}{\sqrt{F_y}}$	15.8	14.7	14.2	13.4	12.8	12.3	11.8	10.0	9.5

FIGURE 7.29 Maximum width-thickness ratios for allowable stress design of compression members.

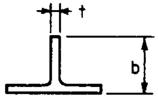
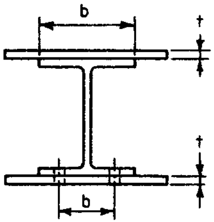
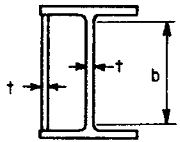
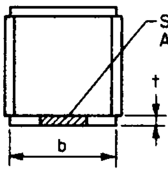
 STEM OF TEES		$\frac{127}{\sqrt{F_y}}$	21.2	19.6	18.9	18.0	17.1	16.4	15.8	13.4	12.7
 COVER PLATES	 COLUMN WEBS AND DIAPHRAGMS	$\frac{253}{\sqrt{F_y}}$	42.2	39.0	37.7	35.8	34.1	32.7	31.4	26.7	25.3
 SERIES OF ACCESS HOLES BOX MEMBERS	SOLID	$\frac{238}{\sqrt{F_y}}$	39.7	36.7	35.5	33.7	32.1	30.7	29.5	25.1	23.8
	PERFORATED	$\frac{317}{\sqrt{F_y}}$	52.8	48.9	47.3	44.8	42.7	40.9	39.3	33.4	31.7

FIGURE 7.29 Maximum width-thickness ratios for allowable stress design of compression members. (*Continued*)

unimportance of these members and the greater restraint likely at their end connections. The full unbraced length should always be used for l.

Tables giving allowable stresses for the entire range of Kl/r appear in the AISC ASD "Manual of Steel Construction." Approximate values may be obtained from Fig. 7.28. Allowable stresses are based on certain minimum sizes of structural members and their elements that make possible full development of strength before premature buckling occurs. The higher the allowable stresses the more stringent must be the dimensional restrictions to preclude buckling or excessive deflections.

The AISC ASD specification for structural steel buildings limits the effective slenderness ratio Kl/r to 200 for columns, struts, and truss members, where K is the ratio of effective length to actual unbraced length l, and r is the least radius of gyration.

A practical rule also establishes limiting slenderness ratios l/r for tension members:

For main members	240
For bracing and secondary members	300

But this does not apply to rods or other tension members that are drawn up tight (prestressed) during erection. The purpose of the rule is to avoid objectionable slapping or vibration in long, slender members.

The AISC ASD specification also specifies several restricting ratios for compression members. One set applies to projecting elements subjected to axial compression or compression due to bending. Another set applies to compression elements supported along two edges.

Figure 7.29 lists maximum width-thickness ratios, b/t, for commonly used elements and grades of steel. Tests show that when b/t of elements normal to the direction of compressive stress does not exceed these limits, the member may be stressed close to the yield stress without failure by local buckling. Because the allowable stress increases with F_y, the specified yield stress of the steel, width-thickness ratios are less for higher-strength steels.

These b/t ratios should not be confused with the width-thickness ratios described in Art. 7.20. There, more restrictive conditions are set in defining compact sections qualified for higher allowable stresses.

7.19.3 LRFD of Compression Members

When the elements of the cross section of a compression member have width-thickness ratios that do not exceed the limits tabulated in Table 7.13, the design compressive strength is $\phi_c P_n$. The resistance factor ϕ_c should be taken as 0.85. The nominal strength is given by $P_n = A_g F_{cr}$, where A_g is the cross-sectional area, in^2, and F_{cr} is the critical compressive stress, ksi. Formulas for F_{ct} are based on a parameter λ_c.

$$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{Kl}{r}\sqrt{\frac{F_y}{286{,}220}} \tag{7.9}$$

where E = modulus of elasticity, ksi = 29,000 ksi. For $\lambda_c \leq 1.5$,

$$F_{cr} = 0.658^{\lambda_c^2} F_y \tag{7.10}$$

For $\lambda_c > 1.5$,

TABLE 7.13 Limiting Width-Thickness Ratios for LRFD of Columns

Compression elements	Width thickness ratio	Limiting width-thickness ratio λ_r		
		General	A36 steel	A50 steel
Flanges of W and other I shapes and channels; outstanding legs of pairs of angles in continuous contact	b/t	$95/\sqrt{F_y}$	15.8	13.4
Flanges of square and rectangular box sections; flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$238/\sqrt{F_y - F_r^*}$	47.7 (rolled) 53.9 (welded)	37.6 (rolled) 41.1 (welded)
Legs of single angle struts and double angle struts with separators; unstiffened elements (i.e., supported along one edge)	b/t	$76/\sqrt{F_y}$	12.7	10.7
Stems of tees	d/t	$127/\sqrt{F_y}$	21.2	18.0
All other stiffened elements (elements supported along two edges)	b/t h_c/t_w	$253/\sqrt{F_y}$	42.2	35.8

$^*F_y$ = compressive residual stress in flange: 10 ksi for rolled shapes, 16.5 ksi for welded sections.

$$F_{cr} = (0.877/\lambda_c^2)F_y \tag{7.11}$$

Computations can be simplified by use of column load tables in the AISC LRFD "Steel Construction Manual."

For design of columns with elements having width-thickness ratios exceeding the limits in Table 7.13, refer to the AISC LRFD specification.

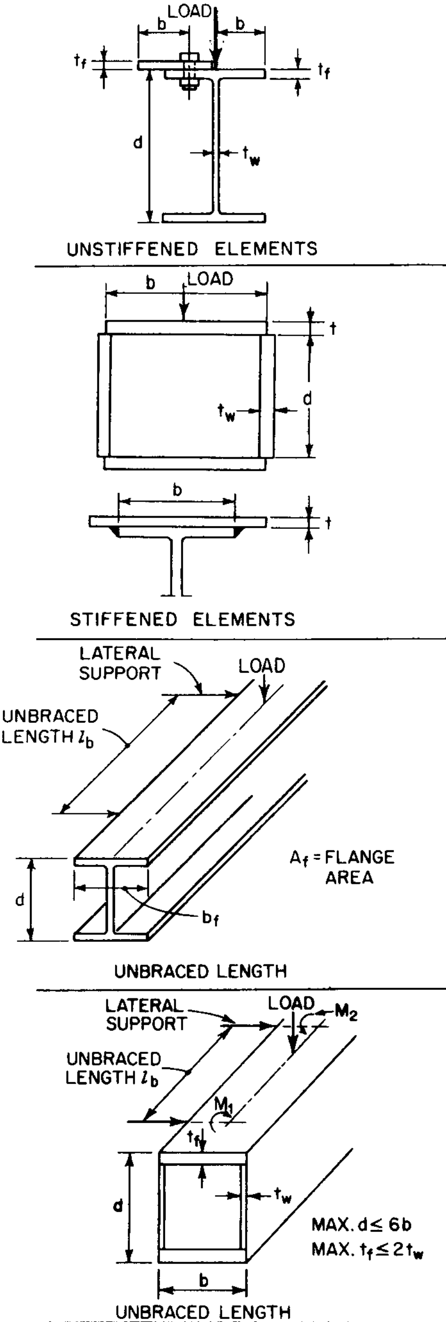
(T. V. Galambos, "Guide to Design Criteria for Metal Compression Members," 4th ed., John Wiley & Sons, Inc., New York.)

7.20 BEAMS AND OTHER FLEXURAL MEMBERS

The capacity of members subject to bending depends on the cross-section geometry, AISC ASD and LRFD procedures incorporate the concept of compact and non-compact sections.

7.20.1 ASD of Flexural Members

Beams classified as compact are allowed a bending stress, ksi, $F_b = 0.66F_y$ for the extreme surfaces in both tension and compression, where F_y, is the specified yield stress, ksi. Such members have an axis of symmetry in the plane of loading, their compression flange is adequately braced to prevent lateral displacement, and they develop their full plastic moment (section modulus times yield stress) before buckling.



F_y =	36.0
$\frac{b}{t_f} \leq \frac{65.0}{\sqrt{F_y}}$	10.8
$\frac{d}{t_w} \leq \frac{640}{\sqrt{F_y}}\left(1 - 3.74\frac{f_a}{F_y}\right)$ FOR $f_a/F_y \leq 0.16$	$107 - 11.1 f_a$
$\frac{d}{t_w} \leq \frac{257}{\sqrt{F_y}}$ FOR $f_a/F_y > 0.16$	42.8
$\frac{b}{t} \leq \frac{190}{\sqrt{F_y}}$	31.7
$l_b \leq \frac{76.0 b_f}{\sqrt{F_y}}$	$12.7 b_f$
$l_b \leq \frac{20{,}000 A_f}{dF_y}$	$556 \frac{A_f}{d}$
$l_b \leq \frac{b}{F_y}\left(1{,}950 + 1{,}200\frac{M_1}{M_2}\right)$	—
BUT NEED NOT BE LESS THAN $1{,}200\frac{b}{F_y}$	33.3b

FIGURE 7.30 Requirements for laterally supported compact beam sections in ASD.

42.0	45.0	50.0	55.0	60.0	65.0
10.0	9.7	9.2	8.8	8.4	8.1
$98.8 - 8.8 f_a$	$95.4 - 7.9 f_a$	$90.5 - 6.8 f_a$	$86.3 - 5.9 f_a$	$82.6 - 5.2 f_a$	$79.4 - 4.6 f_a$
39.7	38.3	36.3	34.7	33.2	31.9
29.3	28.3	26.9	25.6	24.5	23.6
$11.7 b_f$	$11.3 b_f$	$10.7 b_f$	$10.2 b_f$	$9.8 b_f$	$9.4 b_f$
$476 \frac{A_f}{d}$	$444 \frac{A_f}{d}$	$400 \frac{A_f}{d}$	$364 \frac{A_f}{d}$	$333 \frac{A_f}{d}$	$308 \frac{A_f}{d}$
—	—	—	—	—	
28.6b	26.7b	24.0b	21.8b	20.0b	18.5b

FIGURE 7.30 Requirements for laterally supported compact beam sections in ASD. (*Continued*)

Compactness Requirements. To qualify as compact, members must meet the following conditions:

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

2. The width-thickness ratio of unstiffened projecting elements of the compression flange must not exceed $65.0/\sqrt{F_y}$. For computation of this ratio, with b equals one-half the full flange width of I-shaped sections, or the distance from the free edge to the first row of fasteners (or welds) for projecting plates, or the full width of legs of angles, flanges of zees of channels, or tee stems.

3. The web depth-thickness ratio d/t_w must not exceed $640(1 - 3.74_a/F_y)/\sqrt{F}$ when f_a, the computed axial stress, is equal to or less than $0.16F_y$, or $257/\sqrt{F_y}$ when $f_a > 0.16F_y$.

4. The width-thickness ratio of stiffened compression flange plates in box sections and that part of the cover plates for beams and built-up members that is included between longitudinal lines of bolts or welds must not exceed $190/\sqrt{F_y}$.

5. For the compression flange of members not box shaped to be considered supported, unbraced length between lateral supports should not exceed $76.0b_f/\sqrt{F_y}$ or 20,000 $A_f/F_y d$, where b_f is the flange width, A_f the flange area, and d the web depth.

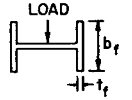
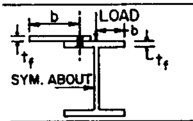
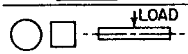
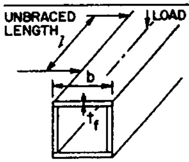
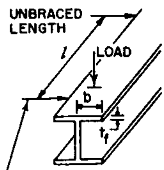
6. The unbraced length for rectangular box-shaped members with depth not more than 6 times the width and with flange thickness not more than 2 times the web thickness must not exceed $(1950 + 1200\ M_1/M_2)b/F_y$. The unbraced length in such cases, however, need not be less than $1200b/F_y$. M_1 is the smaller and M_2 the larger of bending moments at points of lateral support.

7. The diameter-thickness ratio of hollow circular steel sections must not exceed $300/F_y$.

Allowable Bending Stresses for Compact Beams. Most sections used in building framing, including practically all rolled W shapes of A36 steel and most of those with $F_y = 50$ ksi, comply with the preceding requirements for compactness, as illustrated in Fig. 7.30. Such sections, therefore, are designed with $F_b = 0.66F_y$. Excluded from qualifying are hybrid girders, tapered girders, and sections made from A514 steel.

Braced sections that meet the requirements for compactness, and are continuous over their supports or rigidly framed to columns, are also permitted a redistribution of the design moments. Negative gravity-load moments at supports may be reduced 10%. But then, the maximum positive moment must be increased by 1% of the average negative moments. This moment redistribution does not apply to cantilevers, hybrid girders, or members of the A514 steel.

Allowable Bending Stresses for Noncompact Beams. Many other beam-type members, including nearly compact sections that do not meet all seven requirements, are accorded allowable bending stresses, some higher and some considerably lower than $0.66F_y$, depending on such conditions as shape factor, direction of loading, inherent resistance to torsion or buckling, and external lateral support. The common conditions and applicable allowable bending stresses are summarized in Fig. 7.31. In the formulas,

SECTION	REQUIREMENTS	F_b
 SYMMETRICAL ABOUT BOTH AXES	MEETS COMPACTNESS 1 & 2*	0.75 F_y FOR BOTH TENSION AND COMPRESSION
	MEETS COMPACTNESS 1*† $\frac{65.0}{\sqrt{F_y}} < \frac{b_f}{2t_f} < \frac{95.0}{\sqrt{F_y}}$	$F_y\left[1.075 - 0.005\left(\frac{b_f}{2t_f}\right)\sqrt{F_y}\right]$
	MEETS COMPACTNESS 1,3,4,5*† $\frac{65.0}{\sqrt{F_y}} < \frac{b}{t_f} < \frac{95.0}{\sqrt{F_y}}$	$F_y\left[0.79 - 0.002\left(\frac{b}{t_f}\right)\sqrt{F_y}\right]$
	SOLID SQUARES AND ROUNDS SOLID FLAT ELEMENTS BENT ABOUT WEAKER AXIS	0.75 F_y FOR BOTH TENSION AND COMPRESSION
 LATERAL SUPPORT	MEETS COMPACTNESS 1,3,6 $\frac{b}{t_f} \le \frac{238}{\sqrt{F_y}}$ OR $\le \frac{317}{\sqrt{F_y}}$ IF PERFORATED WITH ACCESS HOLES MIN. $l = \frac{1{,}200b}{F_y}$	0.60 F_y FOR BOTH TENSION AND COMPRESSION
TENSION FLANGE FLEXURAL SECTIONS NOT COVERED ABOVE		0.60 F_y
 LATERAL SUPPORT FOR CHANNELS USE ONLY →	$\frac{b}{t_f} \le \frac{95.0}{\sqrt{F_y}}$	FOR COMPRESSION, USE LARGER OF FOLLOWING:
	WHEN $\frac{319}{\sqrt{F_y}} \le \frac{l}{r_t} \le \frac{714}{\sqrt{F_y}}$	$F_y\left[\frac{2}{3} - \frac{F_y(l/r_t)^2}{1{,}530{,}000}\right] \le 0.60F_y$
	WHEN $\frac{l}{r_t} \ge \frac{714}{\sqrt{F_y}}$	$\frac{170{,}000}{(l/r_t)^2} \le 0.60F_y$
	OR FOR SOLID RECTANGULAR FLANGE WITH AREA NOT LESS THAN TENSION FLANGE†	$\frac{12{,}000A_f}{ld} \le 0.60F_y$
COMPRESSION FLANGE FOR FLEXURAL SECTIONS NOT COVERED ABOVE	$\frac{b}{t_f} \le \frac{95.0}{\sqrt{F_y}}$ $l \le \frac{152b}{\sqrt{F_y}}$ FOR MAJOR-AXIS BENDING	0.60 F_y

* EXCEPT A514 STEEL † EXCEPT HYBRID GIRDERS

NOTE: FOR I-SHAPED SECTIONS $b_f = 2b$

FIGURE 7.31 Allowable bending stresses for sections not qualifying as compact.

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

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

A_f = area of the compression flange, in^2

The allowable bending stresses F_b, ksi, for values often used for various grades of steel are listed in Table 7.14.

TABLE 7.14 Allowable Bending Stresses, ksi

F_y	$0.60F_y$	$0.66F_y$	$0.75F_y$
36.0	22.0	24.0	27.0
42.0	25.2	27.7	31.5
45.0	27.0	29.7	33.8
50.0	30.0	33.0	37.5
55.0	33.0	36.3	41.3
60.0	36.0	39.6	45.0
65.0	39.0	42.9	48.8

Lateral Support of Beams. In computation of allowable bending stresses in compression for beams with distance between lateral supports exceeding requirements, a range sometimes called laterally unsupported, the AISC ASD formulas contain a moment factor C_b in recognition of the beneficial effect of internal moments, both in magnitude and direction, at the points of support. For the purpose of this summary, however, the moment factor has been taken as unity and the formulas simplified in Fig. 7.31. The formulas are exact for the case in which the bending moment at any point within an unbraced length is larger than that at both ends of this length. They are conservative for all other cases. Where more refined values are desired, see Art. 7.20.2 or refer to the AISC ASD specification for structural steel buildings.

Limits on Beam Width-Thickness Ratios. For flexural members in which the width-thickness ratios of compression elements exceed the limits given in Fig. 7.31 and which are usually lightly stressed, appropriate allowable bending stresses are suggested in "Slender Compression Elements," Appendix C, AISC specification.

For additional discussion of lateral support, see Art. 7.12.2. Also, addition information on width-thickness ratios of compression elements is given in Fig. 7.29.

ASD for Shear in Flexural Members. The shear strength of a flexural member may be computed by dividing the total shear force at a section by the web area, the product of the web thickness and overall member depth. Whereas flexural strength normally controls selection of rolled shapes, shear strength can be critical when the web has cutouts or holes that reduce the net web area of when a short-span beam carries a large concentrated load. Also, in built-up members, such as plate girders or rigid frame elements, shear often controls web thickness.

The web depth-thickness ratio permitted without stiffeners, $h/t \leq 380/\sqrt{F_y}$ for ASD and $h/t \leq 418/\sqrt{F_y}$ for LRFD, is satisfied by the W shapes of A36 steel. Furthermore, only the lightest one or two W sections in each depth fail to satisfy these criteria for 50-ksi material.

For members with $h/t \leq 380/\sqrt{F_y}$, the unit shear stress on the gross section should not be greater than $F_v = 0.40F_y$, where F_y is the minimum yield point of the web steel ksi (Table 7.15). Members with higher h/t ratios require stiffeners (see Art. 7.21.1).

Beams with web angle or shear-bar end connections and a coped top flange should be checked for shear on the critical plane through the holes in the web. In this case, the allowable unit shear stress is $F_v = 0.30F_u$, where F_u is the minimum tensile strength of the steel, ksi.

TABLE 7.15 Allowable Shear on Gross Area, ksi

For ASD when $h/t \le 380/\sqrt{F_y}$		For LRFD when $h/t \le 418/\sqrt{F_y}$	
F_y	F_u	F_y	ϕV_n
36.0	14.5	36.0	19.4
42.0	17.0	42.0	22.7
45.0	18.0	45.0	24.3
50.0	20.0	50.0	27.0
55.0	22.0	55.0	29.7
60.0	24.0	60.0	32.4

A special case occurs when a web lies in a plane common to intersecting members; for example, the knee of a rigid frame. Then, shear stresses generally are high. Such webs, in elastic design, should be reinforced when the web thickness is less than $32M/A_{bc}F_y$, where M is the algebraic sum of clockwise and counterclockwise moments (in ft-kips) applied on opposite sides of the connection boundary, and A_{bc} is the planar area of the connection web, in^2 (approximately the product of the depth of the member introducing the moment and the depth of the intersecting member). In plastic design, this thickness is determined from $23M_p/A_{bc}F_y$, where M_p is the plastic moment, or M times a load factor of 1.70. In this case, the total web shear produced by the factored loading should not exceed the web area (depth times thickness) capacity in shear. Otherwise, the web must be reinforced with diagonal stiffeners or a doubler plate.

For deep girder webs, allowable shear is reduced. The reduction depends on the ratio of clear web depth between flanges to web thickness and an aspect ratio of stiffener spacing to web depth. In practice, this reduction does not apply when the ratio of web depth to thickness is less than $380/\sqrt{F_y}$.

7.20.2 LRFD of Flexural Members

The AISC LRFD specification for structural steel buildings permits plastic analysis for steels with yield stress not exceeding 65 ksi. Negative moments induced by gravity loading may be reduced 10% for compact beams, if the positive moments are increased by 10% of the average of the negative moments.

Design strength in bending of flexural members is defined as $\phi_b M_n$, where the resistance $\phi_b = 0.90$ and M_n is the nominal flexural strength. M_n depends on several factors, including the geometry of the section, the unbraced length of the compression flange, and properties of the steel. Beams may be compact, noncompact, or slender-element sections. For compact beams, the AISC specification sets limits on the width-thickness ratios of section elements to restrict local buckling. These limits are listed in Table 7.16.

For a compact section bent about the major axis, the unbraced length L_b of the compression flange where plastic hinges may form at failure may not exceed L_{pd} given by Eqs. (7.12) and (7.13). For beams bent about the minor axis and square and circular beams, L_b is not restricted for plastic analysis.

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

TABLE 7.16 Limiting Width-Thickness Ratios for LRFD of Beams

Beam element	Width-thickness ratio	Limiting width-thickness ratio, λ_p General	A36 steel	A50 steel
Flanges of W and other I shapes and channels	b/t	$65/\sqrt{F_y}$	10.8	9.2
Flanges of square and rectangular box sections; flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$190/\sqrt{F_y}$	31.7	26.9
Webs in flexural compression	h_c/t_w	$640/\sqrt{F_y}$	106.7	90.5

$$L_{pd} = \frac{3600 + 2200(M_1/M_p)}{F_{yc}} r_y \tag{7.12}$$

where F_{yc} = minimum yield stress, ksi, of compression flange
M_1 = smaller of the moments, in-kips, at the end of the unbraced length of the beam.
M_p = plastic moment, in-kips
r_y = radius of gyration, in, about minor axis

For homogeneous sections, $M_p = F_y Z$, where Z is the plastic section modulus, in³. (For hybrid girders, Z may be computed from the fully plastic distribution.) M_1/M_p is positive for beams with reverse curvature, negative for single curvature.

For solid rectangular bars and symmetric box beams,

$$L_{pd} = \frac{5000 + 3000(M_1/M_p)}{F_y} r_y \tag{7.13}$$

The flexural design strength $0.90M_n$ 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. For compact sections bent about the major axis, M_n depends on the following unbraced lengths:

L_b = 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
= $3750(r_y/M_p)/\sqrt{JA}$ for box beams and solid rectangular bars
F_{yf} = flange yield stress, ksi
J = torsional constant, in^4 (see AISC LRFD "Manual of Steel Construction")
A = cross-sectional area, in^2
L_r = limiting laterally unbraced length, in, for inelastic lateral buckling

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

$$L_r = \frac{r_y X_1}{(F_{yw} - F_r)} \sqrt{1 + \sqrt{1 + X_2(F_{yw} - F_r)^2}} \tag{7.14}$$

where 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 = 29,000 ksi
G = shear modulus of elasticity = 11,200 ksi
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

7.20.3 Limit-State Moments

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

$$M_r = (F_{yw} - F_r)S_x \tag{7.15}$$

For compact beams with $L_b \le L_r$, bent about the major axis,

$$M_n = C_b \left[M_p - (M_p - M_r) \frac{L_b - L_p}{L_r - L_p} \right] \le M_p \tag{7.16}$$

where $C_b = 1.75 + 1.05(M_1/M_2) + 0.3(M_1/M_2)^2 \le 2.3$, where M_1 is the smaller and M_2 the larger end moment in the unbraced segment of the beam; M_1/M_2 is positive for reverse curvature
= 1.0 for unbraced cantilevers and beams with moment over much of the unbraced segment equal to or greater than the larger of the segment end moments (see T. V. Galambos "Guide to Stability Design Criteria for Metal Structures," 4th ed., John Wiley & Sons, Inc., New York, for use of larger values of C_b)

For solid rectangular bars bent about the major axis,

$$L_r = 57{,}000(r_y/M_r)\sqrt{JA} \tag{7.17}$$

and the limiting buckling moment is given by

$$M_r = F_y S_x \tag{7.18}$$

For symmetric box sections loaded in the plane of symmetry and bent about the major axis, M_r should be determined from Eq. (7.15) and L_r from Eq. (7.17).

