SECTION 5
CONNECTIONS

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In this section, the term *connections* is used in a general sense to include all types of joints in structural steel made with fasteners or welds. Emphasis, however, is placed on the more commonly used connections, such as beam-column connections, main-member splices, and truss connections.

Recommendations apply to buildings and to both highway and railway bridges unless otherwise noted. This material is based on the specifications of the American Institute of Steel Construction (AISC), ‘‘Load and Resistance Factor Design Specification for Structural Steel Buildings,’’ 1999, and ‘‘Specification for Structural Steel Buildings—Allowable Stress Design and Plastic Design,’’ 1989; the American Association of State Highway and Transportation Officials (AASHTO), ‘‘Standard Specifications for Highway Bridges,’’ 1996; and the American Railway Engineering and Maintenance-of-Way Association (AREMA), ‘‘Manual,’’ 1998.

5.1 LIMITATIONS ON USE OF FASTENERS AND WELDS

Structural steel fabricators prefer that job specifications state that ‘‘shop connections shall be made with bolts or welds’’ rather than restricting the type of connection that can be used. This allows the fabricator to make the best use of available equipment and to offer a more competitive price. For bridges, however, standard specifications restrict fastener choice.

High-strength bolts may be used in either slip-critical or bearing-type connections (Art. 5.3), subject to various limitations. Bearing-type connections have higher allowable loads and should be used where permitted. Also, bearing-type connections may be either fully tensioned or snug-tight, subject to various limitations. Snug-tight bolts are much more economical to install and should be used where permitted.

Bolted slip-critical connections must be used for bridges where stress reversal may occur or slippage is undesirable. In bridges, connections subject to computed tension or combined shear and computed tension must be slip-critical. Bridge construction requires that bearing-type connections with high-strength bolts be limited to members in compression and secondary members.

Carbon-steel bolts should not be used in connections subject to fatigue.

In building construction, snug-tight bearing-type connections can be used for most cases, including connections subject to stress reversal due to wind or low seismic loading. The American Institute of Steel Construction (AISC) requires that fully tensioned high-strength bolts or welds be used for connections indicated in Sec. 6.14.2.

The AISC imposes special requirements on use of welded splices and similar connections in heavy sections. This includes ASTM A6 group 4 and 5 shapes and splices in built-up