For compact beams with $L_b > L_r$, bent about the major axis,

$$M_n = M_{cr} \le C_b M_r \tag{7.19}$$

where M_{cr} = critical elastic moments, kip-in. For shapes to which Eq. (17.11) applies,

$$M_{\mathrm{cr}} = C_b\,(\pi/L_b)\,\sqrt{EI_yGJ + I_yC_w(\pi E/L_b)^2} \tag{7.20}$$

For solid rectangular bars and symmetric box sections,

$$M_{\mathrm{cr}} = 57{,}000C_b\,\sqrt{JA}/(L_b/r_y) \tag{7.21}$$

Noncompact Beams. The nominal flexural strength M_n for noncompact beams is the least value determined from the limit states of

1. Lateral-torsional buckling (LTB)
2. Flange local buckling (FLB)
3. Web local buckling (WLB)

The AISC LRFD specification for structural steel buildings presents formulas for determining limit-state moments. In most cases, LRFD computations for flexural members can be simplified by use of tables in the AISC "Manual of Steel Construction—LRFD." See also Art. 7.21.

LRFD for Shear in Flexural Members. The design shear strength is $\phi_V V_n$, where $\phi_V = 0.90$, and for rolled shapes and built-up members without stiffeners is governed by the web depth-thickness ratio. The design shear strength may be computed from

$$\phi V_n = 0.90 \times 0.6F_yA_w = 0.54F_yA_w \qquad \frac{h}{t} \le \frac{418}{\sqrt{F_y}} \tag{7.22}$$

$$\phi V_n = 0.90 \times 0.6\,\frac{418/\sqrt{F_y}}{b/t} = 0.54F_yA_w\,\frac{418/\sqrt{F_y}}{h/t} \qquad \frac{418}{\sqrt{F_y}} < \frac{h}{t} \le \frac{523}{\sqrt{F_y}} \tag{7.23}$$

$$\phi V_n = 0.90 \times A_w\,\frac{132{,}000}{(h/t)^2} = \frac{119{,}000}{(h/t)^2} \qquad \frac{h}{t} > \frac{523}{\sqrt{F_y}} \tag{7.24}$$

where V_n = nominal shear strength, kips
A_w = area of the web, in^2 = dt
d = overall depth, in
t = thickness of web, in
h = the following web dimensions, in: clear distance between fillets for rolled shapes; clear distance between flanges for welded sections
F_y = specified minimum yield stress, ksi, of web steel

See also Art. 7.21.2.

7.20.4 Beam Penetrations

Certain designs, especially buildings with minimal floor-to-floor heights, require penetrations, or openings, in the webs of beams to permit the routing of ductwork or piping. In general, such penetrations can safely be made at locations where the beam shear loading is low if the penetration height is limited to half the beam depth. The central span region of a beam carrying a uniform load is an example of a typical situation. The penetration should be centered on the mutual axis of the member and all re-entrant corners should have a generous radius.

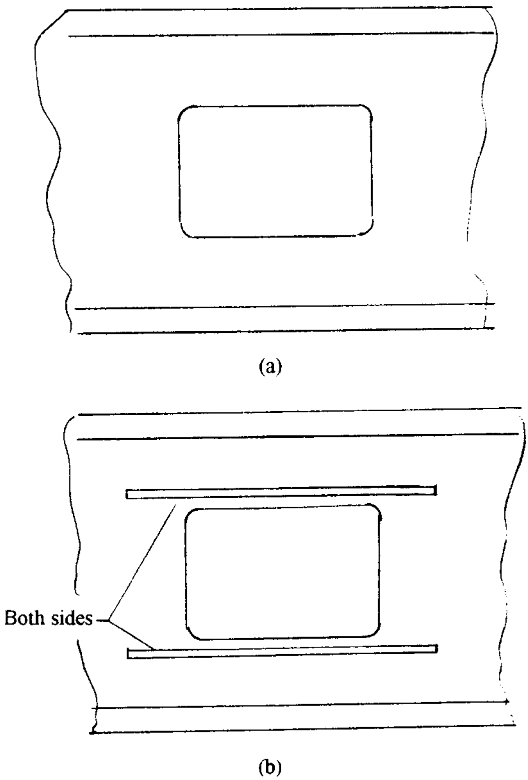


FIGURE 7.32 Typical beam penetrations.

When penetrations are necessary at locations with higher shear loadings, it may be necessary to reinforce the web with longitudinal stiffeners. Figure 7.32 shows typical configurations with (*a*) being unreinforced and (*b*) reinforced. Design of such reinforcement is done by considering a free-body of the section of the beam containing the penetration. Further information on beam penetrations is available in AISC Design Guide No. 2 (Table 7.9).

7.21 PLATE GIRDERS

Plate girders may have either a box or an I shape. Main components are plates or plates and angles, arranged so that the cross section is either singly or doubly symmetrical. Generally, the elements are connected by continuous fillet welds. In existing construction, the connection may have been made with rivets or bolts through plates and angles. Fig. 7.33 depicts typical I-shape girders.

Plate girders are commonly used for long spans where they cost less than rolled W shapes or where members are required with greater depths or thinner webs than those available with rolled W shapes. The AISC LRFD "Specification for Structural Steel for Buildings" distinguished between a plate girder and a beam in that a plate

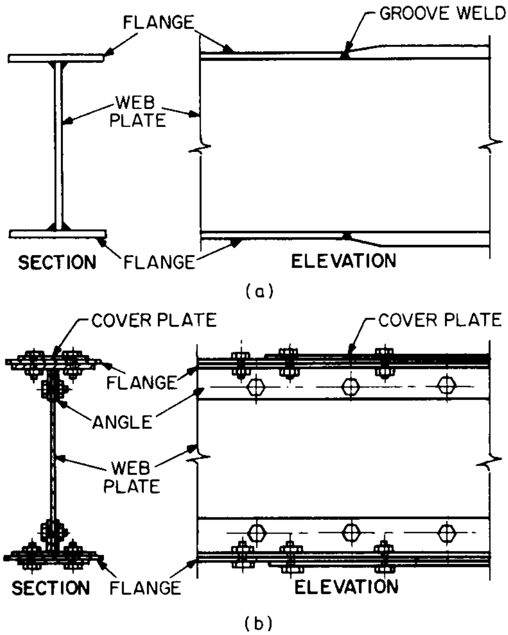


FIGURE 7.33 Plate girders: (*a*) welded (*b*) bolted.

girder has web stiffeners or a web with $h_c/t_w > 970/\sqrt{F_y}$, or both, where h_c is twice the distance from the neutral axis to (1) the inside face of the compression flange when it is welded to the web or (2) the nearest line of fasteners to the compression flange when the web-flange connection is bolted.

7.21.1 ASD Procedure for Plate Girders

Allowable stresses for tension, compression, bending, and shear are the same for plate girders as those given in Arts. 7.18 to 7.20, except where stiffeners are used. But reductions in allowable stress are required under some conditions, and there are limitations on the proportions of girder components.

Web Depth-Thickness Limits. The ratio of the clear distance h between flanges, in, to web thickness t, in, is limited by

$$\frac{h}{t} \leq \frac{14{,}000}{\sqrt{F_y(F_y + 16.5)}} \tag{7.25}$$

where F_y is the specified yield stress of the compression flange steel, ksi (Table 7.17). When, however, transverse stiffeners are provided at spacings not exceeding 1.5 times the girder depth, the limit on h/t is increased to

$$\frac{h}{t} \leq \frac{2{,}000}{\sqrt{F_y}} \tag{7.26}$$

TABLE 7.17 Limiting Depth-Thickness Ratios for ASD of Plate-Girder Webs

F_y, ksi	h/t Eq. (7.25)	h/t Eq. (7.26)	F_y, ksi	h/t Eq. (7.25)	h/t Eq. (7.26)
36.0	322	333	60.0	207	258
42.0	282	309	65.0	192	248
45.0	266	298	90.0	143	211
50.0	243	283	100.0	130	200
55.0	223	270			

General Design Method. Plate girders may be proportioned to resist bending on the assumption that the moment of inertia of the gross cross section is effective. No deductions need be made for fastener holes, unless the holes reduce the gross area of either flange by more than 15%. When they do, the excess should be deducted.

Hybrid girders, which have higher-strength steel in the flanges than in the web, may also be proportioned by the moment of inertia of the gross section when they are not subjected to an axial force greater than 15% of the product of yield stress of the flange steel and the area of the gross section. At any given section, the flanges must have the same cross-sectional area and be made of the same grade of steel.

The allowable compressing bending stress F_b for plate girders must be reduced from that given in Art. 7.20 where h/t exceeds $760/\sqrt{F_b}$. For greater values of this ratio, the allowable compressive bending stress, except for hybrid girders, becomes

$$F_b' \leq F_b \left[1 - 0.0005 \frac{A_w}{A_f} \left(\frac{h}{t} - \frac{760}{\sqrt{F_b}} \right) \right] \tag{7.27}$$

where A_w = the web area, in^2 and A_f = the compression flange area, in^2.

For hybrid girders, not only is the allowable compressive bending stress limited to that given by Eq. (7.24), but also the maximum stress in either flange may not exceed

$$F_b' = F_b \left[\frac{12 + (A_w/A_f)(3\alpha - \alpha^3)}{12 + 2(A_w/A_f)} \right] \tag{7.28}$$

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

Flange Limitations. The projecting elements of the compression flange must comply with the limitations for b/t given in Art. 7.21. The area of cover plates, where used, should not exceed 0.70 times the total flange area. Partial-length cover plates (Fig. 7.33*b*) should extend beyond the theoretical cutoff point a sufficient distance to develop their share of bending stresses at the cutoff point. Preferably for welded-plate girders, the flange should consist of a series of plates, which may differ in thickness and width, joined end to end with complete-penetration groove welds (Fig. 7.33*a*).

Bearing Stiffeners. These are required on girder webs at unframed ends. They may also be needed at concentrated loads, including supports. Set in pairs, bearing stiffeners may be angles or plates placed on opposite sides of the web, usually normal to the bending axis. Angles are attached with one leg against the web. Plates are welded perpendicular to the web. The stiffeners should have close bearing against the flanges through which they receive their loads, and should extend nearly to the edges of the flanges.

These stiffeners are designed as columns, with allowable stresses as given in Art. 7.19. The column section is assumed to consist of a pair of stiffeners and a strip of girder web with width 25 times web thickness for interior stiffeners and 12 times web thickness at ends. In computing the effective slenderness ratio Kl/r, use an effective length Kl of at least 0.75 the length of the stiffeners.

Intermediate Stiffeners. With properly spaced transverse stiffeners strong enough to act as compression members, a plate-girder web can carry loads far in excess of its buckling load. The girders acts, in effect, like a Pratt truss, with the stiffeners as struts and the web forming fields of diagonal tension. The following formulas for stiffeners are based on this behavior. Like bearing stiffeners, intermediate stiffeners are placed to project normal to the web and the bending axis, but they may consist of a single angle or plate. They may be stopped short of the tension flange a distance up to 4 times the web thickness. If the compression flange is a rectangular plate, single stiffeners must be attached to it to prevent the plate from twisting. When lateral bracing is attached to stiffeners, they must be connected to the compression flange to transmit at least 1% of the total flange stress, except when the flange consists only of angles.

The total shear force, kips, divided by the web area, in², for any panel between stiffeners should not exceed the allowable shear F_v given by Eqs. (7.29*a*) and (7.29*b*).

Except for hybrid girders, when C_v is less than unity:

$$F_v = \frac{F_y}{2.89}\left[C_v = \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^3}}\right] \leq 0.4F_y \tag{7.29a}$$

For hybrid girders or when C_v is more than unity or when intermediate stiffeners are omitted:

$$F_v = \frac{F_y C_v}{2.89} \leq 0.4F_y \tag{7.29b}$$

where a = clear distance between transverse stiffeners, in

h = clear distance between flanges within an unstiffened segment, in

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

$= \dfrac{190}{h/t}\sqrt{\dfrac{k}{F_v}}$ when C_v is more than 0.8

t = web thickness, in

$k = 5.34 + 4(a/h)^2$ when $a/h > 1$

$= 4 + 5.34(a/h)^2$ when $a/h < 1$

Stiffeners for an end panel or for any panel containing large holes and for adjacent panels should be so spaced that the largest average web shear f_v in the panel does not exceed the allowable shear given in Eq. (7.29*b*).

Intermediate stiffeners are not required when h/t is less than 260 and f_v is less than the allowable stress given by Eq. (7.29*b*). When these criteria are not satisfied, stiffeners should be spaced so that the applicable allowable shear, Eq. (7.29*a*) or

(7.29b), is not exceeded, and in addition, so that a/h is not more than $[260/(h/t)]^2$ or 3.

Solution of the preceding formulas for stiffener spacing requires assumptions of dimensions and trials. The calculations can be facilitated by using tables in the AISC "Manual of Steel Construction." Also, Fig. 7.34 permits rapid selection of the most efficient stiffener arrangement, for webs of A36 steel. Similar charts can be drawn for other steels.

If the tension field concept is to apply to plate girder design, care is necessary to ensure that the intermediate stiffeners function as struts. When these stiffeners are spaced to satisfy Eq. (7.29a), their gross area, in^2 (total area if in pairs) should be at least

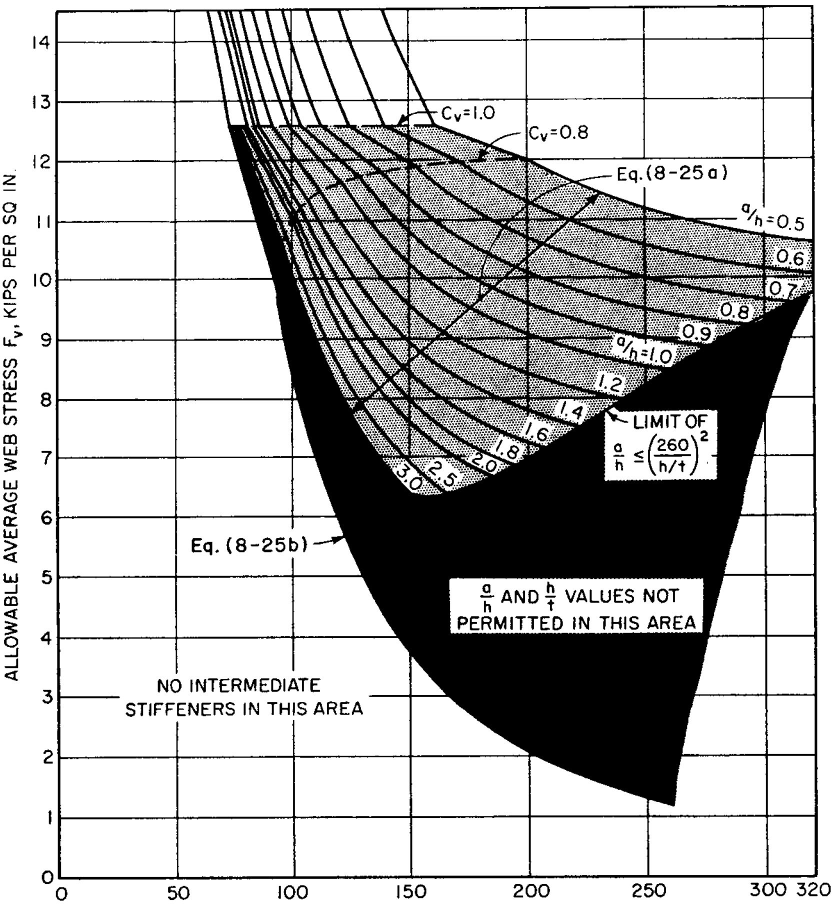


FIGURE 7.34 Plate girder web design. Chart shows the relationship between allowable shears in web of plate girders, with yield stress F_y = 36 ksi, and web thickness, distance between flanges, and stiffener spacing.

$$A_{st} = \frac{1 - C_v}{2} \left[\frac{a}{h} - \frac{(a/h)^2}{\sqrt{1 + (a/h)^2}} \right] YDht \tag{7.30}$$

where Y = ratio of yield stress of web steel to yield stress of stiffener steel
D = 1.0 for stiffeners in pairs
= 1.8 for single-angle stiffeners
= 2.4 for single-plate stiffeners

When the greatest shear stress f_v in a panel is less than F_v determined from Eq. (7.29*a*), the gross area of the stiffeners may be reduced in the ratio f_v/F_v.

The moment of inertia of a stiffener or pair of stiffeners, about the web axis, should be at least $(h/50)^4$. The connection of these stiffeners to the web should be capable of developing shear, in kips per lineal inch of single stiffener or pair, of at least

$$f_{vs} = h \sqrt{\left(\frac{F_{yw}}{340}\right)^3} \tag{7.31}$$

where F_{yw} is the yield stress of the web steel (Table 7.18). This shear also may be reduced in the ratio f_v/F_v as above.

TABLE 7.18 Required Shear Capacity of Intermediate-Stiffener Connections to Girder Web

F_{yw}, ksi	f_{vg}, kips per lin in	F_{yw}, ksi	f_{vg}, kips per lin in
36.0	0.034*h*	60.0	0.074*h*
42.0	0.043*h*	65.0	0.084*h*
45.0	0.048*h*	90.0	0.136*h*
50.0	0.056*h*	100.0	0.160*h*
55.0	0.065*h*		

Combined Stresses in Web. A check should be made for combined shear and bending in the web where the tensile bending stress is approximately equal to the maximum permissible. When f_v, the shear force at the section divided by the web area, is greater than that permitted by Eq. (7.29*a*), the tensile bending stress in the web should be limited to no more than $0.6F_{yw}$ or $F_{yw}(0.825 - 0.375f_v/F_v)$, where F_v is the allowable web shear given by Eq. (7.29*a*). For girders with steel flanges and webs with F_y exceeding 65 ksi, when the flange bending stress is more than 75% of the allowable, the allowable shear stress in the web should not exceed that given by Eq. (7.22).

Also, the compressive stresses in the web should be checked (see Art. 7.22).

7.21.2 LRFD Procedure for Plate Girders

Plate girders are normally proportioned to resist bending on the assumption that the moment of inertia of the gross section is effective. The web must be proportioned such that the maximum web depth-thickness ratio h/t does not exceed h/t given by (7.32) or (7.33), whichever is applicable.

If $a/h \leq 1.5$,

$$\frac{h}{t} \leq \frac{2000}{\sqrt{F_{yf}}} \tag{7.32}$$

If $a/h > 1.5$,

$$\frac{h}{t} \leq \frac{14{,}000}{\sqrt{F_{yf}(F_{yf} + F_r)}} \tag{7.33}$$

where a = clear distance between transverse stiffeners, in
t = web thickness, in
F_{yf} = specified minimum yield stress of steel, ksi
F_r = compressive residual stress in flange = 16.5 ksi for plate girders

Web stiffeners are frequently required to achieve an economical design. However, web stiffeners are not required if $h/t < 260$ and adequate shear strength is provided by the web. The criteria for the design of plate girders are given in the AISC LRFD Specification.

Design Flexural Strength. The design flexural strength is $\phi_b M_n$, where $\phi_b = 0.90$. If $h_c/t \leq 970\sqrt{F_y}$, determine the nominal flexural strength as indicated in Art. 7.15, for either compact or noncompact shapes. If $h_c/t > 970\sqrt{F_y}$, M_n is governed by the limit states of tension-flange yielding or compression-flange buckling.

The design strength is the smaller of the values of $\phi_b M_n$ for yielding of the tension flange, which is

$$\phi_b M_n = 0.90 S_{xt} R_{PG} R_e F_{yt} \tag{7.34}$$

and for buckling of the compression flange, which is

$$\phi_b M_n = 0.90 S_{xc} R_{PG} R_e F_{\text{cr}} \tag{7.35}$$

where R_{PG} = plate-girder bending-strength reduction factor
= $1 - 0.0005 a_r (h_c/t - 970/\sqrt{F_{\text{cr}}}) \leq 1.0$
R_e = hybrid girder factor
= $1 - 0.1(1.3 + a_r)(0.81 - m) \leq 1.0$
= 1 for nonhybrid girders
a_r = ratio of web area to compression-flange area
m = ratio of web yield stress to flange yield stress or to F_{cr}
F_{cr} = critical compression-flange stress, ksi
F_{yt} = yield stress of tension flange, ksi
S_{xt} = section modulus, in^3, with respect to the tension flange
S_{xc} = section modulus, in^3, with respect to the compression flange

The critical stress F_{cr} is different for different limit states. Its value is computed from the values of parameters that depend on the type of limit state: plate girder coefficient C_{PG}, slenderness parameter λ, limiting slenderness parameter λ_p for a compact element, and limiting slenderness parameter λ_r for a noncompact element. Thus, F_{cr} may be computed from one of Eqs. (7.34) to (7.36) for the limit states of lateral-torsional buckling and flange local buckling. The limit state of local buckling of web does not apply.

$$F_{cr} = F_{yf} \qquad \lambda \leq \lambda_p \tag{7.36}$$

$$F_{cr} = C_b F_{yf}\left[1 - \frac{1}{2}\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \leq F_{yf} \qquad \lambda_p < \lambda \leq \lambda_r \tag{7.37}$$

$$F_{cr} = C_{PG}/\lambda^2 \qquad \lambda > \lambda_r \tag{7.38}$$

where F_{yf} = specified minimum flange yield stress, ksi
C_b = bending coefficient dependent on moment gradient
= $1.75 + 1.05(M_1/M_2) + 0.3(M_1/M_2)^2$ for lateral-torsional buckling
= 1 for flange local buckling
C_{PG} = $286{,}000/C_b$ for lateral torsional buckling
= 11,200 for flange local buckling
λ = L_b/r_T for lateral-torsional buckling
= $b_f/2t_f$ for flange local buckling
L_b = laterally unbraced length of girder, in
r_T = radius of gyration, in, of compression flange plus one-sixth the web
b_f = flange width, in
t_f = flange thickness, in
λ_p = $300/\sqrt{F_{yf}}$ for lateral-torsional buckling
= $65/\sqrt{F_{yf}}$ for flange local buckling
λ_r = $756/\sqrt{F_{yf}}$ for lateral-torsional buckling
= $150/\sqrt{F_{yf}}$ for flange local buckling

Design Shear Strength. This is given by $\phi_v V_n$, where $\phi_v = 0.90$. With tension-field action, in which the web is permitted to buckle due to diagonal compression and the web carries stresses in diagonal tension in the panels between vertical stiffeners, the design shear strength is larger than when such action is not permitted.

Tension-field action is not allowed for end panels in nonhybrid plate girders, for all panels in hybrid girders and plate girders with tapered webs, and for panels in which the ratio of panel width to depth a/h exceeds 3.0 or $[260(h/t)]^2$, where t is the web thickness. For these conditions, the design shear strength is given by

$$\phi_n V_n = 0.90 \times 0.6 A_w F_{yw} C_v = 0.54 A_w F_{yw} C_v \tag{7.39}$$

where A_w = web area, in^2
F_{yw} = specified web yield stress, ksi
C_v = ratio of critical web stress, in the linear buckling theory, to the shear yield stress of the web steel

For tension-field action, the design shear strength depends on the ratio of panel width to depth a/h. For $h/t \leq 187\sqrt{k/F_{yw}}$,

$$\phi_v V_n = 0.54 A_w F_{yw} \tag{7.40}$$

For $h/t > 187\sqrt{k/F_{yw}}$,

$$\phi_v V_n = 0.54 A_w F_{yw} \left(C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}} \right) \tag{7.41}$$

where k = web buckling coefficient
$= 5 \quad \text{if } a/h > 0.3 \text{ or } a/h > [260/(h/t)]^2$
$= 5 + 5/(a/h)^2 \quad \text{otherwise}$

$$C_v = \frac{187\sqrt{k/F_{yw}}}{h/t} \quad \text{when } 187\sqrt{k/F_{yw}} \leq h/t \leq 234\sqrt{k/F_{yw}}$$

$$= \frac{44{,}000}{(h/t)^2} \frac{k}{F_y} \quad \text{when } h/t > 234\sqrt{k/F_{yw}}$$

Web Stiffeners. Transverse stiffeners are required if the web shear strength without stiffeners is inadequate, if $h/t > 418/\sqrt{F_{yw}}$, or if h/t does not meet the requirements of Eqs. (7.30) and (7.31). Where stiffeners are required, the spacing of stiffeners should be close enough to maintain the shear within allowable limits. Also, the moment of inertia I_{st}, in^4, of a transverse stiffener should be at least that computed from

$$I_{st} = at^3 j \tag{7.42}$$

where $j = 2.5/(a/h)^2 - 2$.

The moment of inertia for a pair of stiffeners should be taken about an axis through the center of the web. For a single stiffener, I_{st} should be taken about the web face in contact with the stiffener. In addition, for design for tension-field action, the stiffener area A_{st}, in^2, should be at least that computed from

$$A_{st} = \frac{F_{yw}}{F_{ys}} \left[0.15 Dht(1 - C_v) \frac{V_u}{\phi_u V_n} - 18t^2 \right] \geq 0 \tag{7.43}$$

where F_{ys} = specified yield stress of stiffener, ksi
V_u = required shear strength at stiffener, kips, calculated for the factored loads
D = 1.0 for a pair of stiffeners
= 1.8 for a single-angle stiffener
= 2.4 for a single-plate stiffener

Bending and Shear Interaction. Plate girders should also be proportioned to satisfy Eq. (7.43) if they are designed for tension-field action, stiffeners are required, and V_u/M_u lies between 60 and 133% of V_n/M_n.

$$\frac{M_u}{M_n} + 0.625 \frac{V_u}{V_n} \leq 1.24 \tag{7.44}$$

where M_n = design flexural strength
M_u = required flexural strength calculated for the factored loads but may not exceed $0.90M_n$
V_n = design shear strength
V_u = required shear strength calculated for the factored loads but may not exceed $0.90V_n$

7.22 WEB OR FLANGE LOAD-BEARING STIFFENERS

Members subject to large concentrated loads within their length or large end reactions should be proportioned so that the forces on the web or flange cannot cause local failure or the webs or flanges should be stiffened to carry the concentrated loads. Both ASD and LRFD procedures include design criteria.

7.22.1 ASD for Load-Bearing Stiffeners

Webs of rolled beams and plate girders should be so proportioned that the compressive stress, ksi, at the web toe of the fillets does not exceed

$$F_a = 0.66F_y \tag{7.45}$$

where F_y = specified minimum yield stress, ksi.

Web failure probably would be in the form of buckling caused by concentrated loading, either at an interior load or at the supports. The capacity of the web to transmit the forces safely should be checked.

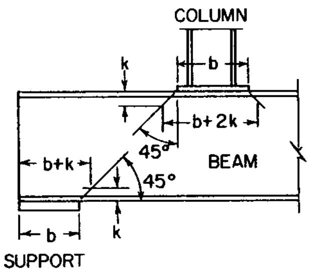


FIGURE 7.35 Web crippling in a simple beam. The critical web section is assumed to occur at the fillet.

Load Distribution. Loads are resisted not only by the part of the web directly under them but also by the parts immediately adjacent. A 45° distribution usually is assumed, as indicated in Fig. 7.35 for two common conditions. The distance k is determined by the point where the fillet of the flange joins the web; it is tabulated in the beam tables of the AISC "Manual of Steel Construction." F_a is applicable to the horizontal web strip of length $b + k$ at the end support or $b + 2k$ under an interior load. Bearing stiffeners are required when F_a is exceeded.

Bearing atop Webs. The sum of the compression stresses resulting from loads bearing directly on or through a flange on the compression edge of a plate-girder web should not exceed the following:

When the flange is restrained against rotation, the allowable compressive stress, ksi, is

$$F_a = \left[5.5 + \frac{4}{(a/h)^2}\right]\frac{10{,}000}{(h/t)^2} \tag{7.46}$$

When the flange is not restrained against rotation,

$$F_a = \left[2 + \frac{4}{(a/h)^2}\right]\frac{10{,}000}{(h/t)^2} \tag{7.47}$$

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

The load may be considered distributed over a web length equal to the panel length (distance between vertical stiffeners) or girder depth, whichever is less.

Web Stiffeners on Columns. The web of a column may also be subject to crippling by the thrust from the compression flange of a rigidly connected beam, as shown at point a in Fig. 7.36. Likewise, to ensure full development of the beam plastic moment, the column flange opposite the tensile thrust at point b may require stiffening.

When stiffeners having a combined cross-sectional area A_{st}, in^2, are required on the column whenever A_{st} computed from Eq. (7.48) is positive

$$A_{st} = \frac{P - F_{yc}t(t_b + 5k)}{F_{ys}} \tag{7.48}$$

where t = thickness of column web, in
t_b = thickness, in, of beam flange delivering concentrated load
F_{yc} = column steel yield stress, ksi
F_{ys} = stiffener steel yield stress, ksi
P = computed force delivered by beam flange or connection plate multiplied by 5⁄3 when force is a result of dead and live loads, or by 4⁄3 when it is a result of wind or earthquake forces, kips
k = distance from face of column to edge of fillet on rolled sections (use equivalent for welded sections)

Regardless of the preceding requirement, a single or double stiffener is needed opposite the compression force delivered to the column at point a when

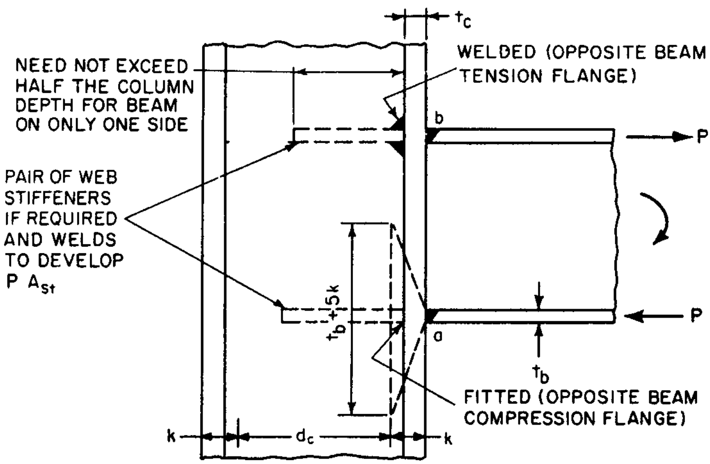


FIGURE 7.36 Web crippling in a column at a welded joint with a beam.

$$d_c > \frac{4100\, t^3 \sqrt{F_{yc}}}{P} \tag{7.49}$$

where d_c = clear distance, in, between column flanges (clear of fillets). Also, a pair of stiffeners is needed opposite the tension force at point b when

$$t_f < 0.4 \sqrt{\frac{P}{F_{yc}}} \tag{7.50}$$

where t_f = thickness of column flange, in.

The thickness of a stiffener should not be less than one-half the thickness of the beam flange or plate that delivers force P to the column. Stiffener width should not be less than one-third of the flange or plate width.

7.22.2 LRFD for Load-Bearing Stiffeners

Six limit states should be considered at locations where a large concentrated force acting on a member introduces high local stresses. These limit states are local flange bending, local web yielding, web crippling, sidesway web buckling, compression buckling of the web, and high shear in column web panels. Detailed requirements for determining the design strength for each of these limit states are contained in the AISC LRFD "Specification for Structural Steel for Buildings."

When web stiffeners are required to prevent web crippling or compression buckling of the web, they are designed as columns with an effective length of Kl = 0.75h, where h is the clear distance between flanges. The effective cross section is the area of the stiffeners plus 25t for interior stiffeners for 12t for stiffeners at the end of a member, where t is the web thickness.

7.23 BEARING

For bearing on finished surfaces, such as milled ends and ends of fitted bearing stiffeners, or on the projected area of pins in finished holes, the allowable stress in ASD is

$$F_p = 0.90F_y \tag{7.51}$$

where F_y is the specified minimum yield stress of the steel, ksi. When the parts in contact have different yield stresses, use the smaller F_y (Table 7.19).

The allowable bearing stress on expansion rollers and rockers, kip/in, is

$$F_p = \frac{F_y - 13}{20}\, 0.66d \tag{7.52}$$

where d is the diameter of roller or rocker, in (Table 7.20).

Allowable bearing stresses on masonry usually can be obtained from a local or state building code, whichever governs. In the absence of such regulations, however, the values in Table 7.21 may be used.

TABLE 7.19 Bearing on Finished Surfaces, ksi

Allowable stress (ASD)		Design strength (LRFD)	
F_y	F_p	F_y	$\phi R_n / A_{pb}$
36	32.4	36	54.0
42	37.8	42	63.0
45	40.5	45	67.5
50	45.0	50	75.0
55	49.5	55	82.5
60	54.0	60	90.0

TABLE 7.20 Allowable Bearing Loads on Expansion Rollers or Rockers, kips per in of Bearing

Allowable load (ASD)		Design strength (LRFD)	
F_y	F_p	F_y	ϕR_n^*
36	$0.76d$	36	$1.30d$
42	$0.96d$	42	$1.64d$
45	$1.06d$	45	$1.81d$
50	$1.22d$	50	$2.08d$
55	$1.39d$	55	$2.37d$
60	$1.55d$	60	$2.64d$

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

TABLE 7.21 Allowable Bearing on Masonry, ksi

On sandstone and limestone	0.40
On brick in cement mortar	0.25
On the full area of concrete	$0.35f'_c$
On less than full concrete area	$0.35f'_c\sqrt{A_2/A_1} \leq 0.7f'_c$

where f'_c = specified compressive strength, ksi, of the concrete
A_1 = bearing area
A_2 = concrete area

LRFD Procedure for Bearing. The design strength in bearing on the projected bearing area for finished surfaces, such as milled ends and ends of bearing stiffeners, or on the projected area of pins in finished holes, is ϕR_n, where $\phi = 0.75$.

$$R_n = 2.0F_yA_{pb} \tag{7.53}$$

where F_y is the lesser minimum yield stress, ksi, of the steel (Table 7.19) and A_{pb} is the projected bearing area, in^2.

For expansion rollers and rockers, R_n, kips, is given by

$$R_n = 1.5(F_y - 13)Ld/20 \tag{7.54}$$

where L is the length, in, of bearing, and d is the diameter, in (see Table 7.20).

7.24 COMBINED AXIAL COMPRESSION AND BENDING

A member carrying both axial and bending forces is subjected to secondary bending moments resulting from the axial force and the displacement of the neutral axis. This effect is referred to as the P-Δ effect. Such secondary bending moments are more critical in members where the axial force is a compressive force, because the P-Δ secondary moment increases the deflection of the member. In ASD, the effects of these secondary moments may be neglected where the axial force is a tensile force or where the actual compressive stress is less than 15% of the allowable compressive stress. LRFD does not include this concept.

The following design criteria apply to singly and doubly symmetrical members.

7.24.1 ASD for Compression and Bending

When the computed axial stress, f_a is less than 15% of F_a, the stress that would be permitted if axial force alone were present, a straight-line interaction formula may be used. Thus, when $f_a/F_a \leq 0.15$:

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \tag{7.55}$$

where subscripts x and y indicate, respectively, the major and minor axes of bending (if bending is about only one axis, then the term for the other axis is omitted), and

f_b = computed compressive bending stress, ksi, at point under consideration
F_b = compressive bending stress, ksi, that is allowed if bending alone existed

When $f_a/F_a > 0.15$, the effect of the secondary bending moment should be taken into account and the member proportioned to satisfy Eqs. (7.56*a*) and (7.56*b*) where, as before, subscripts x and y indicates axes of bending:

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{[1 - f_a/F'_{ex}]F_{bx}} + \frac{C_{my} f_{by}}{[1 - f_a/F'_{ey}]F_{by}} \leq 1.0 \tag{7.56a}$$

$$\frac{f_a}{0.60F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \tag{7.56b}$$

$$F'_e = \frac{12\pi^2 E}{23(Kl_b/r_b)^2} \tag{7.57}$$

where E = modulus of elasticity, 29,000 ksi
I_b = actual unbraced length, in, in the plane of bending

r_b = corresponding radius of gyration, in
K = effective-length factor in the plane of bending
C_m = reduction factor determined from the following conditions:

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

2. For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending, $C_m = 0.6 - 0.4M_1/M_2$, but not less than 0.4. M_1/M_2 is the ratio of the smaller to larger moments at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1/M_2 is positive when the member is bent in reverse curvature, and negative when it is bent in single curvature.

3. For compression members in frames braced against joint translation in the plane of loading and subjected to transverse loading between their supports, the value of C_m may be determined by rational analysis. Instead, however, C_m may be taken as 0.85 for members whose ends are restrained, and 1.0 for ends unrestrained.

In wind and seismic design F_e' may be increased one-third. The resultant section, however, should not be less than that required for dead and live loads alone without the increase in allowable stress.

Additional information, including illustrations of the foregoing three conditions for determining the value of C_m, is given in the AISC "Commentary" on the AISC ASD "Specification for Structural Steel for Buildings."

7.24.2 LRFD for Compression and Bending

Members subject to both axial compression and bending stresses should be proportioned to satisfy Eq. (7.58) or (7.59), whichever is applicable.

For $(P_u/\phi_c P_n) \geq 0.2$,

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \tag{7.58}$$

For $(P_u/\phi_c P_n) < 0.2$,

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}}\right) \leq 1.0 \tag{7.59}$$

where P_u = required compressive strength, kips, calculated for the factored axial loads
M_u = required flexural strength, kip-in calculated for primary bending and P-Δ effects
$\phi_c P_n$ = design compressive strength (Art. 7.19.3)
$\phi_b M_n$ = design flexural strength (Art. 7.20.2)

M_u may be determined for the factored loads from a second-order elastic analysis. The AISC LRFD specification, however, permits M_u to be determined from Eq. (7.60) with the variables in this equation determined from a first-order analysis.

$$M_u = B_1 M_{nt} + B_2 M_{lt} \tag{7.60}$$

where M_{nt} = required flexural strength, kip-in, with no relative displacement of the member ends; for example, for a column that is part of a rigid frame, drift is assumed prevented

M_{lt} = required flexural strength, kip-in, for the effects only of drift as determined from a first-order analysis

B_1 = magnification factor for M_{nt} to account for the P-Δ effects

$= \dfrac{C_m}{1 - P_u/P_e}$

C_m = reduction factor defined for Eq. (7.57)

B_2 = magnification factor for M_{lt} to account for the P-Δ effects

B_2 may be calculated from either Eq. (7.61) or (7.62), the former usually being the simpler to evaluate.

$$B_2 = \frac{1}{1 - (\Sigma P_u/\Sigma HL)\Delta_{oh}} \qquad (7.61)$$

$$B_2 = \frac{1}{1 - \Sigma P_u/\Sigma P_e} \qquad (7.62)$$

where ΣP_u = sum of the axial-load strengths, kips, of all the columns in a story

$P_e = A_g F_y/\lambda_c^2$

A_g = gross area of member, in^2

F_y = specified yield stress, ksi

$\lambda_c = \dfrac{Kl}{r\pi}\sqrt{\dfrac{F_y}{E}} = \dfrac{Kl}{r}\sqrt{\dfrac{F_y}{286{,}220}}$

K = effective column length factor in the plane of bending, to be determined by structural analysis, but not to exceed unity in calculation of B_1 and not to be less than unity in calculation of B_2

r = governing radius of gyration, in, about the plane of buckling

Δ_{oh} = drift, in, of the story in which the column is located

L = story height, in

ΣH = sum of all the horizontal forces on the story that cause Δ_{oh}

7.25 COMBINED AXIAL TENSION AND BENDING

For ASD, members subject to both axial tension and bending stresses should be proportioned to satisfy Eq. (7.55), with f_b and F_b, respectively, as the computed and allowable bending tensile stress. But the compressive bending stresses must not exceed the values given in Art. 7.20.1.

LRFD for Tension and Bending. Symmetric members subject to both axial tension and bending stresses should be proportioned to satisfy either Eq. (7.58) or Eq. (7.59), whichever is applicable.

7.26 COMPOSITE CONSTRUCTION

In composite construction, rolled or built-up steel shapes are combined with reinforced concrete to form a structural member. Examples of this type of construction include: (*a*) concrete-encased steel beams (Fig. 7.37*c*), (*b*) concrete decks interactive with steel beams (Fig. 7.37*a* and *b*), (*c*) concrete encased steel columns, and (*d*) concrete filled steel columns. The most common use of this type of construction is for composite beams, where the steel beam supports and works with the concrete slab to form an economical building element.

Design procedures require that a decision be made regarding the use of shoring for the deck pour. (Procedures for ASD and LRFD differ in this regard.) If shoring is not used, the steel beam must carry all dead loads applied until the concrete hardens, even if full plastic capacity is permitted for the composite section afterward.

The assumed composite cross section is the same for ASD and LRFD procedures. The effective width of the slab is governed by beam span and beam spacing or edge distance (Fig. 7.37*a* and *b*).

Slab compressive stresses are seldom critical for interior beams but should be investigated, especially for edge beams. Thickening the slab key and minimum requirements for strength of concrete can be economical.

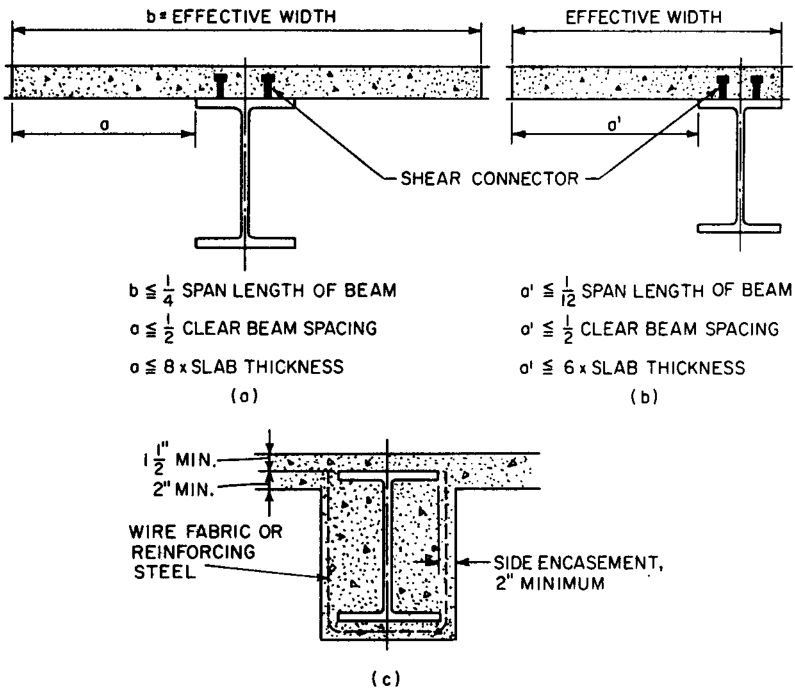


FIGURE 7.37 Steel-concrete composite-beam construction: (*a*) and (*b*) with welded-stud shear connectors; (*c*) with encasement in concrete.

Connector Details. In composite construction, shear connectors welded to the top flange of the steel beam are typically used to ensure composite action by transferring shear between the concrete deck and steel beam. Location, spacing, and size limitations for shear connectors are the same for ASD and LRFD procedures. Connectors, except those installed in ribs of formed steel decks, should have a minimum lateral concrete cover of 1 in. The diameter of a stud connector, unless located directly over the beam web, is limited to 2.5 times the thickness of the beam flange to which it is welded. Minimum center-to-center stud spacing is 6 diameters along the longitudinal axis, 4 diameters transversely. Studs may be spaced uniformly, rather than in proportion to horizontal shear, inasmuch as tests show a redistribution of shear under high loads similar to the stress redistribution in large bolted joints. Maximum spacing is 8 times the slab thickness.

Formed Steel Decking. Concrete slabs are frequently cast on permanent steel decking with a ribbed, corrugated, cellular, or blended cellular cross section (see Sec. 8). Two distinct composite-design configurations are inherent: ribs parallel or ribs perpendicular to the supporting beams or girders (Fig. 7.38) The design procedures, for both ASD and LRFD, prescribed for composite concrete-slab and steel-beam construction are also applicable for systems utilizing formed steel decking, subject to additional requirements of the AISC "Specification for Structural Steel for Buildings" and as illustrated in Fig. 7.38.

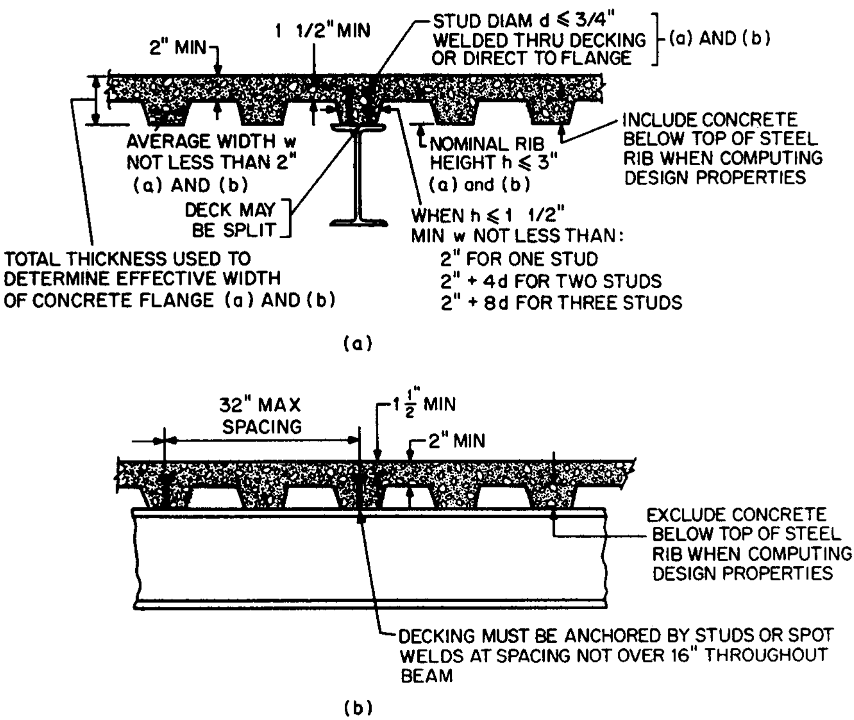


FIGURE 7.38 Steel-concrete composite-beam construction with formed steel decking: (*a*) ribs parallel to beam; (*b*) ribs transverse to beam [refer to (*a*) for applicable requirements].

Shear and Deflection of Composite Beams. In ASD and LRFD, shear forces are assumed to be resisted by the steel beam. Deflections are calculated based on composite section properties. It should be noted that, because of creep of the concrete, the actual deflections of composite beams under long-term loads, such as dead load, will be greater than those computed.

7.26.1 ASD of Encased Beams

Two design methods are allowed for encased beams. In one method, stresses are computed on the assumption that the steel beam alone supports all the dead load applied prior to concrete hardening (unless the beam is temporarily shored), and the composite beam supports the remaining dead and live loads. Then, for positive bending moments, the total stress, ksi, on the steel-beam bottom flange is

$$f_b = \frac{M_D}{S} + \frac{M_L}{S_t} \leq 0.66F_y \tag{7.63}$$

where F_y = specified yield stress of the steel, ksi
M_D = dead-load bending moment, kip-in
M_L = live-load bending moment, kip-in
S = section modulus of steel beam, in^3
S_t = section modulus of transformed section, in^3. To obtain the transformed equivalent steel area, divide the effective concrete area by the modular ratio n (modulus of elasticity of steel divided by modulus of elasticity of concrete). In computation of effective concrete area, use effective width of concrete slab (Fig. 7.37*a* and *b*)

The stress $0.66F_y$ is allowed because the steel beam is restrained against lateral buckling.

The second method stems from a "shortcut" provision contained in many building codes. This provision simply permits higher bending stresses in beams encased in concrete. For example,

$$f_b = \frac{M_D + M_L}{S} \leq 0.76F_y \tag{7.64}$$

This higher stress would not be realized, however, because of composite action.

7.26.2 ASD of Beams with Shear Connectors

For composite construction where shear connectors transfer shear between slab and beam, the design is based on behavior at ultimate load. It assumes that all loads are resisted by the composite section, even if shores are not used during construction to support the steel beam until the concrete gains strength. For this case, the computed stress in the bottom flange for positive bending moment is

$$f_b = \frac{M_D + M_L}{S_t} \leq 0.66F_y \tag{7.65}$$

where S_t = section modulus, in^3, of transformed section of composite beam. To prevent overstressing the bottom flange of the steel beam when temporary shoring is omitted, a limitation is placed on the value of S_t used in computation of f_b with Eq. (7.65):

$$S_t \leq \left(1.35 + 0.35\,\frac{M_L}{M_D}\right) S_s \tag{7.66}$$

where M_D = moment, kip-in, due to loads applied prior to concrete hardening (75% cured)
M_L = moment, kip-in, due to remaining dead and live loads
S_s = section modulus, in^3, of steel beam alone relative to bottom flange

Shear on Connectors. Shear connectors usually are studs or channels. The total horizontal shear to be taken by the connectors between the point of maximum positive moment and each end of a simple beam, or the point of counterflexure in a continuous beam, is the smaller of the values obtained from Eqs. (7.67) and (7.68).

$$V_h = \frac{0.85 f'_c A_c}{2} \tag{7.67}$$

$$V_h = \frac{A_s F_y}{2} \tag{7.68}$$

where f'_c = specified strength of concrete, ksi
A_c = actual area of effective concrete flange, as indicated in Fig. 7.36*a* and *b*, in^2
A_s = area of steel beam, in^2

In continuous composite beams, where shear connectors are installed in negative-moment regions, the longitudinal reinforcing steel in the concrete slab may be considered to act compositely with the steel beam in those regions. In such cases, the total horizontal shear to be resisted by the shear connectors between an interior support and each adjacent infection point is

$$V_h = \frac{A_{sr} F_{yr}}{2} \tag{7.69}$$

where A_{sr} = total area, in^2, of longitudinal reinforcing steel within the effective width of the concrete slab at the interior support
F_{yr} = specified yield stress of the reinforcing steel, ksi

These formulas represent the horizontal shear at ultimate load divided by 2 to approximate conditions at working load.

Number of Connectors. The minimum number of connectors N_1, spaced uniformly between the point of maximum moment and adjacent points of zero moment, is V_h/q, where q is the allowable shear load on a single connector, as given in Table 7.22. Values in this table, however, are applicable only to concrete made with aggregates conforming to ASTM C33. For concrete made with rotary-kiln-produced aggregates conforming to ASTM C330 and with concrete weight of 90 pcf or more, the allowable shear load for one connector is obtained by multiplying the values in Table 7.22 by the factors in Table 7.23.

TABLE 7.22 Allowable Horizontal-Shear Loads, q, for Connectors, kips (Applicable only to concrete made with ASTM C33 aggregates)

Connector	$f'_c = 3.0$	$f'_c = 3.5$	$f'_c \geq 4.0$
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†	5.5w†	5.6w†

*Length given is minimum.
†w = length of channel, in.

TABLE 7.23 Shear-Load Factors for Connectors in Lightweight Concrete

Air dry weight, pcf, of concrete	90	95	100	105	110	115	120
Factors for $f'_c \leq 4.0$ ksi	0.73	0.76	0.78	0.81	0.83	0.86	0.88
Factors for $f'_c \geq 5.0$ ksi	0.82	0.85	0.87	0.91	0.93	0.96	0.99

If a concentrated load occurs between the points of maximum and zero moments, the minimum number of connectors required between the concentrated load and the point of zero moment is given by

$$N_2 = \frac{V_h}{q}\left(\frac{S_t M_c/M - S_s}{S_t - S_s}\right) \tag{7.70}$$

where M = maximum moment, in-kips
M_c = moment, in-kips, at concentrated load $< M$
S_s = section modulus, in^3, of steel beam relative to bottom flange
S_t = section modulus, in^3, of transformed section of composite beam relative to bottom flange but not to exceed S_t computed from Eq. (7.66).

The allowable shear loads for connectors incorporate a safety factor of about 2.5 applied to ultimate load for the commonly used concrete strengths. Not to be confused with shear values for fasteners given in Art. 7.30, the allowable shear loads for connectors are applicable only with Eqs. (7.67) to (7.69).

The allowable horizontal shear loads given in Tables 7.22 and 7.23 may have to be adjusted for use with formed steel decking. For decking with ribs parallel to supports (Fig. 7.38*a*), the allowable loads should be reduced when w/h is less than 1.5 by multiplying the tabulated values by

$$0.6\left(\frac{w}{h}\right)\left(\frac{H}{h} - 1\right) \leq 1 \tag{7.71}$$

where w = average width of concrete rib, in
h = nominal rib height, in

H = length of stud after welding, in, but not more than $(h + 3)$ for computations

For decking with ribs perpendicular to supports, the reduction factor is:

$$\left(\frac{0.85}{\sqrt{N}}\right)\left(\frac{w}{h}\right)\left(\frac{H}{h} - 1\right) \leq 1 \tag{7.72}$$

where N = number of studs on a beam and in one rib, but three studs are the maximum that may be considered effective.

7.26.3 LRFD of Encased Beams

Two methods of design are allowed, the difference being whether or not shoring is used. In both cases, the design strength is $\phi_b M_n$, where $\phi_b = 0.90$. M_n is calculated for the elastic stress distribution on the composite section if shoring is used or the plastic stress distribution on the steel section alone if shoring is not used.

7.26.4 LRFD of Composite Beams

As with ASD, the use of shoring to carry deal loads prior to the time the concrete has hardened determines which design procedures are used. For composite construction where the steel beams are exposed, the design flexural strength for positive moment (compression in the concrete) is $\phi_b M_n$. It is dependent on the depth-thickness ratio h_c/t_w of the steel beam, where t_w is the web thickness and, for webs of rolled or formed sections, h_c is twice the distance from the neutral axis to the toe of the fillet at the compression flange, and for webs of built-up sections, h_c is twice the distance from the neutral axis to the nearest line of fasteners at the compression flange or the inside face of a welded compression flange.

When $h_c/t_w \leq 640/\sqrt{F_y}$, $\phi_b = 0.85$ and M_n is calculated for plastic stress distribution on the composite section. If $h_c/t_w > 640/\sqrt{F_y}$, $\phi_b = 0.90$ and M_n is calculated for elastic stress distribution, with consideration of the effects of shoring. When the member is subject to negative moment, the practical design approach is to neglect the composite section and use the requirements for beams in flexure, as given in Art. 7.20.2.

LRFD of Beams with Shear Connectors. The concepts described in Art. 7.26.2 for ASD apply also to LRFD of beams with shear connectors. Inasmuch as factored loads are used for LRFD, however, the equations used in the two types of design differ.

In regions of positive moment, the total horizontal shear V_h, kips, to be carried by the shear connectors between the point of maximum moment and the point of zero moment is the smallest value computed from Eqs. (7.73) to (7.75).

$$V_h = 0.85 f_c' A_c \tag{7.73}$$

$$V_h = A_s F_y \tag{7.74}$$

$$V_h = \Sigma Q_n \tag{7.75}$$

where f'_c = 28-day compressive strength of concrete, ksi
A_c = area of concrete slab within the effective width, in^2
A_s = area of the cross section, in^2, of steel beam
F_y = specified minimum yield stress of the steel
ΣQ_n = sum of the nominal strengths, kips, of the shear connectors between the point of maximum moment and the point of zero moment

The number of shear connectors n must equal or exceed V_h/Q_n, where Q_n is the nominal strength of one shear connector.

The nominal strength, kips, of one stud shear connector embedded in a solid concrete slab is

$$Q_n = 0.5A_{sc}\sqrt{f'_cE_c} \leq A_{sc}F_u \tag{7.76}$$

where A_{sc} = cross-sectional area of a stud shear connector, in^2
f'_c = 28-day compressive strength of concrete, ksi
F_u = minimum specified tensile strength of a stud shear connector, ksi
E_c = modulus of elasticity of the concrete, ksi

The nominal strength, kips, of one channel shear connector embedded in a solid concrete slab is

$$Q_n = 0.3(t_f + 0.5t_w)L_c\sqrt{f'_cE_c} \tag{7.77}$$

where t_f = flange thickness of channel shear connector, in
t_w = web thickness of channel, in
L_c = length of channel, in

As in ASD, the shear capacity of stud connectors may have to be reduced if they are used with formed metal decking. The reduction factors, Eqs. (7.71) and (7.72), also apply to LRFD.

7.27 MEMBERS SUBJECT TO TORSION

This is a special type of load application, since in normal practice eccentric loads on beams are counterbalanced to the point where slight eccentricities may be neglected. For example, spandrel beams supporting a heavy masonry wall may not be concentric with the load, thus inducing torsional stresses, but these will largely be canceled out by the equally eccentric loads of the floor, partitions, attached beams, and similar restraints. For this reason, one seldom finds any ill effects from torsional stresses.

It is during the construction phase that torsion may be in evidence, usually the result of faulty construction procedure. In Fig. 7.39 are illustrated some of the bad practices that have caused trouble in the field: when forms for concrete slabs are hung on one edge of a beam (usually the light secondary beam) the weight of the wet concrete may be sufficient to twist the beam. Figure 7.39 shows the correct method, which reduces torsion. Likewise for spandrels, the floor ties, if any, forms or the slab itself should be placed prior to the construction of the eccentric wall (Fig. 7.39*b*). Connectors for heavy roofing sheets when located on one side of the

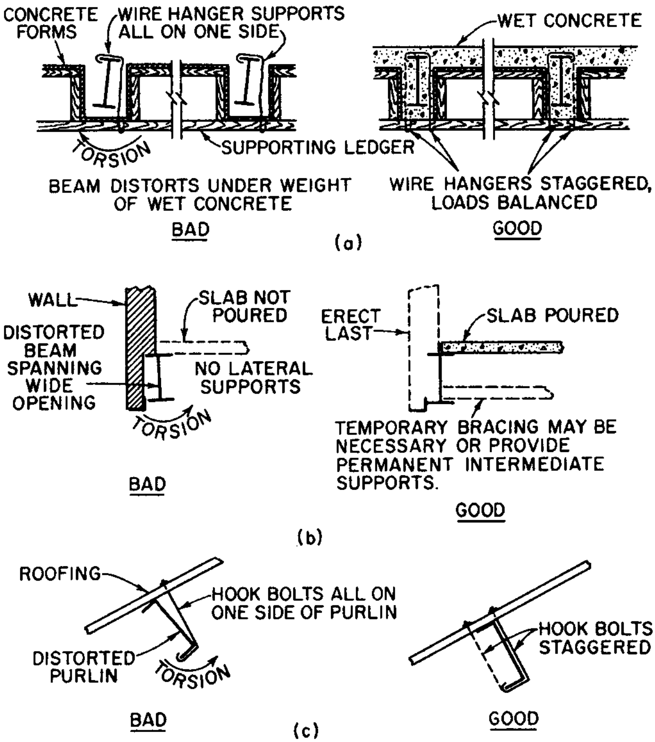


FIGURE 7.39 Steel beams subject to torsion—good and bad practice.

purlin may distort the section; the condition should be corrected by staggering, as indicated in Fig. 7.39*c*.

Equations for computing torsion stresses are given in Art. 5.4.2. Also, see Bibliography, Art. 7.55.

7.28 MEMBERS SUBJECT TO CYCLIC LOADING

Relatively few structural members in a building are ever subjected to large, repeated variations of stress or stress reversals (tension to compression, and vice versa) that could cause fatigue damage to the steel. Members need not be investigated for this possibility unless the number of cycles of such stresses exceeds 20,000, which is nearly equivalent to two applications every day for 25 years.

DESIGN OF CONNECTIONS

Design of connections and splices is a critical aspect of the design process. Because each fabricator has unique equipment and methods, the detailed configuration of connections plays an important part in determining the cost of the fabricated product. Consequently, the detailed design of these elements is a part of the work performed by the fabricator. In the industry, this work is known as **detailing.**

Usually, the structural engineer indicates the type of connections and type and size of fasteners required; for example, ‘‘framed connections with ⅞-in A325 bolts in bearing-type joints,’’ or the type of connection with reference to AWS D1.1 requirements. For beams, the design drawings should specify the reactions. If, however, the reactions are not noted, the detailer will determine the reactions from the uniform-load capacity (tabulated in the AISC Manual), giving due consideration to the effect of large concentrated loads near the connection. For connections resisting lateral loads, live, wind, or seismic, the design drawing should stipulate the forces and moments to be carried. Generally, the design should also include a sketch showing the type of moment connection desired.

Design Criteria for Connections. Either ASD or LRFD may be used to design the connections of a structure. Selection of the design procedure, however, must be consistent with the method used to proportion the members. When LRFD procedures are used, the loads and load factors discussed in Arts. 7.15 to 7.28 should be incorporated. The AISC Manual, Vol. II, Connections, provides many design aids for both design procedures.

7.29 COMBINATIONS OF FASTENERS

The AISC ASD and LRFD ‘‘Specification for Structural Steel for Buildings’’ distinguish between existing and new framing in setting conditions for use of fasteners in connection design.

In new work, A307 bolts or high-strength bolts in bearing-type connections should not be considered as sharing the load with welds. If welds are used, they should be designed to carry the load in the connection. However, when one leg of a connection angle is connected with one type of fastener and the other leg with a different type, this rule does not apply. The load is transferred across each joint by one type of fastener. Such connections are commonly used, since one type of fastener may be selected for shop work and a different type for field work.

High-strength bolts in slip-critical joints may share the load with welds on the same connection interface if the bolts are fully tightened before the welds are made.

For connections in existing frames, existing rivets and high-strength bolts may be used for carrying stresses from existing dead loads, and welds may be provided for additional dead loads and design live loads. This provision assumes that whatever slip that could occur in the existing joint has already occurred.

7.30 LOAD CAPACITY OF BOLTS

Under service conditions, bolts may be loaded in tension, shear, or a combination of tension and shear. The load capacities specified in AISC ASD and LRFD specifications are closely related and are based on the ‘‘Specification for Structural Joints Using ASTM A325 or A490 Bolts,’’ Research Council on Structural Connections of the Engineering Foundation. Both bearing-type and slip-critical bolted connections are proportional for the shear forces on the gross area of bolts.

7.30.1 ASD for Bolts

Allowable tension and shear stresses for bolts are listed in Table 7.24. The allowable bearing load at a bolt hole is $1.5F_u dt$, where F_u is the specified tensile strength, d is the nominal bolt diameter, and t = thickness of connected part.

Table 7.25 tabulates maximum sizes for standard, oversize, and slotted bolt holes. Oversize holes are permitted only in slip-critical connections. In slip-critical connections, slots may be formed without regard to the direction of loading; but in bearing-type connections, slot length should be placed normal to the direction of

TABLE 7.24 Allowable Stresses, ksi, for Bolts and Threaded Parts[a]

Fasteners	Shear in slip-critical connections F_v[b]				Bearing-type connections	
	Standard-size holes	Oversize and short-slot holes	Long-slot holes: Transverse load[c]	Long-slot holes: Parallel load[c]	Shear F_v	Tension F_t, including reduction for shear stress f_v[d]
A407 bolts					10.0[e,f]	$26 - 1.8f_v \leq 20$
Threaded Parts and A449 bolts, threaded[g] not excluded from shear planes					$0.17F_v$[h]	$0.43F_v - 1.8f_v \leq 0.33F_v$[h,i]
Threaded parts and A449 bolts, threads excluded from shear planes[g]					$0.22F_v$[h]	$0.43\ F_v - 1.4f_v \leq 0.33F_v$[h]
A325 bolts, when threads are not excluded from shear planes	17.0	15.0	12.0	10.0	21.0[f]	$\sqrt{(44)^2 - 4.39f_v^2}$
A325 bolts, when threads are excluded from shear planes	17.0	15.0	12.0	10.0	30.0[f]	$\sqrt{(44)^2 - 2.15f_v^2}$
A490 bolts, when threads are not excluded from shear planes	21.0	18.0	15.0	13.0	28.0[f]	$\sqrt{(54)^2 - 3.75f_v^2}$
A490 bolts, when threads are excluded from shear planes	21.0	18.0	15.0	13.0	40.0[f]	$\sqrt{(54)^2 - 1.82f_v^2}$

[a] For wind or seismic loading, acting alone or in combination with design dead and live loads, allowable stresses the table may be increased one-third, if the required section then is at least that required for design, dead, live, and impact loads without this increase. For tension combined with shear, the coefficients of f_v in the tabulated formulas could not be changed.

[b] Assumes clean mill scale and blast-cleaned surfaces with Class A coatings (slip coefficient 0.33). For special faying-surface conditions, see the Research Council on Structural Connections specification.

[c] Relative to the long axis of the slotted hole.

[d] Static loading only. For fatigue conditions, see the AISC ASD "Specification for Structural Steel for Buildings."

[e] Threads permitted in shear planes.

[f] Reduce 20% for bolts in bearing-type splices of tension members if the fastener pattern has a length, parallel to the line of force, exceeding 50 in.

[g] Applicable to threaded parts meeting the requirements of ASTM A36, A242, A441, A529, A572, A588, A709, A852 and to A449 bolts in bearing-type connections requiring bolt diameters exceeding 1½ in.

[h] F_v = minimum tensile strength, ksi, of bolts.

[i] For the threaded portion of an upset rod, $A_b F_t$ should be larger than $0.60 A_s F_y$, where A_b is the area at the major lead diameter, A_s is the nominal body area before upsetting, and F_y is the specified yield stress, ksi.

TABLE 7.25 Maximum Bolt-Hole Sizes, in*

Bolt diameter, in	Diameter of standard hole	Diameter of oversize hole	Short-slot hole (width × length)	Long-slot hole (width × length)
1/2	9/16	5/8	9/16 × 11/16	9/16 × 1 1/4
5/8	11/16	13/16	11/16 × 7/8	11/16 × 1 9/16
3/4	13/16	15/16	13/16 × 1	13/16 × 1 7/8
7/8	15/16	1 1/16	15/16 × 1 1/8	15/16 × 2 3/16
1	1 1/16	1 1/4	1 1/16 × 1 5/16	1 1/16 × 2 1/2
1 1/8	d + 1/16	d + 5/16	(d + 1/16) × (d + 3/8)	(d + 1/16) × (2.5 × d)

* Approval of the designer is required for use of oversize or slotted holes. Larger holes than those listed in the table, if required for tolerance in location of anchor bolts in concrete foundations, may be used in column base details.

loading. Washers, hardened when used with high-strength bolts, should be placed over oversize and short-slot holes.

Long-slot holes may be used in only one ply of the connected parts at an individual faying surface. When the slot is in an outer ply, plate washers or a continuous bar with standard holes should be installed to cover the entire slot. Washers or bars for A325 or A490 bolts should be 5/16 in or more thick but need not be hardened. If hardened washers are required, they should be placed over the outer surface of a plate washer or bar.

7.30.2 LRFD for Bolts

The design strength of bolts or threaded parts is ϕR_n (tabulated in Table 7.26) applied to the nominal body area of bolts and threaded parts except upset rods (see footnote *h* for Table 7.26). The applied load is the sum of the factored external loads plus the tension, if any resulting from prying action caused by deformation of connected parts. If high-strength bolts are required to support the applied loads by direct tension, they should be proportioned so that the average required strength (not including initial bolt tightening force) applied to the nominal bolt area will not exceed the design strength.

The design strength in tension for a bolt or threaded part subject to combined tension and shear stresses is also listed in Table 7.26. The value of f_v, the shear caused by the factored loads producing tensile stress, should not exceed the values for shear alone given in Table 7.26.

Table 7.25 lists maximum dimensions for standard, oversize, and slotted bolt holes. The limitations on these are the same as those for ASD (Art. 7.26.1).

The design bearing strength at a bolt hole may be taken as $\phi R_n = \phi 3.0 dtF_u$, or with $\phi = 0.75$, as $2.25 dtF_u$, where d is the nominal bolt diameter, t is the thickness of the connected part, and F_u is the tensile strength of the connected part.

7.31 LOAD CAPACITY OF WELDS

For welds joining structural steel elements, the load capacity depends on type of weld, strength of electrode material, and strength of the base metal. Fillet or groove

TABLE 7.26 Design Strength, ksi, for Bolts and Threaded Parts

Fasteners	Shear in slip-critical connections F_v[a]: Standard-size holes	Oversized and short-slot holes	Long-slot holes: Transverse loads[a]	Long-slot holes: Parallel load[b]	Bearing-type connections: Design shear strength ϕP_n	Bearing-type connections: Tension F_t, including reduction for shear stress f_v[c]
A307 bolts					16.2[d,e]	$39 - 1.8f_v \leq 30$
Threaded parts and A449 bolts, threads[f] not excluded from shear planes					$0.45F_u$[g]	$0.73F_u - 1.8f_v \leq 0.56F_u$[g,h]
Threaded parts and A449 bolts, threads excluded from shear planes[f]					$0.60F_u$[g]	$0.73F_u - 1.4f_v \leq 0.56F_u$[g]
A325 bolts, when threads are not excluded from shear planes	17.0	15.0	12.0	10.0	35.1[e]	$85 - 1.8f_v \leq 68$
A325 bolts, when threads are excluded from shear planes	17.0	15.0	12.0	10.0	46.8[e]	$85 - 1.4f_v \leq 68$
A490 bolts, when threads are not excluded from shear planes	21.0	18.0	15.0	13.0	43.9[e]	$106 - 1.8f_v \leq 84$
A490 bolts, when threads are excluded from shear planes	21.0	18.0	15.0	13.0	58.5[e]	$106 - 1.4f_v \leq 84$

[a] Assumes clean mill scale and blast-cleaned surfaces with Class A coatings (slip coefficient 0.33). For special faying-surface conditions, see the Research Council on Structural Connections LRFD specification for structural joints.

[b] Relative to the long axis of the slotted holes.

[c] Static loading only. For fatigue conditions, see the AISC ASD "Specification for Structural Steel for Buildings."

[d] $\phi = 0.60$. Threads permitted in shear planes.

[e] $\phi = 0.65$. Reduce design shear strength 20% for bolts in bearing-type splices of tension members if the fastener pattern has a length, parallel to the line of force, exceeding 50 in.

[f] Applicable to threaded parts meeting the requirements of ASTM A36, A242, A441, A529, A572, A588, A709, or A852 and to A449 bolts in bearing-type connections requiring bolt diameters exceeding 1½ in.

[g] F_u = minimum tensile strength, ksi, of bolts.

[h] For the threaded portion of an upset rod, A_bR_n should be larger than A_sF_y, where A_b is the area at the major thread diameter, A_s is the nominal body area before upsetting, F_y is the specified yield stress, ksi, and ϕR_n is the design tensile strength, where $\phi = 0.65$.

welds (Fig. 7.43) are commonly used for steel connections. Groove welds are classified as complete or partial penetration. (See Art. 7.3.5.)

A significant characteristic of fillet-welded joints is that all forces, regardless of the direction in which they act, are resolved as shear on the effective throat of the weld. For instance, when joining elements such as a girder flange to a web, fillet welds are designed to carry the horizontal shear without regard to the tensile or compressive stresses in the elements.

For computation of load capacity, the effective area of groove and fillet welds is the effective length times the effective throat thickness. The effective area for a plug or slot weld is the nominal cross-sectional area of the hole or slot in the plane of the faying surface.

Except for fillet welds in holes or slots, the effective length of a fillet weld is the overall length of weld, including the return. For a groove weld, the effective length should be taken as the width of the part joined.

The effective throat thickness of a fillet weld is the shortest distance from the root of the joint to the nominal face of the weld (Fig. 7.3). For fillet welds made by the submerged-arc process, however, the effective throat should be taken as the leg size for welds ⅜ in and smaller but as the theoretical throat plus 0.11 in for larger fillet welds.

For a complete-penetration groove weld, the effective throat is the thickness of the thinnest part joined. For partial-penetration groove welds, the effective throat thickness depends on the included angle at the root of the groove. For all J or U joints and for bevel or V joints with an included angle of 60° or more, the effective throat thickness may be taken as the depth of the chamfer. When the included angle for bevel or V joints is between 45° and 60°, the effective throat thickness should be the depth of chamfer minus ⅛ in. For flare bevel and flare V-groove welds when flush to the surface of a bar or a 90° bend in a formed section, the effective throat thickness is ⁵⁄₁₆ and ½ the radius of the bar or bend, respectively. When the radius is 1 in or more, for gas metal arc welding, the effective thickness is ¼ the radius.

Welds subject to static loads should be proportioned by ASD for the allowable stresses and by LRFD for the design strengths in Table 7.27. If connections will

TABLE 7.27 Design Shear Strength for Welds, ksi*

Types of weld and stress	Material	LRFD: Resistance factor ϕ	LRFD: Nominal strength† F_{BM} or F_w	ASD: Allowable stress
		Complete penetration groove weld		
Tension normal to effective area	Base	0.90	F_y	Same as base metal
Compression normal to effective area	Base	0.90	F_y	Same as base metal
Tension or compression parallel to axis of weld				
Shear on effective area	Base	0.90	$0.60F_y$	0.30 × nominal tensile strength of weld metal
	Weld electrode	0.80	$0.60F_{EXX}$	
		Partial penetration groove welds		
Compression normal to effective area				
Tension or compression parallel to axis of weld†	Base	0.90	F_y	Same as base metal
Shear parallel to axis of weld	Base Weld electrode	0.75	$0.60F_{EXX}$	0.30 × nominal tensile strength of weld metal
Tension normal to effective area	Base	0.90	F_y	3.0 × nominal tensile strength of weld metal
	Weld electrode	0.80	$0.60F_{EXX}$	
		Fillet welds		
Shear on effective area	Base Weld electrode	0.75	$0.60F_{EXX}$	0.30 × nominal tensile strength of weld metal
Tension or compression parallel to axis of weld†	Base	0.90	F_y	
		Plug or slot welds		
Shear parallel to faying surfaces (on effective area)	Base Weld electrode	0.75	$0.60F_{EXX}$	3.0 × nominal tensile strength of weld metal

*Reprinted with permission from F. S. Merritt and R. L. Brockenbrough, "Structural Steel Designers Handbook," 2d ed., McGraw-Hill, Inc., New York.

†Design strength is the smaller of F_{BM} and F_u:
F_{BM} = nominal strength of base metal to be welded, ksi.
F_w = nominal strength of weld electrode material, ksi.
F_y = specified minimum yield stress of base metal, ksi.
F_{EXX} = classification strength of weld metal, as specified in appropriate AWS specification, ksi.

be subject to fatigue from stress fluctuations, load capacity should be reduced as provided in the AISC "Specification for Structural Steel for Buildings."

7.32 BEARING-TYPE BOLTED CONNECTIONS

When some slip, although very small, may occur between connected parts, the fasteners are assumed to function in shear. The presence of paint on contact surfaces is therefore of no consequence. Fasteners may be A307 bolts or high-strength bolts or any other similar fastener not dependent on development of friction on the contact surfaces.

Single shear occurs when opposing forces act on a fastener as shown in Fig. 7.39*a*, tending to slide on their contact surfaces. The body of the fastener resists this tendency; a state of shear then exists over the cross-sectional area of the fastener.

Double-shear takes place whenever three or more plates act on a fastener as illustrated in Fig. 7.40*b*. There are two or more parallel shearing surfaces (one on each side of the middle plate in Fig. 7.40*b*). Accordingly, the shear strength of the fastener is measured by its ability to resist two or more single shears.

Bearing on Base Metal. This is a factor to consider; but calculation of bearing stresses in most joints is useful only as an index of efficiency of the net section of tension members.

Edge Distances. The AISC "Specification for Structural Steel for Buildings," ASD and LRFD, recommends minimum edge distances, center of hole to edge of connected part, as given in Table 7.28. In addition, the edge distance, in, when in the direction of force should not be less than $2P/F_ut$ for ASD or $P/\phi F_ut$ for LRFD, where p is the force, kips, transmitted by one fastener to the part for which the edge distance is applicable; $\phi = 0.75$; F_u is the specified minimum tensile strength of the part (not the fastener), ksi; and t is the thickness of the part, in.

A special rule applies to beams with framed connections that are usually designed for the shear due to beam reactions. The edge distance for the beam web, with standard-size holes, should be not less than $2P_R/F_ut$ for ASD or $P_R/\phi F_ut$ for

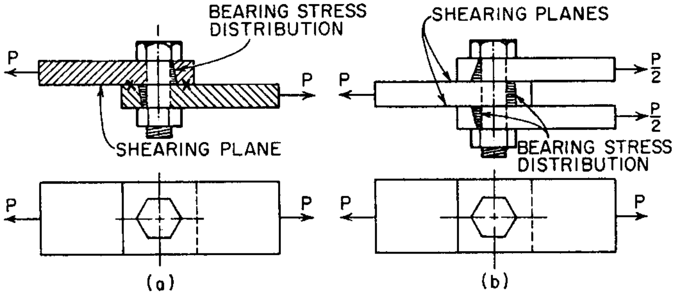


FIGURE 7.40 Bolted connection in shear and bearing: (*a*) with bolt in single shear; (*b*) with bolt in double shear (two shearing planes).

TABLE 7.28 Minimum Edge Distance for Punched, Reamed, or Drilled Holes, in

Fastener diameter, in	At sheared edges	At rolled edges of plates, shapes or bars or gas-cut edges†
½	⅞	¾
⅝	1⅛	⅞
¾	1¼	1
⅞	1½*	1⅛
1	1¾*	1¼
1⅛	2	1½
1¼	2¼	1⅝
Over 1¼	1¾ × diameter	1¼ × diameter

*These may be 1¼ in at the ends of beam connection angles.
† All edge distances in this column may be reduced ⅛ in when the hole is at a point where stress does not exceed 25% of the maximum allowed stress in the element.

LRFD, where P_R is the beam reaction per bolt, kips. This rule, however, need not be applied when the bearing stress transmitted by the fastener does not exceed $0.90F_u$.

The maximum distance from the center of a fastener to the nearest edge of parts in contact should not exceed 6 in or 12 times the part thickness.

Minimum Spacing. The AISC specification also requires that the minimum distance between centers of bolt holes be at least 2⅔ times the bolt diameter. But at least three diameters is desirable. Additionally, the hole spacing, in, when along the line of force, should be at least $2P/F_ut + d/2$ for ASD or $P/\phi F_ut + d/2$ for LRFD, where P, F_u, and t are as previously defined for edge distance and d = nominal diameter of fastener, in. Since this rule is for standard-size holes, appropriate adjustments should be made for oversized and slotted holes. In no case should the clear distance between holes be less than the fastener diameter.

Eccentric Loading. Stress distribution is not always as simple as for the joint in Fig. 7.40*a* where the fastener is directly in the line of significant. Sometimes, the load is applied eccentrically, as shown in Fig. 7.41. For such connections, tests show that use of actual eccentricity to compute the maximum force on the extreme fastener is unduly conservative because of plastic behavior and clamping force generated by the fastener. Hence, it is permissible to reduce the actual eccentricity to a more realistic "effective" eccentricity.

For fasteners equally spaced on a single gage line, the effective eccentricity in inches is given by

$$l_{\text{eff}} = l - \frac{1 + 2n}{4} \tag{7.78}$$

where l = the actual eccentricity and n = the number of fasteners. For the bracket in Fig. 7.41*b* the reduction applied to l_1 is $(1 + 2 \times 6)/4 = 3.25$ in.

For fasteners on two or more gage lines

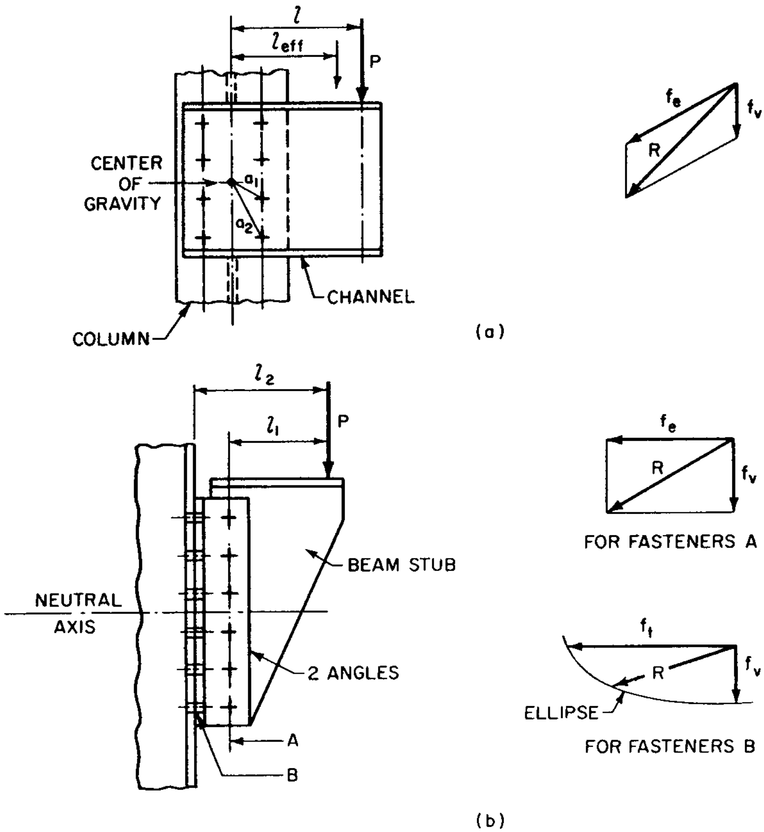


FIGURE 7.41 Eccentrically loaded fastener groups: (*a*) with bolts in shear only; (*b*) with bolts in combined tension and shear.

$$l_{\text{eff}} = l - \frac{1 + n}{2} \tag{7.79}$$

when n is the number of fasteners per gage line. For the bracket in Fig. 7.41*a*, the reduction is $(1 + 4)/2 = 2.5$ in.

In Fig. 7.41*a*, the load P can be resolved into an axial force and a moment: Assume two equal and opposite forces acting through the center of gravity of the fasteners, both forces being equal to and parallel to P. Then, for equal distribution on the fasteners, the shear on each fastener caused by the force acting in the direction of P is $f_v = P/n$, where n is the number of fasteners.

The other force forms a couple with P. The shear stress f_e due to the couple is proportional to the distance from the center of gravity and acts perpendicular to the line from the fastener to the center. In determining f_e, it is convenient to first express it in terms of x, the force due to the moment Pl_{eff} on an imaginary fastener at unit distance from the center. For a fastener at a distance a from the center, $f_e = ax$, and the resisting moment is $f_e a = a^2 x$. The sum of the moments equals Pl_{eff}. This equation enables x to be evaluated and hence, the various values of f_e. The resultant R of f_e and f_v can then be found; a graphical solution usually is sufficiently accurate. The stress so obtained must not exceed the allowable value of the fastener in shear (Art. 7.30).

For example, in Fig. 7.41*a*, $f_v = P/8$. The sum of the moments is

$$4a_1^2x + 4a_2^2x = Pl_{\text{eff}}$$

$$x = \frac{Pl_{\text{eff}}}{4a_1^2x + 4a_2^2x}$$

Then, $f_e = a_2x$ for the most distant fastener, and R can be found graphically as indicated in Fig. 7.41*a*.

Tension and Shear. For fastener group B in Fig. 7.41*b*, use actual eccentricity l_2 since these fasteners are subjected to combined tension and shear. Here too, the load P can be resolved into an axial shear force through the fasteners and a couple. Then, the stress on each fastener caused by the axial shear is P/n, where n is the number of fasteners. The tensile forces on the fasteners vary with distance from the center of rotation of the fastener group.

A simple method, erring on the safe side, for computing the resistance moment of group B fasteners assumes that the center of rotation coincides with the neutral axis of the group. It also assumes that the total bearing pressure below the neutral

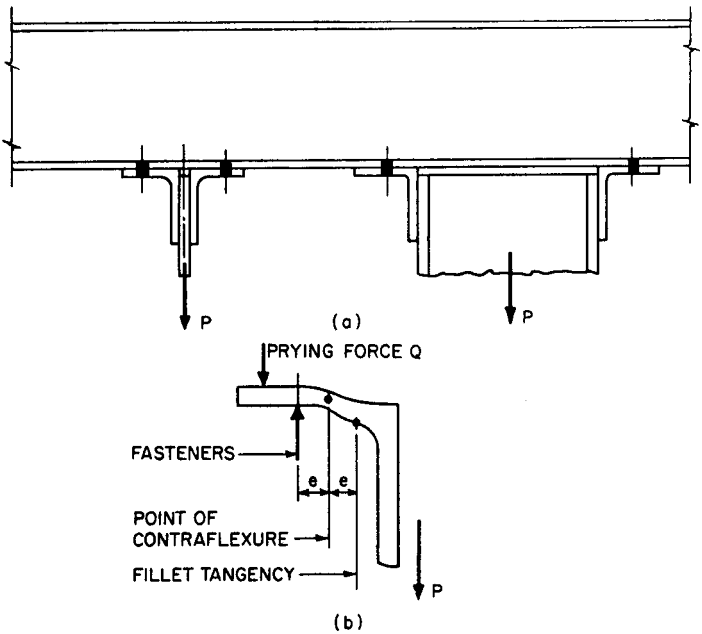


FIGURE 7.42 Fasteners in tension. Prying action on the connection causes a moment $M = Pe/n$ on either side, where P = applied load, e its eccentricity, as shown above, and n the number of fasteners resisting the moment.

axis equals the sum of the tensile forces on the fasteners above the axis. Then, with these assumptions, the tensile force on the fastener farthest from the neutral axis is

$$f_t = \frac{d_{\max} P l_2}{\Sigma A d^2} \tag{7.80}$$

where d = distance of each fastener from the neutral axis
$d_{\max}$ = distance from neutral axis of farthest fastener
A = nominal area of each fastener

The maximum resultant stresses f_t and $f_v = P/n$ are then plotted as an ellipse and R is determined graphically. The allowable stress is given as the tensile stress F_t as a function of the computer shear stress f_v. (In Tables 7.24 and 7.26, allowable stresses are given for the ellipse approximated by three straight lines.)

Note that the tensile stress of the applied load is not additive to the internal tension (pretension) generated in the fastener on installation. On the other hand, the AISC Specification does require the addition to the applied load of tensile stresses resulting from prying action, depending on the relative stiffness of fasteners and connection material. Prying force Q (Fig. 7.42*b*) may vary from negligible to a substantial part of the total tension in the fastener. A method for computing this force is given in the AISC Manual.

The old method for checking the bending strength of connection material ignored the effect of prying action. It simply assumed bending moment equal to P/n times e (Fig. 7.42). This procedure may be used for noncritical applications.

7.33 SLIP-CRITICAL BOLTED CONNECTIONS

Design of this type of connection assumes that the fastener, under high initial tensioning, develops frictional resistance between the connected parts, preventing slippage despite external load. Properly installed A307 bolts provide some friction, but since it is not dependable it is ignored. High-strength steel bolts tightened nearly to their yield strengths, however, develop substantial, reliable friction. No slippage will occur at design loads if the contact surfaces are clean and free of paint or have only scored galvanized coatings, inorganic zinc-rich paint, or metallized zinc or aluminum coatings.

The AISC "Specification for Structural Steel for Buildings," ASD and LRFD, lists allowable shear for high-strength bolts in slip-critical connections. Though there actually is not shear on the bolt shank, the shear concept is convenient for measuring bolt capacity.

Since most joints in building construction can tolerate tiny slippage, bearing-type joints, which are allowed much higher shears for the same high-strength bolts when the threads are not in shear planes, may, for reasons of economy, lessen the use of slip-critical joints.

The capacity of a slip-critical connection does not depend on the bearing of the bolts against the sides of their holes. Hence, general specification requirements for protection against high bearing stresses or bending in the bolts may be ignored.

If the fasteners B in Fig. 7.41*b* are in a slip-critical connection, the bolts above the neutral axis will lose part of their clamping force; but this is offset by a compressive force below the neutral axis. Consequently, there is no overall loss in frictional resistance to slippage.

When it is apparent that there may be a loss of friction (which occurs in some type of brackets and hangers subject to tension and shear) and slip under load cannot be tolerated, the working value in shear should be reduced in proportion to the ratio of residual tension to initial tension.

Slip-critical connections subjected to eccentric loading, such as that illustrated in Fig. 7.41, are analyzed in the same manner as bearing-type connections (Art. 7.32).

7.34 ECCENTRICALLY LOADED WELDED CONNECTIONS

Welds are of two general types, fillet (Fig. 7.43*a*) and groove (Fig. 7.43*b*), with allowable stresses dependent on grade of weld and base steels. Since all forces on a fillet weld are resisted as shear on the effective throat (Art. 7.31), the strength of connections resisting direct tension, compression and shear are easily computed on the basis that a kip of fillet shear resists a kip of the applied forces. Many connections, some of which are shown in Fig. 7.44, are not that simple because of eccentricity of applied force with respect to the fillets. In designing such joints it is customary to take into account the actual eccentricity.

The underlying design principles for eccentric welded connections are similar to those for eccentric bolted connections (Art. 7.32). Consider the welded bracket in Fig. 7.45. The first step is to compute the center of gravity of the weld group. Then, the load P can be resolved into an equal and parallel load through the center of gravity and a couple. The load through the center of gravity is resisted by a uniform shear on the welds; for example, if the welds are all the same size, shear per linear inch is $f_v = P/n$ where n is the total linear inches of weld. The moment Pl of the couple is resisted by the moment of the weld group. The maximum stress, which occurs on the weld element farthest from the center of gravity, may be expressed as $f_e = Pl/S$, where S is the polar section modulus of the weld group.

To find S, first compute the moments of inertia I_x of the welds about the XX axis and I_Y about the perpendicular YY axis. (If the welds are all the same size, their lengths, rather than their relative shear capacities, can be conveniently used in all moment calculations.) The polar moment of inertia $J = I_X + U_Y$, and the polar section modulus $S = J/a$, where a is the distance from the center of gravity to the farthest weld element. The resultant R of f_v and f_e, which acts normal to the

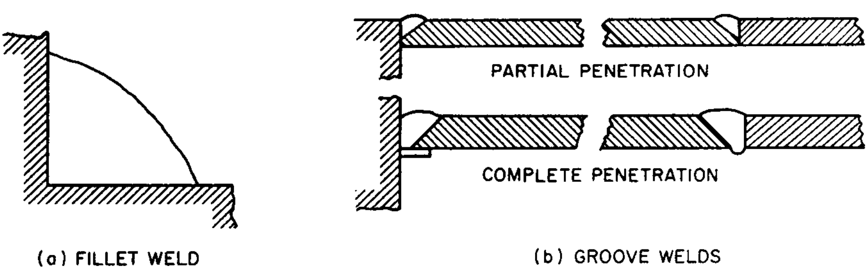


FIGURE 7.43 Two main types of weld—fillet and grove. Grove welds may be complete or partial penetration.

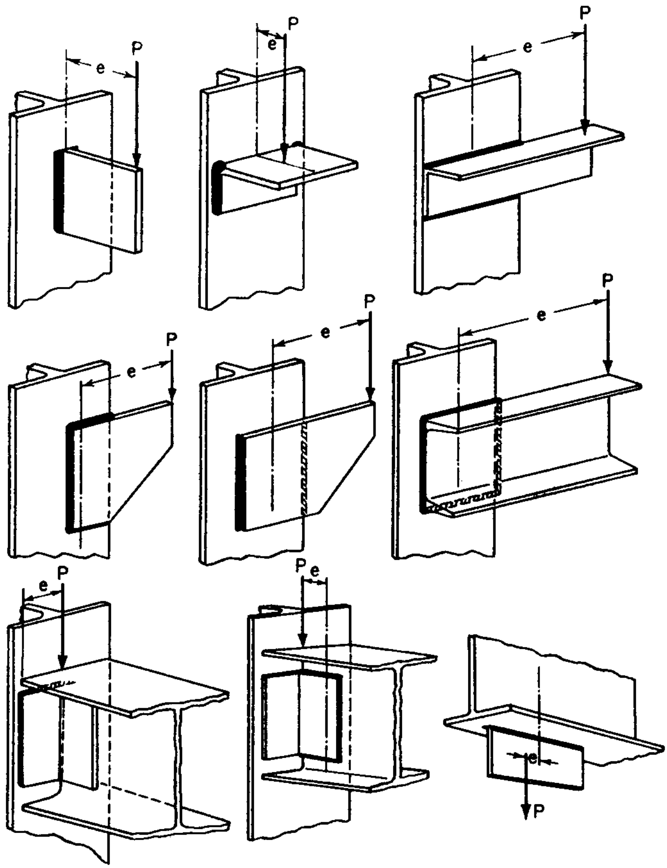


FIGURE 7.44 Typical eccentric welded connections.

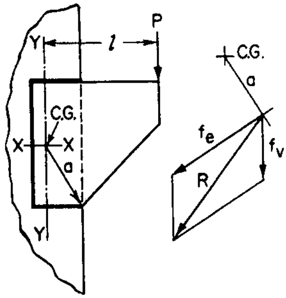


FIGURE 7.45 Stresses on welds caused by eccentricity.

line from the center of gravity to the weld element for which the stress is being determined, should not exceed the capacity of the weld element (Art. 7.31).

7.35 TYPES OF BEAM CONNECTIONS

In general, all beam connections are classified as either **framed** or **seated.** In the framed type, the beam is connected to the supporting member with fittings (short angles are common) attached to the beam web. With seated connections, the ends of the beam rest on a ledge or seat, in much the same manner as if the beam rested on a wall.

7.35.1 Bolted Framed Connections

When a beam is connected to a support, a column or a girder, with web connection angles, the joint is termed "framed." Each connection should be designed for the end reaction of the beam, and type, size and strength of the fasteners, and bearing strength of base materials should be taken into account. To speed design, the AISC Manual lists a complete range of suitable connections with capacities depending on these variables. Typical connections for beam or channels ranging in depth from 3 to 30 in are shown in Fig. 7.46.

To provide sufficient stability and stiffness, the length of connection angles should be at least half the clear depth of beam web.

For economy, select the minimum connection adequate for the load. For example, assume an 18-in beam is to be connected. The AISC Manual (ASD) lists three- and four-row connections in addition to the five-row type shown in Fig. 7.46. Total shear capacity ranges from a low of 26.5 kips for ¾-in-diam A307 bolts in a three-row regular connection to a high of 263.0 kips for 1-in-diam A325 bolts in a five-row heavy connection, bearing type. This wide choice does not mean that all types of fasteners should be used on a project, but simply that the tabulated data cover many possibilities, enabling an economical selection. Naturally, one type of fastener should be used throughout, if practical; but shop and field fasteners may be different.

Bearing stresses on beam webs should be checked against allowable stresses (Arts. 7.30.1 and 7.30.2), except for slip-critical connections, in which bearing is

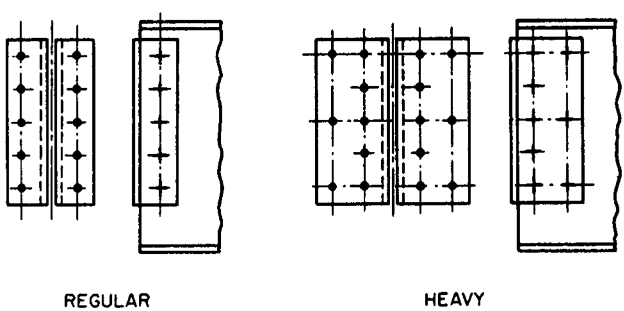


FIGURE 7.46 Typical bolted framed connections.

not a factor. Sometimes, the shear capacity of the field fasteners in bearing-type connections may be limited by bearing on thin webs, particularly where beams frame into opposite sides of a web. This could occur where beams frame into column or girder webs.

One side of a framed connection usually is shop connected, the other side field connected. The capacity of the connection is the smaller of the capacities of the shop or field group of fasteners.

In the absence of specific instructions in the bidding information, the fabricator should select the most economical connection. Deeper and stiffer connections, if desired by the designer, should be clearly specified.

7.35.2 Bolted Seated Connections

Sizes, capacities, and other data for seated connections for beams, shown in Fig. 7.47, are tabulated in the AISC Manual. Two types are available, stiffened seats (Fig. 7.47*a*) and unstiffened seats (Fig. 7.47*b*).

Unstiffened Seats. Capacity is limited by the bending strength of the outstanding horizontal leg of the seat angle. A 4-in leg 1 in thick generally is the practical limit. In ASD, an angle of A36 steel with these dimensions has a top capacity of 60.5 kips for beams of A36 steel, and 78.4 kips when F_y = 50 ksi for the beam steel. Therefore, for larger end reactions, stiffened seats are recommended.

The actual capacity of an unstiffened connection will be the lesser of the bending strength of the seat angle, the shear resistance of the fasteners in the vertical leg, or the bearing strength of the beam web. (See also Art. 7.22 for web crippling stresses.) Data in the AISC Manual make unnecessary the tedious computations of balancing the seat-angle bending strength and beam-web bearing.

The nominal setback from the support of the beam to be seated is ½ in. But tables for seated connections assume ¾ in to allow for mill underrun of beam length.

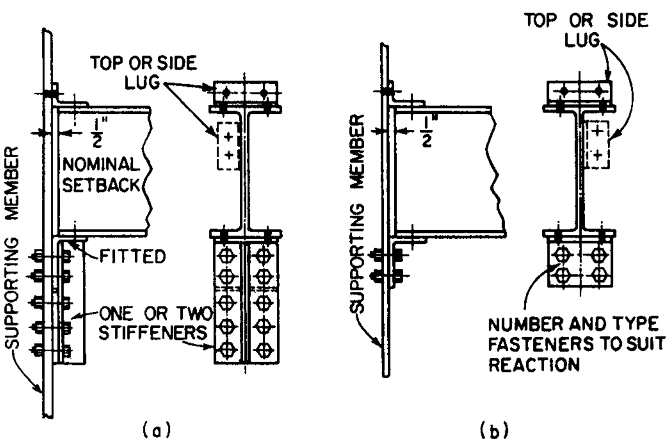


FIGURE 7.47 Typical bolted seated connections: (*a*) stiffened seat; (*b*) unstiffened seat.

Stiffened Seats. These may be obtained with either one or two stiffener angles, depending on the load to be supported. As a rule, stiffeners with outstanding legs having a width less than 5 in are not connected together; in fact, they may be separated, to line up the angle gage line (recommended centerline of fasteners) with that of the column.

The capacity of a stiffened seat is the lesser of the bearing strength of the fitted angle stiffeners or the shear resistance of the fasteners in the vertical legs. Crippling strength of the beam web usually is not the deciding factor, because of ample seat area. When legs larger than 5 in wide are required, eccentricity should be considered, in accordance with the technique given in Art. 7.32. The center of the beam reaction may be taken at the midpoint of the outstanding leg.

Advantages of Seated Connections. For economical fabrication, the beams merely are punched and are free from shop-fastened details. They pass from the punching machine to the paint shed, after which they are ready for delivery. In erection, the seat provides an immediate support for the beam while the erector aligns the connection hole. The top angle is used to prevent accidental rotation of the beam. For framing into column webs, seated connections allow more erection clearance for entering the trough formed by column flanges than do framed connections. A framed beam usually is detailed to whim 1⁄16 in of the column web. This provides about 1⁄8 in total clearance, whereas a seated beam is cut about 1⁄2 in short of the column web, yielding a total clearance of about 1 in. Then, too, each seated connection is wholly independent, whereas for framed beams on opposite sides of a web, there is the problem of aligning the holes common to each connection.

Frequently, the angles for framed connections are shop attached to columns. Sometimes, one angle may be shipped loose to permit erection. This detail, however, cannot be used for connecting to column webs, because the column flanges may obstruct entering or tightening of bolts. In this case, a seated connection has a distinct advantage.

7.35.3 Welded Framed Connections

The AISC Manual tabulates sizes and capacities of angle connections for beams for three conditions: all welded, both legs (Fig. 7.48); web leg shop welded, outstanding leg for hole-type fastener; and web leg for hole-type fastener installed in shop, outstanding leg field welded. Tables are based on E70 electrodes. Thus, the connections made with A36 steel are suitable for beams of both carbon and high-strength structural steels.

Eccentricity of load with respect to the weld patterns causes stresses in the welds that must be considered in addition to direct shear. Assumed forces, eccentricities, and induced stresses are shown in Fig. 7.48*b*. Stresses are computed as in the example in Art. 7.34, based on vector analysis that characterizes elastic design. The capacity of welds A or B that is smaller will govern design.

If ultimate strength (plastic design) of such connections is considered, many of the tabulated ''elastic'' capacities are more conservative than necessary. Although AISC deemed it prudent to retain the ''elastic'' values for the weld patterns, recognition was given to research results on plastic behavior by reducing the minimum beam-web thickness required when welds A are on opposite sides of the web. As a result, welded framed connections are now applicable to a larger range of rolled beams than strict elastic design would permit.

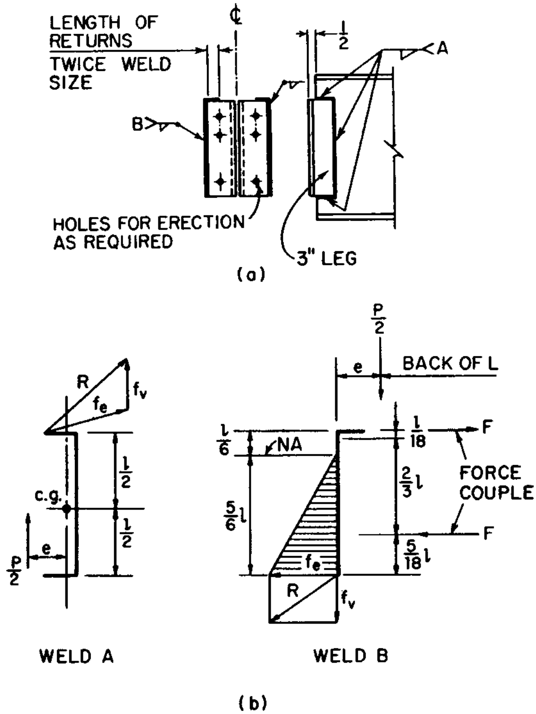


FIGURE 7.48 Welded framed connections on beam web: (*a*) weld locations along connection angles; (*b*) forces on welds.

Shear stresses in the supporting web for welds B should also be investigated, particularly when beams frame on opposite side of the web.

7.35.4 Welded-Seat Connections

Also tabulated in the AISC Manual, welded-seat connections (Fig. 7.49) are the welded counterparts of bolted-seat connections for beams (Art. 7.35.2). As for welded frame connections (Art. (7.35.3), the load capacities for seats, taking into account for eccentricity of loading on welds, are computed by "elastic" vector analysis. Assumptions and the stresses involved are shown in Fig. 7.49*c*.

In ASD, an unstiffened seat angle of A36 steel has a maximum capacity of 60.5 kips for supporting beams of A36 steel, and 78.4 kips for steel with F_y = 50 ksi (Fig. 7.49*a*). For heavier loads, a stiffened seat (Fig. 7.49*b*) should be used.

Stiffened seats may be a beam stub, a tee section, or two plates welded together to form a tee. Thickness of the stiffener (vertical element) depends on the strengths of beam and seat materials. For a seat of A36 steel, stiffener thickness should be at least that of the supported beam web when the web is A36 steel, and 1.4 times thicker for web steel with F_y = 50 ksi.

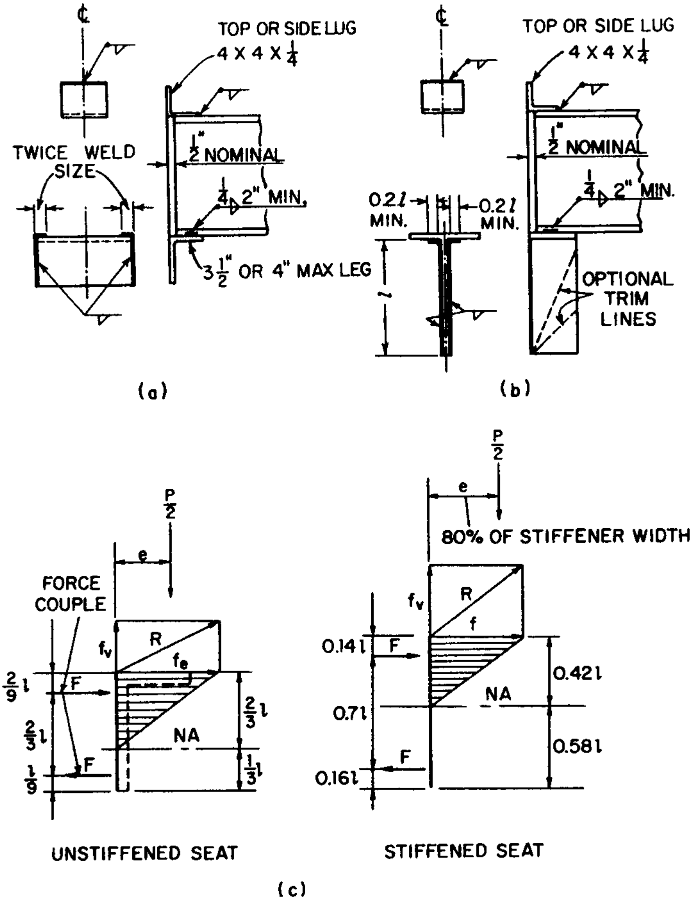


FIGURE 7.49 Welded-seat connections: (*a*) unstiffened seat; (*b*) stiffened seat; (*c*) stresses in the welds.

When stiffened seats are on line on opposite sides of a supporting web of A36 steel, the weld size made with E70 electrodes should not exceed one-half the web thickness, and for web steel with F_y = 50 ksi, two-thirds the web thickness.

Although top or side lug angles will hold the beam in place in erection, it often is advisable to use temporary erection bolts to attach the bottom beam flange to the seat. Usually, such bolts may remain after the beam flange is welded to the seat.

7.35.5 End-Plate Connections

The art of welding makes feasible connections that were not possible with older-type fasteners, e.g., end-plate connections (Fig. 7.50).

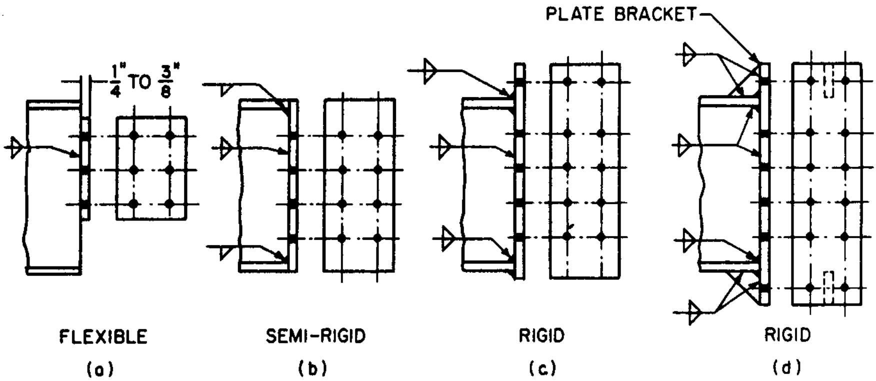


FIGURE 7.50 End-plate connection between beam and column flange.

Of the several variations, only the flexible type (Fig. 7.50*c*) has been "standardized" with tabulated data in the AISC Manual. Flexibility is assured by making the end plate ¼ in thick wherever possible (never more than ⅜ in). Such connections in tests exhibit rotations similar to those for framed connections.

The weld connecting the end plate to the beam web is designed for shear. There is no eccentricity. Weld size and capacity are limited by the shear strength of the beam web adjoining the weld. Effective length of weld is reduced by twice the weld size to allow for possible deficiencies at the ends.

As can be observed, this type of connection requires accurate cutting of the beam to length. Also the end plates must be squarely positioned so as to compensate for mill and shop tolerances.

The end plate connection is easily adapted for resisting beam moments (Fig. 7.50*b*, *c* and *d*). One deterrent, however, to its use for tall buildings where column flanges are massive and end plates thick is that the rigidity of the parts may prevent drawing the surfaces into tight contact. Consequently, it may not be easy to make such connections accommodate normal mill and shop tolerances.

7.35.6 Special Connections

In some structural frameworks, there may be connections in which a standard type (Arts. 7.35.1 to 7.35.5) cannot be used. Beam centers may be offset from column centers, or intersection angles may differ from 90°, for example.

For some skewed connections the departure from the perpendicular may be taken care of by slightly bending the framing angles. When the practical limit for bent angles is exceeded, bent plates may be used (Fig. 7.51*a*).

Special one-sided angle connections, as shown in Fig. 7.51*b*, are generally acceptable for light beams. When such connections are used, the eccentricity of the fastener group in the outstanding leg should be taken into account. Length l may be reduced to the effective eccentricity (Art. 7.32).

Spandrel and similar beams lined up with a column flange may be conveniently connected to it with a plate (Fig. 7.51*c* and *d*). The fasteners joining the plate to the beam web should be capable of resisting the moment for the full lever arm l for the connection in Fig. 7.51*c*. For beams on both sides of the column with equal

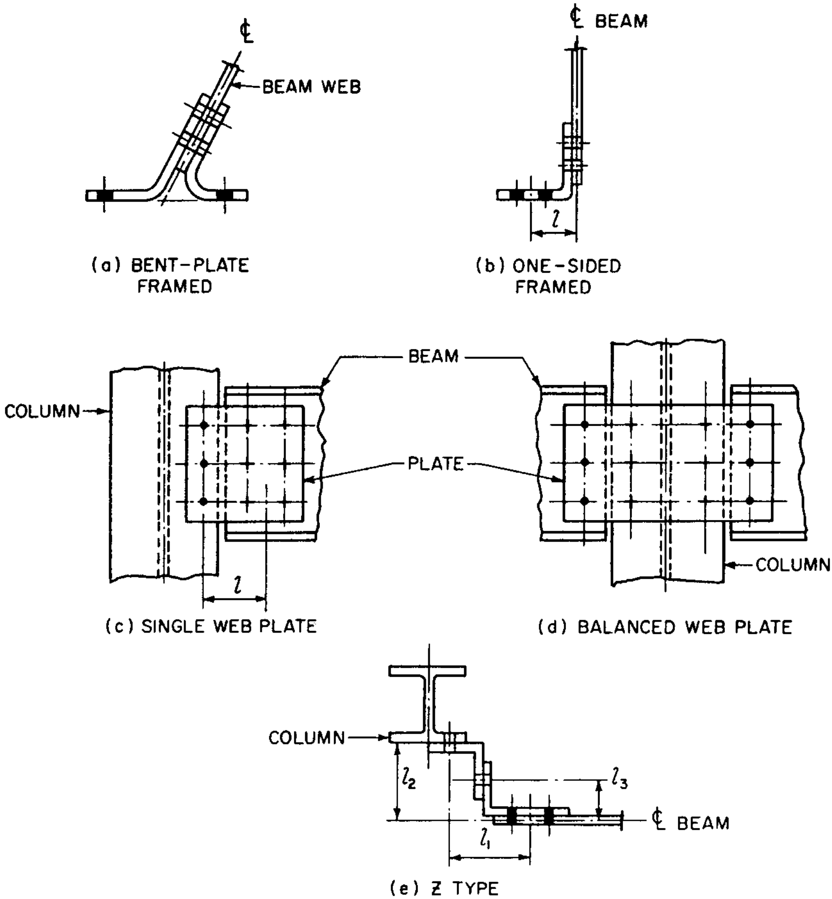


FIGURE 7.51 Examples of special connections.

reactions, the moments balance out. But the case of live load on one beam only must be considered. And bear in mind the necessity of supporting the beam reaction as near as possible to the column center to relieve the column of bending stresses.

When spandrels and girts are offset from the column, a Z-type connection (Fig. 7.51*e*) may be used. The eccentricity for beam-web fasteners should be taken as l_1, for column-flange fasteners as l_2, and for fasteners joining the two connection angles as l_3 when l_3 exceeds 2½ in; smaller values of l_3 may be considered negligible.

7.35.7 Simple, Rigid, and Semirigid Connections

Moment connections are capable of transferring the forces in beam flanges to the column. This moment transfer, when specified, must be provided for in addition to and usually independent of the shear connection needed to support the beam reaction. Framed, seated, and end-plate connections (Arts. 7.35.1 to 7.35.5) are examples of shear connections. Those in Fig. 7.17 (p. 7.32), are moment connections. In Fig. 7.17*a* to *g*, flange stresses are developed independently of the shear connections, whereas in *h* and *i*, the forces are combined and the entire connection resolved as a unit.

Moment connections may be classified according to their design function: those resisting moment due to lateral forces on the structure, and those needed to develop continuity, with or without resistance to lateral forces.

The connections generally are designed for the computed bending moment, which often is less than the beam's capacity to resist moment. A maximum connection is obtained, however, when the beam flange is developed for its maximum allowable stress.

The ability of a connection to resist moment depends on the elastic behavior of the parts. For example, the light lug angle shown connected to the top flange of the beam in Fig. 7.52*b* is not designed for moment and accordingly affords negligible resistance to rotation. In contrast, full rigidity is expected of the direct welded flange-to-column connection in Fig. 7.52*a*. The degree of fixity, therefore, is an important factor in design of moment connections.

Fixity of End Connections. Specifications recognize three types of end connections: simple, rigid, and semirigid. The type designated *simple* (unrestrained) is intended to support beams and girders for shear only and leave the ends free to

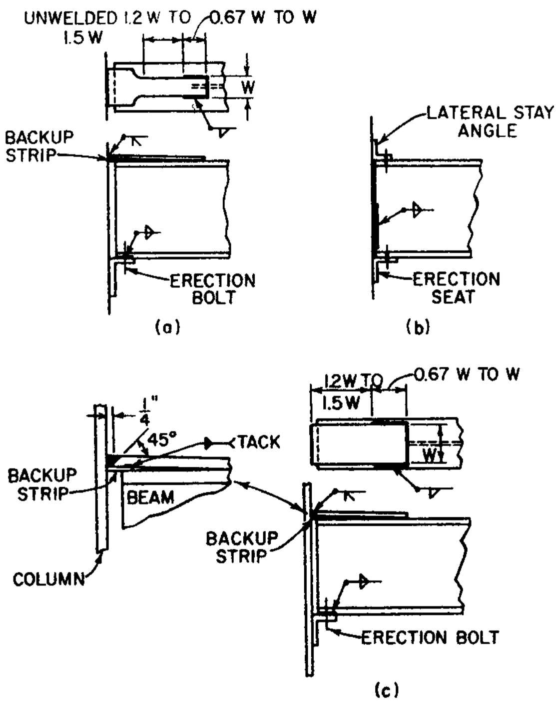


FIGURE 7.52 Methods of constructing flexible welded connections.

rotate under load. The type designated *rigid* (known also as rigid-frame, continuous, restrained frame) aims at not only carrying the shear but also providing sufficient rigidity to hold virtually unchanged the original angles between members connected. *Semirigid,* as the name implies, assumes that the connections of beams and girders possess a dependable and known moment capacity intermediate in degree between the simple and rigid types. Figure 7.54 illustrates these three types together with the uniform-load moments obtained with each type.

Although no definite relative rigidities have been established, it is generally conceded that the simple or flexible type could vary from zero to 15% (some researchers recommend 20%) end restraint and that the rigid type could vary from 90 to 100%. The semirigid types lie between 15 and 90%, the precise value assumed in the design being largely dependent on experimental analysis. These percentages of rigidity represent the ratio of the moment developed by the connection, with no column rotation, to the moment developed by a fully rigid connection under the same conditions, multiplied by 100.

Framed and seated connections offer little or no restraint. In addition, several other arrangements come within the scope of simple-type connections, although they appear to offer greater resistance to end rotations. For example, in Fig. 7.52*a*, a top plate may be used instead of an angle for lateral support, the plate being so designed that plastic deformation may occur in the narrow unwelded portion. Naturally, the plate offers greater resistance to beam rotation than a light angle, but it can provide sufficient flexibility that the connection can be classified as a simple type. Plate and welds at both ends are proportional for about 25% of the beam moment capacity. The plate is shaped so that the metal across the least width is at yield stress when the stresses in the wide portion, in the butt welds, and in the fillet welds are at allowable working values. The unwelded length is then made from 20 to 50% greater than the least width to assure ductile yielding. This detail can also be developed as an effective moment-type connection.

Another flexible type is the direct web connection in Fig. 7.52*b*. Figured for shear loads only, the welds are located on the lower part of the web, where the rotational effect of the beam under load is the least. This is a likely condition when the beam rests on erection seats and the axis of rotation centers about the seat rather then about the neutral axis.

Tests indicate that considerable flexibility also can be obtained with a property proportioned welded top-plate detail as shown in Fig. 7.52*c* without narrowing it as in Fig. 7.52*a*. This detail is usually confined to wind-braced simple-beam designs. The top plate is designed for the wind moment on the joint, at the increased stresses permitted for wind loads.

The problem of superimposing wind bracing on what is otherwise a clear-cut simple beam with flexible connections is a complex one. Some compromise is usually effected between theory and actual design practice. Two alternatives usually are permitted by building codes:

1. Connections designed to resist assumed wind moments should be adequate to resist the moments induced by the gravity loading and the wind loading, at specified increased unit stresses.

2. Connections designed to resist assumed wind moments should be so designed that larger moments, induced by gravity loading under the actual condition of restraint, will be relieved by deformation of the connection material.

Obviously, these options envisage some nonelastic, but self-limiting, deformation of the structural-steel parts. Innumerable wind-braced buildings of riveted, bolted, or welded construction have been designed on this assumption of plastic behavior and have proved satisfactory in service.

Fully rigid, bolted beam end connections are not often used because of the awkward, bulky details, which, if not interfering with architectural clearances, are often so costly to design and fabricate as to negate the economy gained by using smaller beam sections. In appearance, they resemble the type shown in Fig. 7.17 for wind bracing; they are developed for the full moment-resisting capacity of the beam.

Much easier to accomplish and more efficient are welded rigid connections (Fig. 7.53). They may be connected simply by butt welding the beam flanges to the columns—the ''direct'' connection shown in Fig. 7.53*a* and *b*. Others may prefer the ''indirect'' method, with top plates, because this detail permits ordinary mill tolerance for beam length. Welding of plates to stiffen the column flanges, when necessary, is also relatively simple.

In lieu of the erection seat angle in Fig. 7.53*b*, a patented, forged hook-and-eye device, known as Saxe erection units, may be used. The eye, or seat, is shop welded to the column, and the hook, or clip, is shop welded to the underside of the beam bottom flange. For deep beams, a similar unit may be located on the top flange to prevent accidental turning over of the beams. Saxe units are capable of supporting normal erection loads and deadweight of members; but their contribution to the strength of the connection is ignored in computing resistance to shear.

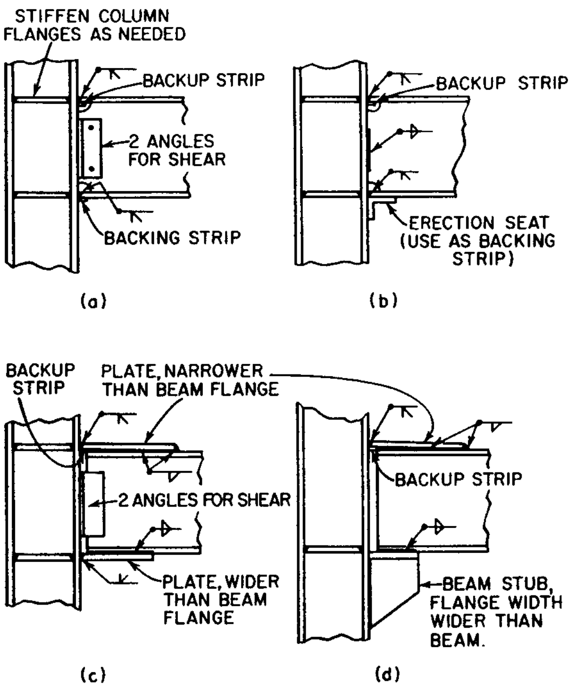


FIGURE 7.53 Methods of constructing welded rigid connections.

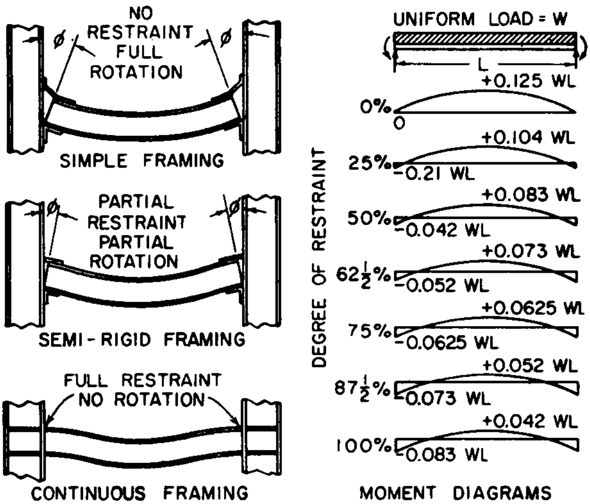


FIGURE 7.54 Effect of rigidity of connections on end moments.

A comparison of fixities intermediate between full rigidity and zero restrain in Fig. 7.54 reveals an optimum condition attainable with 75% rigidity; end and center-span moments are equal, each being $WL/16$, or one-half the simple-beam moment. The saving in weight of beam is quite apparent.

Perhaps the deterrent to a broader usage of semirigid connections has been the proviso contained in specifications: "permitted only upon evidence that the connections to be used are capable of resisting definite moments without overstress of the fasteners." As a safeguard, the proportioning of the beam joined by such connections is predicated upon no greater degree of end restraint than the minimum known to be effected by the connection. Suggested practice, based on research with welded connections, is to design the end connections for 75% rigidity but to provide a beam sized for the moment that would result from 50% restraint; i.e., $WL/12$. ("Report of Tests of Welded Top Plate and Seat Building Connections," *The Welding Journal,* Research Supplement 146S–165S, 1944.) The type of welded connection in Fig. 7.52*c* when designed for the intended rigidity, is generally acceptable.

End-plate connections (Fig. 7.50) are another means of achieving negligible, partial, and full restraint.

7.36 BEAM SPLICES

These are required in rigid frames, suspended-span construction, and continuous beams. Such splices are usually located at points of counterflexure or at points where moments are relatively small. Therefore, splices are of moderate size. Flanges and web may be spliced with plates or butt welded.

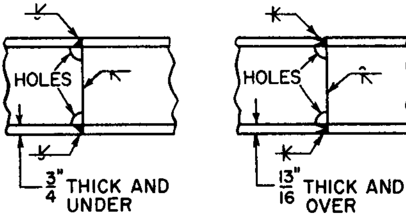


FIGURE 7.55 Welded beam splices.

For one reason or another it is sometimes expedient to make a long beam from two short lengths. A welded joint usually is selected, because the beams can be joined together without splice plates and without loss of section because of bolt holes. Also, from the viewpoint of appearance, the welded joint is hardly discernible.

Usually, the joint must be 100% efficient, to develop the full section. Figure 7.55 illustrates such a detail. The back side of the initial weld is gouged or chipped out; access holes in the beam webs facilitate proper edge preparation and depositing of the weld metal in the flange area in line with the web. Such holes are usually left open, because plugs would add undesirable residual stresses to the joint.

7.37 COLUMN SPLICES

Column-to-column connections are usually determined by the change in section. In general, a change is made at every second floor level, where a shop or field splice is located. From an erection viewpoint, as well as for fabrication and shipment, splices at every third floor may be more economical because of the reduced number of pieces to handle. This advantage is partly offset by extra weight of column material, because the column size is determined by loads on the lowest story of each tier, there being an excess of section for the story or two above.

Splices are located just above floor-beam connections, usually about 2 to 3 ft above the floor. Because column stresses are transferred from column to column by bearing, the splice plates are of nominal size, commensurate with the need for safe erection and bending moments the joint may be subjected to during erection. From the viewpoint of moment resistance, a conventional column splice develops perhaps 20% of the moment capacity of the column.

Figure 7.56 illustrates the common types of column splices made with high strength bolts. In Fig. 7.56*a* and *b*, the upper column bears directly on the lower column; filler plates are supplied in (*b*) when the differences in depth of the two columns are greater than can be absorbed by erection clearance.

As a rule, some erection clearance should be provided. When columns of the same nominal depth are spliced, it is customary to supply a ⅛-in fill under each splice plate on the lower column, or, as an alternate, to leave the bolt holes open on the top gage line below the finished joint until the upper shaft is erected. The latter procedure permits the erector to spring the plates apart to facilitate entry of the upper column.

When the upper column is of such dimension that its finished end does not wholly bear on the lower column, one of two methods must be followed: In Fig. 7.56*c*, stresses in a portion of the upper column not bearing on the lower column are transferred by means of flange plates that are finished to bear on the lower column. These bearing plates must be attached with sufficient single-shear bolts to develop the load transmitted through bearing on the finished surface.

When the difference in column size is pronounced, the practice is to use a horizontal bearing plate as shown in Fig. 7.56*d*. These plates, known as **butt plates,**

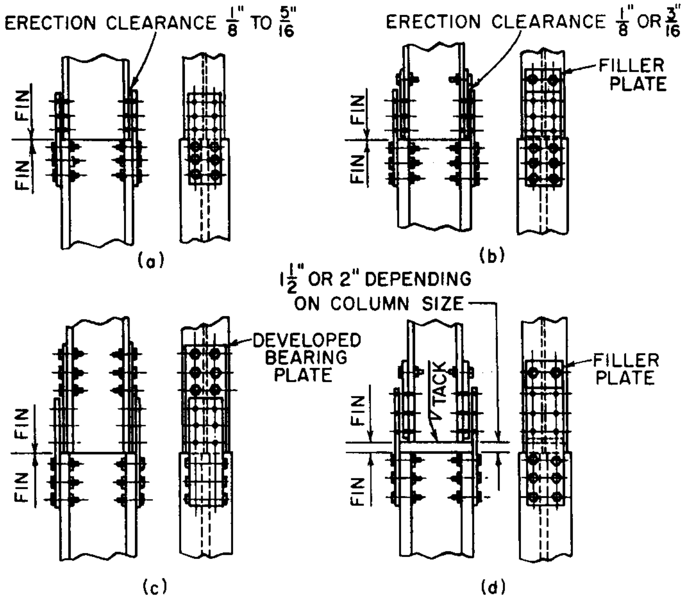


FIGURE 7.56 Slip-critical bolted column splices.

may be attached to either shaft with tack welds or clip angles. Usually it is attached to the upper shaft, because a plate on the lower shaft may interfere with erection of the beams that frame into the column web.

Somewhat similar are welded column splices. In Fig. 7.57*a*, a common case, holes for erection purposes are generally supplied in the splice plates and column flanges as shown. Some fabricators, however, prefer to avoid drilling and punching of thick pieces, and use instead clip angles welded on the inside flanges of the columns, one pair at diagonally opposite corners, or some similar arrangement, Figure 7.57*b* and *c* corresponds to the bolted splices in Fig. 7.56*c* and *d*. The shop and field welds for the welded butt plate in Fig. 7.57*c* may be reversed, to provide erection clearance for beams seated just below the splice. The erection clip angles would then be shop welded to the underside of the butt plate, and the field holes would pierce the column web.

The butt-weld splice in Fig. 7.57*d* is the most efficient from the standpoint of material saving. The depth of the bevel as given in the illustration is for the usual column splice, in which moment is unimportant. However, should the joint be subjected to considerable moment, the bevel may be deepened; but a ⅛-in minimum shoulder should remain for the purpose of landing and plumbing the column. For full moment capacity, a complete-penetration welded joint would be required.

STEEL ERECTION

A clear understanding of what the fabricator furnishes or does not furnish to the erector, particularly on fabrication contracts that may call for delivery only, is all-

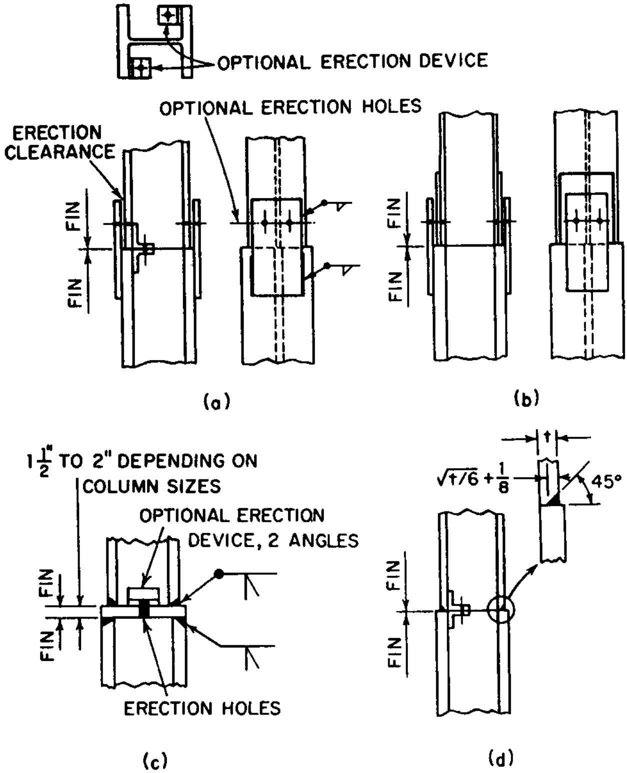


FIGURE 7.57 Welded column splices.

important—and in many instances fabricated steel is purchased on delivery basis only.

Purchasing structural steel is simplified by the "Code of Standard Practice for Buildings and Bridges," (Table 7.1). A provision in the construction contract making the code a part of the contract is often used, since it establishes a commonly accepted and well-defined line of demarcation between what is, and what is not, to be furnished under the contract. Lacking such a provision, the contract, to avoid later misunderstandings, must enumerate in considerable detail what is expected of both parties to the contract.

Under the code—and unless otherwise specifically called for in the contract documents—such items as steel sash, corrugated-iron roofing or siding, and open-web steel joists, and similar items, even if made of steel and shown on the contract design drawings, are not included in the category "structural steel." Also, such items as door frames are excluded, even when made of structural shapes, if they are not fastened to the structure in such way as to comply with "constituting part of the steel framing." On the other hand, loose lintels shown on design plans or in separate scheduling are included.

According to the code, a fabricator furnishes with "structural steel," to be erected by someone else, the field bolts required for fastening the steel. The fabricator, however, does not furnish the following items unless specified in the invitation to bid: shims, fitting-up bolts, drift pins, temporary cables, welding electrodes, or thin leveling plates for column bases.

The code also defines the erection practices. For example, the erector does not paint field boltheads and nuts, field welds, or touch up abrasions in the shop coat, or perform any other field painting unless required in specifications accompanying the invitation to bid.

7.38 ERECTION EQUIPMENT

If there is a universal piece of erection equipment, it is the crane. Mounted on wheels or tractor threads, it is extremely mobile, both on the job and in moving from job to job. Practically all buildings are erected with this efficient raising device. The exception, of course, is the skyscraper whose height exceeds the reach of the crane. Operating on ground level, cranes have been used to erect buildings of about 20 stories, the maximum height being dependent on the length of the boom and width of building.

The guy derrick is a widely used raising device for erection of tall buildings. Its principal asset is the ease by which it may be "jumped" from tier to tier as erection proceeds upward. The boom and mast reverse position; each in turn serves to lift up the other. It requires about 2 h to make a two-story jump.

Stiff-leg derricks and gin poles are two other rigs sometimes used, usually in the role of auxiliaries to cranes or guy derricks. Gin poles are the most elementary—simply a guyed boom. The base must be secure because of the danger of kicking out. The device is useful for the raising of incidental materials, for dismantling and lowering of larger rigs, and for erection of steel on light construction where the services of a crane are unwarranted.

Stiff-leg derricks are most efficient where they may be set up to remain for long periods of time. They have been used to erect multistory buildings but are not in popular favor because of the long time required to jump from tier to tier. Among the principal uses for stiff legs are (1) unloading steel from railroad cars for transfer to trucks, (2) storage and sorting, and (3) when placed on a flat roof, raising steel to roof level, where it may be sorted and placed within each of a guy derrick.

Less time for "jumping" the raising equipment is needed for cranes mounted on steel box-type towers, about three stories high, that are seated on interior elevator wells or similar shafts for erecting steel. These tower cranes are simply jacked upward hydraulically or raised by cables, with the previously erected steel-work serving as supports. In another method, a stiff-leg derrick is mounted on a trussed platform, spanning two or more columns, and so powered that it can creep up the erected exterior columns. In addition to the advantage of faster jumps, these methods permit steel erection to proceed as soon as the higher working level is reached.

7.39 CLEARANCE FOR ERECTING BEAMS

Clearances to permit tightening bolts and welding are discussed in Art. 7.3.7. In addition, designers also must provide sufficient field clearance for all members so as to permit erection without interference with members previously erected. The

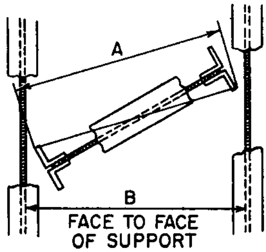


FIGURE 7.58 Erection clearance for beams.

shop drafter should always arrange the details so that the members can be swung into their final position with shifting the members to which they connect from their final positions. The following examples illustrate the conditions most frequently encountered in building work:

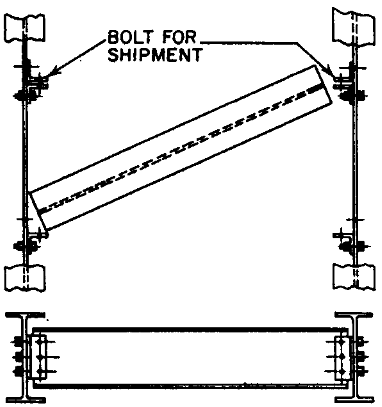


FIGURE 7.59 Alternative method for providing erection clearance.

In framed beam connections (Fig. 7.58), the slightly shorter distance out-to-out of connection angles (B—⅛ in), as compared with the face-to-face distance between supporting members, is usually sufficient to allow forcing the beam into position. Occasionally, however, because the beam is relatively short, or because heavy connection angles with wide outstanding legs are required, the diagonal distance A may exceed the clearance distance B. If so, the connection for one end must be shipped bolted to the framed beam to permit its removal during erection.

An alternative solution is to permanently fasten on connection angle of each pair to the web of the supporting beam, temporarily bolting the other angle to the same web for shipment, as shown in Fig. 6.59. The beam should be investigated for the clearance in swinging past permanently bolted connection angles. Attention must also be paid to possible interference of stiffeners in swinging the beam into place when the supporting member is a plate girder.

Another example is that of a beam seated on column-web connections (Fig. 7.60). The first step is to remove the top angles and shims temporarily. Then, while hanging from the derrick sling, the beam is tilted until its ends clear the edges of the column flanges, after which it is rotated back into a horizontal position and landed on the seats. The greatest diagonal length G of the beam should be about ⅛ in less than the face-to-face distance F between column webs. It must also be such as to clear any obstruction above; e.g., G must be equal to or less than C, or the obstructing detail must be shipped bolted for temporary removal. To allow for possible overrun, the ordered length L of the beam should be less than the detailing length E by at least the amount of the permitted cutting tolerance.

Frequently, the obstruction above the beam connection may be the details of a column splice. As stated in Art. 7.37, it may be necessary to attach the splice

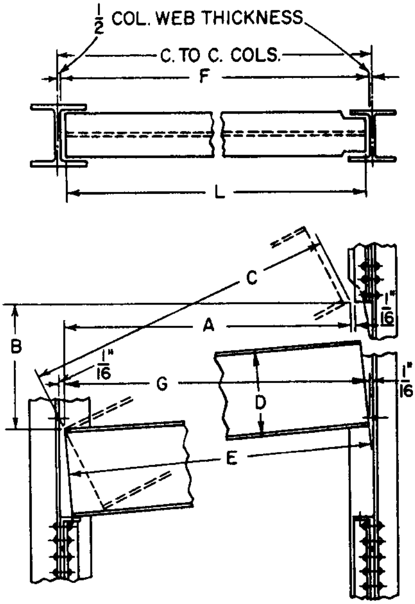


FIGURE 7.60 Clearance for beam seated on column-web connections.

material on the lower end of the upper shaft, if erection of the beam precedes erection of the column in the tier above.

7.40 ERECTION SEQUENCE

The order in which steel is to be fabricated and delivered to the site should be planned in advance so as not to conflict with the erector's methods or construction schedule. For example, if steel is to be erected with derricks, the approximate locations at which the derricks will be placed will determine the shipping installments, or sections, into which the frame as a whole must be segregated for orderly shipment. When installments are delivered to the site at predetermined locations, proper planning will eliminate unnecessary rehandling. Information should be conveyed to the drafting room so that the shipping installments can be indicated on the erection plans and installments identified on the shipping lists.

In erection of multistory buildings with guy derricks, the practice is to hoist and place all columns in each story first, spandrel beams and wall bracing next, and interior beams and wall bracing next, and interior beams with filler beams last. More specifically, erection commences with bays most distant from the derrick and progresses toward the derrick, until it is closed in. Then, the derrick is jumped to the top and the process is repeated for the next tier. Usually, the top of the tier is planked over to obtain a working platform for the erectors and also to afford protection for the trades working below. However, before the derrick is jumped, the corner panels are plumbed; similarly when panels are erected across the building, cables are stretched to plumb the structure.

There is an established sequence for completing the connections. The raising gang connects members together with temporary fitting-up bolts. The number of bolts is kept to a minimum, just enough to draw the joint up tight and take care of the stresses caused by deadweight, wind, and erection forces. Permanent connections are made as soon as alignment is within tolerance limits. Usually, permanent bolting or welding follows on the heels of the raising gang. Sometimes, the latter moves faster than the gang making the permanent connections, in which case it may be prudent to skip every other floor, thus obtaining permanent connections as close as possible to the derrick—a matter of safe practice.

Some erectors prefer to use permanent high-strength (A325 and A490) bolts for temporary fitting up. Because bolts used for fit-up are not tightened to specified minimum tension, they may be left in place and later tightened as required for permanent installation.

7.41 FIELD-WELDING PROCEDURES

The main function of a welding sequence is to control distortion due primarily to the effects of welding heat. In general, a large input of heat in a short time tends to produce the greatest distortion. Therefore, it is always advisable, for large joints, to weld in stages, with sufficient time between each stage to assure complete dispersal of heat, except for heat needed to satisfy interpass-temperature requirements (Art. 7.3.5). Equally important, and perhaps more efficient from the erector's viewpoint, are those methods that balance the heat input in such a manner that the distortional effects tend to cancel out.

Welding on one flange of a column tends to leave the column curled toward the welded side cooling, because of shrinkage stresses. A better practice for beams connecting to both sides of a column is to weld the opposite connections simultaneously. Thus the shrinkage of each flange is kept in balance and the column remain plumb.

If simultaneous welding is not feasible, then the procedure is to weld in stages. About 60% of the required weld might be applied on the first beam, then the joint on the opposite flange might be completely welded, and finally, welded on the first beam would be completed. Procedures such as this will go far to reduce distortion.

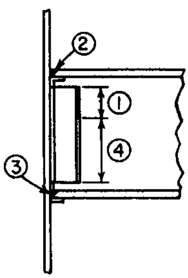


FIGURE 7.61 Indication of sequence in welding a connection.

Experience has shown that it is good practice to commence welding at or near the center of a building and work outward. Columns should be checked frequently for vertical alignment, because shrinkage in the welds tends to shorten the distance between columns. Even though the dimensional change at each joint may be very small, it can accumulate to an objectionable amount in a long row of columns. One way to reduce the distortion is to allow for shrinkage at each joint, say, $\frac{1}{16}$ in for a 20-ft bay, by tilting or spreading the columns. Thus, a spread of $\frac{1}{8}$ in for the two ends of a beam with flanges butt welded to the columns may be built in at the fabricating shop; for example, by increasing the spacing of erection-bolt holes in the beam bottom flange. Control in the field, however, is maintained by guy wires until all points are welded.

Shortening of bays can become acute in a column row in which beams connect to column flanges, because the shrinkage shortening could possibly combine with the mill underrun in column depths. Occasionally, in addition to spreading the columns, it may be necessary to correct the condition by adding filler plates or building out with weld metal.

Some designers of large welded structures prefer to detail the welding sequence for each joint. For example, on one project, the procedure for the joint shown in Fig. 7.61 called for four distinct operations, or stages: first, the top 6 inches of the shear weld on the vertical connection was made; second, the weld on the top flange; third, the bottom-flange weld; and fourth, the remaining weld of the vertical connection. The metal was allowed to return to normal temperature before starting each stage. One advantage of this procedure is the prestressing benefits obtained in the connecting welds. Tensile stresses are developed in the bottom-flange weld on cooling; compressive stresses of equal magnitude consequently are produced in the top flange. Since these stresses are opposite to those caused by floor loads, welding stresses are useful in supporting the floor loads. Although this by-product assistance may be worthwhile, there are no accepted methods for resolving the alleged benefits into design economy.

Multistory structures erected with equipment supported on the steelwork as it rises will be subjected by erection loads to stresses and strains. The resulting deformations should be considered in formulating a field-welding sequence.

7.42 ERECTION TOLERANCES

Dimensional variations in the field often are a consequence of permissible variations in rolling of steel and in shop fabrication. Limits for mill variations 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 per 10 ft, with a maximum deviation of ⅜ in.

It is standard practice to compensate in shop details for certain mill variations. The adjustments are made in the field, usually with clearances and shims.

Shop-fabrication tolerance for straightness of columns and other compression members often is expressed as a ratio, 1:1000, between points of lateral support. (This should be recognized as approximately the equivalent of ⅛ in per 10 ft, and since such members rarely exceed 30 ft in length, between lateral supports, the ⅜-in maximum deviation prevails.) Length of fabricated beams have a tolerance of 1⁄16 in up to 30 ft and ⅛ in over 30 ft. Length of columns finished to bear on their ends have a tolerance of 1⁄32 in.

Erected beams are considered level and aligned if the deviation does not exceed 1:500. Similarly, columns are plumb and aligned if the deviation of individual pieces, between splices in the usual multistory building, does not exceed 1:500. The total or accumulative displacement for multistory columns cannot exceed the limits prescribed in the American Institute of Steel Construction "Code of Standard Practice." For convenience, these are indicated in Fig. 7.62. Control is placed only on the exterior columns and those in the elevator shaft.

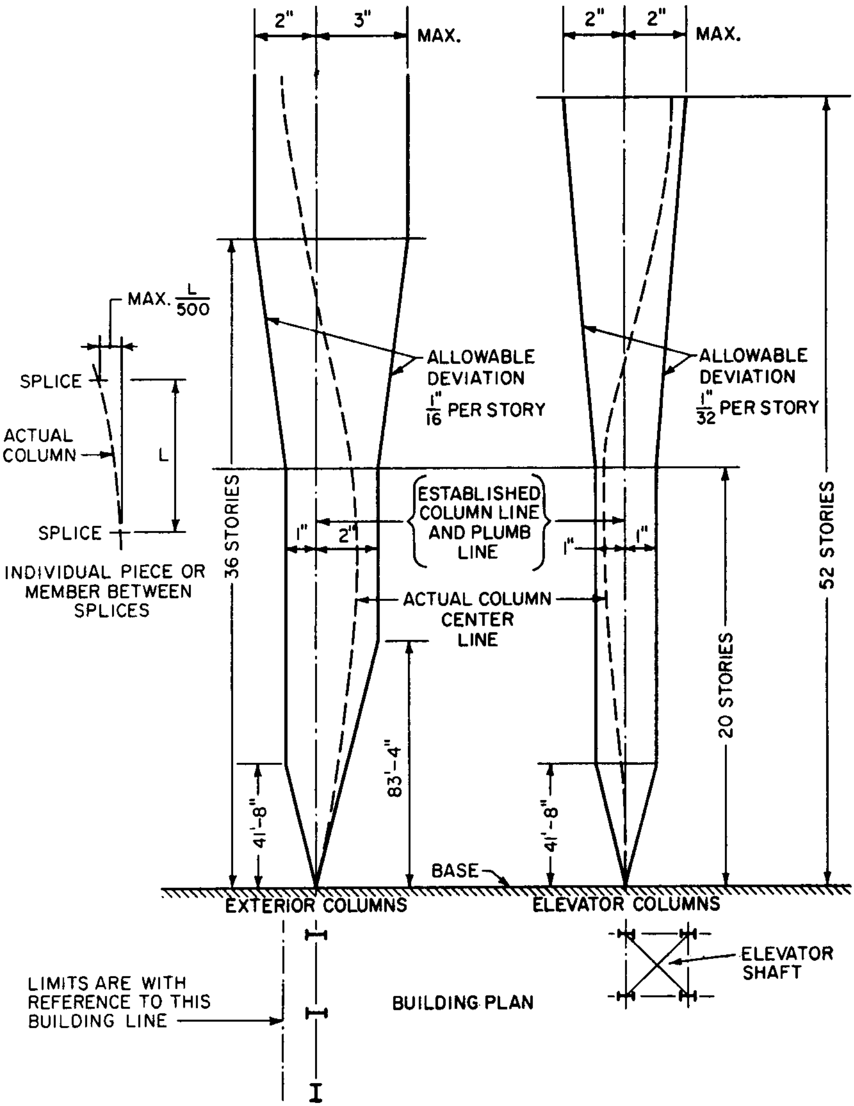


FIGURE 7.62 Permissible deviations from plumb for columns. Limits shown are based on the assumption that the center of the column base coincides with the established column line.

Field measurements to determine whether columns are plumb should always be made at night or on cloudy days, never in sunshine. Solar radiation induces differential thermal strains, which cause the structure to curl away from the sun by an amount that renders plumbing measurements useless.

If beam flanges are to be field welded (Fig. 7.56*a*) and the shear connection is a high-strength-bolted, slip-critical joint, the holes should be made oversize or horizontal slotted (Art. 7.3.1), thus providing some built-in adjustment to accommodate mill and shop tolerances for beams and columns.

Similarly, for beams with framed connections (Fig. 7.46 and 7.47) that will be field bolted to columns, allowance should be made in the details for finger-type shims, to be used where needed for column alignment.

Because of several variables, bearing of column joints is seldom in perfect contact across the entire cross-sectional area. The AISC recommends acceptance if gaps between the bearing surfaces do not exceed 1⁄16 in. Should a gap exceed 1⁄16 in and an engineering investigation shows need for more contact area, the gap may be filled with mild steel shims.

Tolerance for placing machinery directly on top of several beams is another problem occasionally encountered in the field. The elevation of beam flanges will vary because of permissible variations for mill rolling, fabrication, and erection. This should be anticipated and adequate shims provided for field adjustments.

7.43 ADJUSTING LINTELS

Lintels supported on the steel frame (sometimes called shelf angles) may be permanently fastened in the shop to the supporting spandrel beam, or they may be attached so as to allow adjustment in the field (see Fig. 7.9, p. 7.21). In the former case, the final position is solely dependent on the alignment obtained for the spandrel itself, whereas for the latter, lintels may be adjusted to line and grade independently of the spandrel. Field adjustment is the general rule for all multistory structures. Horizontal alignment is obtained by using slotted holes in the connection clip angles. Vertical elevation (grade) is obtained with shims.

When walls are of masonry construction, a reasonable amount of variation in the position of lintels may be absorbed without much effort by masons. So the erector can adjust the lintels immediately following the permanent fastening of the spandrels to the columns. This procedure is ideal for the steel erector, because it allows him to complete his contract without costly delays and without interference with other trades. Subsequent minor variations in the position of the lintels, because of deflection or torsional rotation of the spandrel when subjected to deadweight of the floor slab, are usually absorbed without necessitating further lintel adjustment.

With lightweight curtain walls, however, the position of the lintels is important, because large paneled areas afford less latitude for variation. As a rule, the steel erector is unable to adjust the lintels to the desired accuracy at the time the main framework is erected. If the erector has contracted to do the adjusting, this work must wait until the construction engineer establishes the correct lines and grades. In the usual case, floor slabs are concreted immediately after the steelwork is inspected and accepted. The floor grades then determined become the base to which the lintels can be adjusted. At about the same time, the wall contractor has scaffolds in place, and by keeping pace with wall construction, the steel erector, working from the wall scaffolds, adjusts the lintels.

In some cases, the plans call for concrete encasement of the spandrel beams, in which case concreting is accomplished with the floor slab. The construction engineer should ensure that the adjustment features provided for the lintels are not frozen in the concrete. One suggestion is to box around the details, thus avoiding chopping out concrete. In some cases, it may be possible to avoid the condition entirely by locating the connection below the concrete encasement, where the adjustment is always accessible.

The whole operation of lintel adjustment is one of coordination between the several trades. That this be carried out in an orderly fashion is the duty of the construction engineer. Furthermore, the desired procedure should be carefully spelled out in the job specifications so that erection costs can be estimated fairly.

Particularly irksome to the construction engineer is the lintel located some distance below the spandrel and supported on flexible, light steel hangers. This detail can be troublesome because it has no capacity to resist torsion. Avoid this by developing the lintel and spandrel to act together as a single member.

CORROSION PROTECTION

Protection of steel surfaces has been, since the day steel was first used, a vexing problem for the engineers, paint manufacturers, and maintenance personnel. Over the years, there have been many developments, the result of numerous studies and research activities. Results are published in the ''Steel Structures Painting Manual.'' This work is in two volumes—Vol. 1, ''Good Painting Practice,'' and Vol. II, ''Systems and Specifications'' (Steel Structures Painting Council, 40 24th Street, Suite 600, Pittsburgh, PA 15213). Each of the paint systems covers the method of cleaning surfaces, types of paint to be used, number of coats to be applied, and techniques to be used in their applications. Each surface treatment and paint system is identified by uniform nomenclature, e.g., Paint System Specification SSPC-PS7.00-64T, which happens to be the identity of the minimum-type protection as furnished for most buildings.

7.44 CORROSION OF STEEL

Ordinarily, steel corrodes in the presence of both oxygen and water, but corrosion rarely takes place in the absence of either. For instance, steel does not corrode in dry air, and corrosion is negligible when the relative humidity is below 70%, the critical humidity at normal temperature. Likewise, steel does not corrode in water that has been effectively deaerated. Therefore, the corrosion of structural steel is not a serious problem, except where water and oxygen are in abundance and where these primary prerequisites are supplemented with corrosive chemicals such as soluble salts, acids, cleaning compounds, and welding fluxes.

In ideal dry atmosphere, a thin transparent film of iron oxide forms. This layer of ferric oxide is actually beneficial, since it protects the steel from further oxidation.

When exposed to water and oxygen in generous amounts, steel corrodes at an average rate of roughly 5 mils loss of surface metal per year. If the surface is comparatively dry, the rate drops to about ½ mil per year after the first year, the usual case in typical industrial atmospheres. Excessively high corrosion rates occur only in the presence of electrolytes or corrosive chemicals. Usually, this condition is found in localized areas of a building.

Mill scale, the thick layer of iron oxides that forms on steel during the rolling operations, is beneficial as a protective coating, if it is intact and adheres firmly to the steel. In the mild environments generally encountered in most buildings, mill scale that adheres tightly after weathering and handling offers no difficulty. In buildings exposed to high humidity and corrosive gases, broken mill scale may be detrimental to both the steel and the paint. Through electrochemical action, corrosion sets in along the edges of the cracks in the mill scale and in time loosens the scale, carrying away the paint.

Galvanic corrosion takes place when dissimilar metals are connected together. Noble metals such as copper and nickel should not be connected to structural steel with steel fasteners, since the galvanic action destroys the fasteners. On the other hand, these metals may be used for the fasteners, because the galvanic action is distributed over a large area and consequently little or no harm is done. When dissimilar metals are to be in contact, the contacting surfaces should be insulated; paint is usually satisfactory.

7.45 PAINTING STEEL STRUCTURES

Evidence obtained from dismantled old buildings and from frames exposed during renovation indicates that corrosion does not occur when steel surfaces are protected from the atmosphere. Where severe rusting was found and attributed to leakage of water, presence or absence of shop paint had no significant influence. Consequently, the AISC ''Specifications for Structural Steel for Buildings'' exempts from one-coat shop paint, at one time mandatory, all steel framing that is concealed by interior finishing materials—ceilings, fireproofing partitions, walls, and floors.

Structures may be grouped as follows: (1) those that need no paint, shop or field; (2) those in which interior steelwork will be exposed, probably field painted; (3) those fully exposed to the elements. Thus, shop paint is required only as a primer coat before a required coat of field paint.

Group (1) could include such structures as apartment buildings, hotels, dormitories, office buildings, stores, and schools, where the steelwork is enclosed by other materials. The practice of omitting the shop and field paint for these structures, however, may not be widely accepted because of tradition and the slowness of building-code modernization. Furthermore, despite the economic benefit of paint omission, clean, brightly painted steel during construction has some publicity value.

In group (2) are warehouses, industrial plants, parking decks, supermarkets, one-story schools, inside swimming pools, rinks, and arenas, all structures shielded from the elements but with steel exposed in the interior. Field paint may be required for corrosion protection or appearance or both. The severity of the corrosion environment depends on type of occupancy, exposure, and climatic conditions. The paint system should be carefully selected for optimum effectiveness.

In group (3) are those structures exposed at all times to the weather: crane runways, fire escapes, towers, exposed exterior columns, etc. When made of carbon steel, the members will be painted after erection and therefore should be primed with shop paint. The paint system selected should be the most durable one for the atmospheric conditions at the site. For corrosion-resistant steels, such as those meeting ASTM A242 and A588, field painting may be unnecessary. On exposure, these steels acquire a relatively hard coat of oxide, which shields the surface from progressive rusting. The color, russet brown, has architectural appeal.

7.46 PAINT SYSTEMS

The Steel Structures Painting Council has correlated surface preparations and primer, intermediate, and finish coats of paints into systems, each designed for a common service condition (‘‘Steel Structure Painting Manual’’). In addition, the Council publishes specifications for each system and individual specifications for surface preparations and paints. Methods for surface cleaning include solvent, hand-tool, power-tool, pickling, flame, and several blast techniques.

Surface preparation is directly related to the type of paints. In general, a slow-drying paint containing oil and rust-inhibitive pigments and one possessing good wetting ability may 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. Therefore, in specifying a particular paint, the engineer should include the type of surface preparation, to prevent an improper surface condition from reducing the effectiveness of an expensive paint.

Paint selection and surface preparation are a matter of economics. For example, while blast-cleaned surfaces are concealed to be the best paint foundation for lasting results, the high cost is not always justified. Nevertheless, the Council specifies a minimum surface preparation by a blast cleaning process for such paints as alkyd, phenolic, vinyl, coal tar, epoxy, and zinc-rich.

As an aid for defining and evaluating the various surface preparations, taking into account the initial condition of the surface, an international visual standard is available and may be used. A booklet of realistic color photographs for this purpose can be obtained from the Council or ASTM. The applicable standard and acceptance criteria are given in ‘‘Quality Criteria and Inspection Standards,’’ American Institute of Steel Construction.

The Council stresses the relationship between the prime coat (shop paint) and the finish coats. A primer that is proper for a particular type of field paint could be an unsatisfactory base for another type of field paint. Since there are numerous paint formulations, refer to Council publications when faced with a painting condition more demanding than ordinary.

In the absence of specific contract requirements for painting, the practice described in the AISC ‘‘Specification for Structural Steel for Buildings’’ may be followed. This method may be considered ‘‘nominal.’’ The steel is brushed, by hand or power, to remove loose mill scale, loose rust, weld slag, flux deposit, dirt, and foreign matter. Oil and grease spots are solvent cleaned. The shop coat is a commercial-quality paint applied by brushing, dipping, roller coating, flow coating, or spraying to a 2-mil thickness. It affords only short-time protection. Therefore, finished steel that may be in ground storage for long periods or otherwise exposed to excessively corrosive conditions may exhibit some paint failure by the time it is erected, a condition beyond the control of the fabricator. Where such conditions can be anticipated, as for example, an overseas shipment, the engineer should select the most effective paint system.

7.47 FIELD-PAINTING STEEL

There is some question as to justification for protecting steelwork embedded in masonry or in contact with exterior masonry walls built according to good workmanship standards but not impervious to moisture. For example, in many instances, the masonry backing for a 4-in brick wall is omitted to make way for column flanges. Very definitely, a 4-in wall will not prevent penetration of water. In many cases, also, though a gap is provided between a wall and steelwork, mortar drippings fall into the space and form bridges over which water may pass, to attack the steel. The net effect is premature failure of both wall and steel. Walls have been shattered—sheared through the brick—by the powerful expansion of rust formations. The preventatives are: (1) coating the steel with suitable paint and (2) good wall construction.

A typical building code reads: ''Special precautions shall be taken to protect the outer surfaces of steel columns located in exterior walls against corrosion, by painting such surfaces with waterproof paints, by the use of mastic, or by other methods of waterproofing approved by the building inspector.''

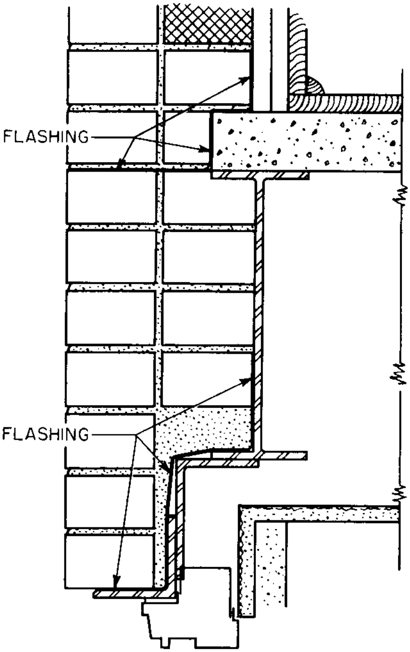


FIGURE 7.63 Flashing at spandrel and lintels.

In most structures an asphalt-type paint is used for column-flange protection. The proviso is sometimes extended to include lintels and spandrels, since the danger of corrosion is similar, depending on the closeness and contact with the wall. However, with the latter members, it is often judicious to supplement the paint with flashing, either metallic or fabric. A typical illustration, taken from an actual apartment-building design, is shown in Fig. 7.63.

In general, building codes differ on field paint; either paint is stipulated or the code is silent. From a practical viewpoint, the question of field painting cannot be properly resolved with a single broad rule. For an enclosed building in which the structural members are enveloped, for example, a field coat is sheer wastage, except for exterior steel members in contact with walls. On the other hand, exposed steel subject to high-humidity atmospheres and to exceptionally corrosive gases and contaminants may need two or three field coats.

Manufactured buildings should always be closely scrutinized, bearing in mind that original conditions are not always permanent. As manufacturing processes change, so do the corrosive environments stimulated by new methods. It is well to prepare for the most adverse eventuality.

Special attention should be given to steel surfaces that become inaccessible, e.g., tops of purlins in contact with roof surfaces. A three-coat job of particularly suitable paint may pay off in the long run, even though it delays placement of the roof covering.

7.48 STEEL IN CONTACT WITH CONCRETE

According to the ''Steel Structures Painting Manual,'' Vol. I, ''Good Painting Practice'' (Steel Structures Painting Council, 40 24th Street, Suite 600, Pittsburgh, PA 15213):

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 that is encased in 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 when 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 be built 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.

FIRE PROTECTION FOR STRUCTURAL STEEL

Structural steel is a noncombustible material. It is therefore satisfactory for use without protective coverage in many types of buildings where combustibility loading is low, from the viewpoint of either building ordinances or owner's preference. When structural steel is used in this fashion, it is described as "exposed" or "unprotected." Unprotected steel may be selected wherever building codes permit combustible construction.

Exposed or unprotected structural steel is commonly used for industrial-type buildings, hangars, auditoriums, stadiums, warehouses, parking garages, billboards, towers, and low stores, schools, and hospitals. In most cases, these structures contain little combustible material. In others, where the contents are highly combustible, sprinkler systems may be incorporated to protect the steelwork.

Steel building frames and floor systems should be covered with fire-resistant materials in certain buildings to reduce the chance of fire damage. These structures may be tall buildings, such as offices, apartments, and hotels, or low-height buildings, such as warehouses, where there is a large amount of combustible content. The buildings may be located in congested areas, where the spread of fire is a strong possibility. So for public safety, as well as to prevent property loss, building codes regulate the amount of fire resistance that must be provided.

The following are some of the factors that enter into the determination of minimum fire resistance for a specific structure: height, floor area, type of occupancy (a measure of combustible contents), fire-fighting apparatus, sprinkler systems, and location in a community (fire zone), which is a measure of hazard to adjoining properties.

7.49 EFFECT OF HEAT ON STEEL

A moderate rise in temperature of structural steel, say up to 500°F, is beneficial in that the strength is about 10% greater than the normal value. Above 500°F, strength falls off, until at 700°F it is nearly equal to the normal temperature strength. At a temperature of 1000°F, the compressive strength of steel is about the same as the maximum allowable working stress in columns.

Unprotected steel members have a rating of about 15 min, based on fire tests of columns with cross-sectional areas of about 10 in^2. Heavier column, possessing greater mass for dissipation of heat, afford greater resistance—20 min perhaps. Columns with reentrant space between flanges filled with concrete, but otherwise exposed, have likewise been tested. Where the total area of the solid cross section approximates 36 in^2, the resistance is 30 min, and where the area is 64 in^2, the resistance is 1 hr.

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

$$C = 0.0000061 + 0.0000000019t \tag{7.81}$$

in which C = coefficient of expansion per °F and t = temperature, °F.

Below 100°F, the average coefficient of expansion is taken as 0.0000065.

The modulus of elasticity of structural steel, about 29,000 ksi at room temperature, decreases linearly to 25,000 ksi at 900°F. Then, it drops at an increasing rate at higher temperatures.

7.50 FIRE PROTECTION OF EXTERIOR

Steel members, such as spandrel beams and columns, on the exterior of a building may sometimes be left exposed or may be protected in an economical manner from fire damage, whereas interior steel members of the same building may be required to be protected with more expensive insulating materials, as discussed in Art. 7.51. Standard fire tests for determining fire-endurance ratings of exterior steel members are not available. But from many tests, data have been obtained that provide a basis for analytical, thermodynamic methods for fire-safe design. (See for example, ''Fire-Safe Structural Steel—A Design Guide,'' American Iron and Steel Institute, 1101 17th St., N.W., Washington, DC 20036.)

The tests indicate that an exterior steel spandrel beam with its interior side protected by fire-resistant construction need only have its flanges fire protected. This may be simply done by application of fireproofing, such as sprayed-on mineral fibers, to the upper surface of the top flange and the under surface of the bottom flange. In addition, incombustible flame-impingement shields should enclose the flanges to deflect flames that may be emitted through windows. The shields, for example, may be made of ¼-in-thick weathering steel. This construction prevents the temperature of the spandrel beam from reaching a critical level.

Exposed-steel columns on the outside of a building may be made fire safe by placement at adequate distances from the windows. Such columns may also be located closer to the building when placed on the side of windows at such distances that the steel is protected by the building walls against flame impingement. Thermodynamic analysis can indicate whether or not the chosen locations are fire safe.

7.51 MATERIALS FOR IMPROVING FIRE RESISTANCE

Structured steel may be protected with any of many materials—brick, stone, concrete, gypsumboard, gypsum block, sprayed-on mineral fibers, and various fire-resistant plasters.

Concrete insulation serves well for column protection, in that it gives additional stability to the steel section. Also, it is useful where abrasion resistance is needed. Concrete, however, is not an efficient insulating medium compared with fire-resistant plasters. Normally, it is placed completely around the columns, beams, or girders, with all reentrant spaces filled solid (Fig. 7.64*a*). Although this procedure contributes to the stability of columns and effects composite action in beams and slabs, it has the disadvantage of imposing great weight on the steel frame and foundations. For instance, full protection of a W12 column with stone concrete weighs about 355 psf, whereas plaster protection weighs about 40 psf, and lightweight concretes made with such aggregates as perlite, vermiculite, expanded shale, expanded slag, pumice, pumicite and sintered flyash weigh less than 100 psf.

Considerable progress has been made in the use of lightweight plasters with aggregates possessing good insulating properties. Two aggregates used extensively are perlite and vermiculite. They replace sand in the sanded-gypsum plaster mix.

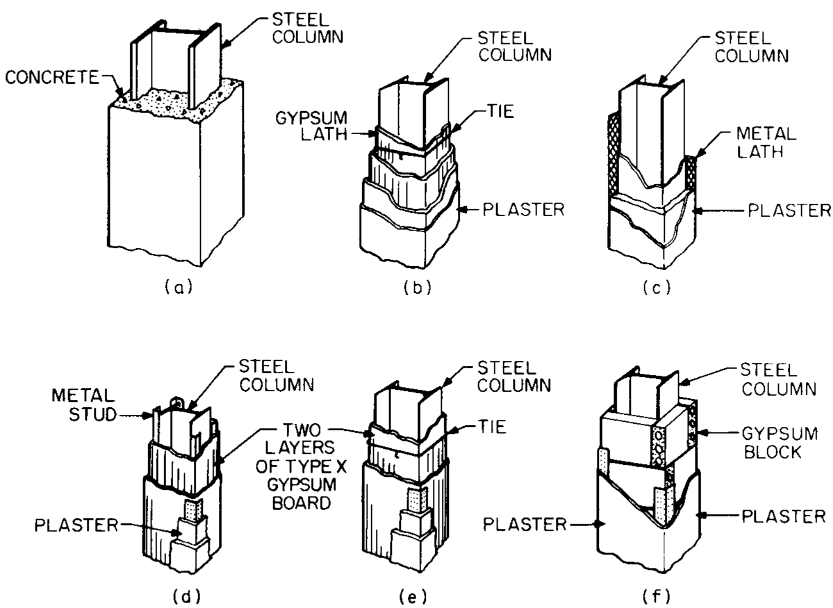


FIGURE 7.64 Fire protection of steel columns by encasement with (*a*) concrete, (*b*) plaster on gypsum lath, (*c*) plaster on metal lath, (*d*) furring and gypsumboard, (*e*) gypsumboard without furring, and (*f*) gypsum block and plaster.

A 1-in thickness weighs about 4 psf, whereas the same thickness of sanded-gypsum plaster weighs about 10 psf.

Typical details of lightweight plaster protection for columns are shown in Fig. 7.64*b* and *c*. Generally, vermiculite and perlite plastic thicknesses of 1 to 1¾ in afford protection of 3 and 4 h, depending on construction details. Good alternatives include gypsum board (Fig. 7.64*d* and *e*) or gypsum block (Fig. 7.64*f*).

For buildings where rough usage is expected, a hard, dense insulating material such as concrete, brick, or tile would be the logical selection for fire protection.

For many buildings, finished ceilings are mandatory. It is therefore logical to employ the ceiling for protecting roof and floor framing. All types of gypsum plasters are used extensively for this dual purpose. Figure 7.65 illustrates typical installations. For 2-h floors, ordinary sand-gypsum plaster ¾ in thick is sufficient. Three- and four-hour floors may be obtained with perlite gypsum and vermiculite gypsum in the thickness range of ¾ to 1 in.

Instead of plastered ceilings, use may be made of fire-rated dry ceilings, acoustic tiles, or drop (lay-in) panels (Fig. 7.65*d* and *e*).

Another alternative is to spray the structural steel mechanically (where it is not protected with concrete) with plasters of gypsum, perlite, or vermiculite, proprietary cementitious mixtures, or mineral fibers not deemed a health hazard during spraying (Fig. 7.66). In such cases, the fire-resistance rating of the structural system is independent of the ceiling. Therefore, the ceiling need not be of fire-rated construction. Drop panels, if used, need not be secured to their suspended supports.

Still another sprayed-on material is the intumescent fire-retardant coating, essentially a paint. Tested in conformance with ASTM Specification E119, a 3⁄16-in-thick coat applied to a steel column has been rated 1 h, a ½-in-thick coating 2 h. As applied, the coating has a hard, durable finish, but at high temperatures, it puffs to many times its original thickness, thus forming an effective insulating blanket. Thus, it serves the dual need for excellent appearance and fire protection.

Aside from dual functioning of ceiling materials, the partitions, walls, etc., being of incombustible material, also protect the structural steel, often with no additional assistance. Fireproofing costs, therefore, may be made a relatively minor expense in the overall costs of a building through dual use of materials.

7.52 PIERCED CEILINGS AND FLOORS

Some buildings require recessed light fixtures and air-conditioning ducts, thus interrupting the continuity of fire-resistive ceilings. A rule that evolved from early standard fire tests permitted 100 in^2 of openings for noncombustible pipes, ducts, and electrical fixtures in each 100 ft^2 of ceiling area.

It has since been demonstrated, with over 100 fire tests that included electrical fixtures and ducts, that the fire-resistance integrity of ceilings is not impaired when, in general:

Recessed light fixtures, 2 by 4 ft, set in protective boxes, occupy no more than 25% of the gross ceiling area.

Air-duct openings, 30 in maximum in any direction, are spaced so as not to occupy more than 576 in^2 of each 100 ft^2 of gross ceiling area. They must be protected with fusible-link dampers against spread of smoke and heat.

These conclusions are not always applicable. Reports of fire tests of specific floor systems should be consulted.

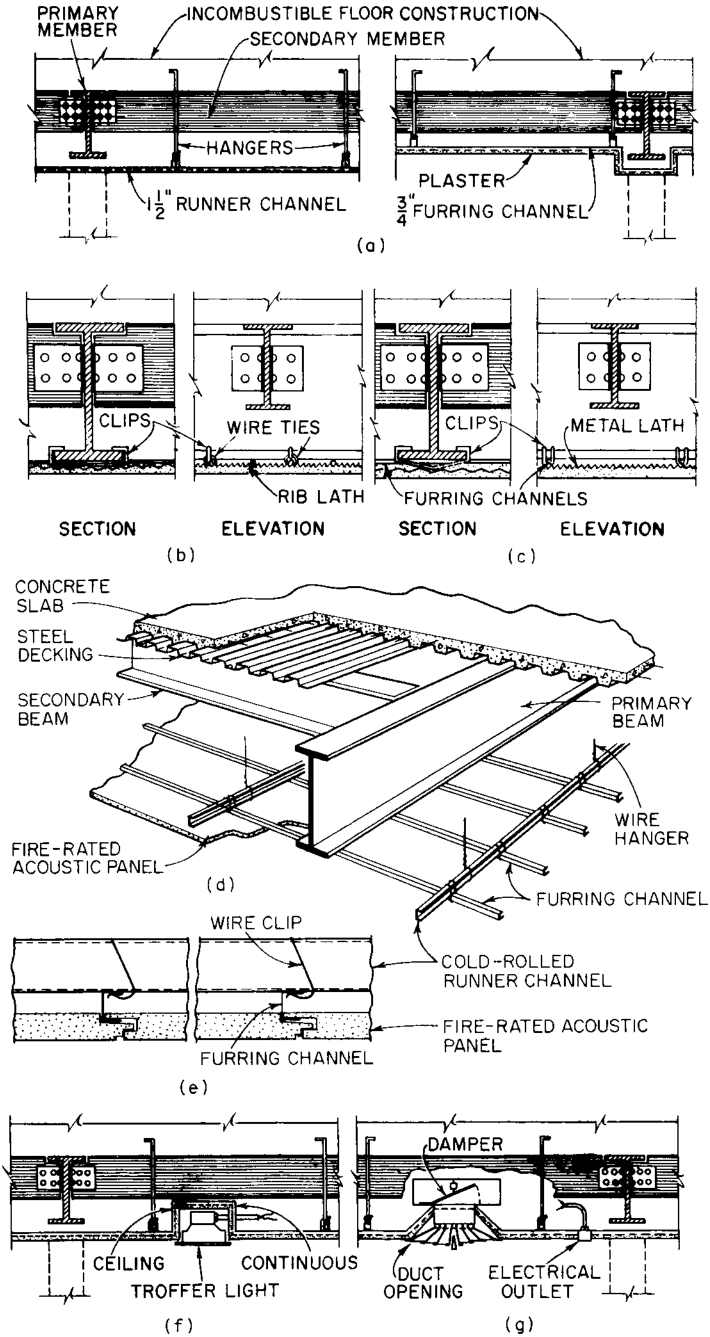


FIGURE 7.65 Fire protection of floor framing with incombustible floor construction: (*a*) section showing suspended plaster ceiling; (*b*) attachd plaster ceiling; (*c*) furred plaster ceiling; (*d*) suspended ceiling with lay-in, fire-rated acoustic panels; (*e*) detail of panel support in (*d*); (*f*) detail showing fire protection around recessed lighting; (*g*) detail showing fire protection around air-conditioning duct

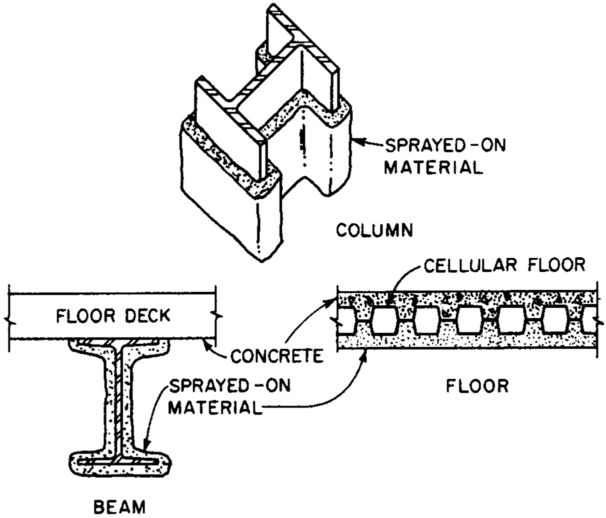


FIGURE 7.66 Typical fire protection with sprayed material.

A serious infringement of the fire rating of a floor system could occur when pipes, conduit, or other items pierce the floor slab, a practice called "poke-through." Failure to calk the openings with insulating material results in a lowering of fire ratings from hours to a few minutes.

7.53 FIRE-RESISTANCE RATINGS

Most standard fire tests on structural-steel members and assemblies have been conducted at one of two places—the National Institute of Standards and Technology, Washington, D.C., or the Underwriters Laboratories, Northbrook, Ill. Fire-testing laboratories also are available at Ohio State University, Columbus, Ohio, and the University of California, Berkeley, Calif. Laboratory test reports form the basis for establishing ratings. Summaries of these tests, together with tabulation of recognized ratings, are published by a number of organizations listed below. The trade associations, for the most part, limit their ratings to those constructions employing the material they represent.

The American Insurance Association (formerly The National Board of Fire Underwriters), 1130 Connecticut Ave., N.W., Suite 100, Washington, DC 20036

The National Institute of Standards and Technology, 100 Bureau Drive, Administration Bldg. #101, Mailstop 4701, Gaithersburg, MD 20899

Gypsum Association, 810 First St., N.E., #510, Washington, DC 20002

Metal Lath/Steel Framing Association, 600 Federal St., Chicago, IL 60605

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

American Iron and Steel Institute, 1000 16th St., N.W., Washington, DC 20036

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

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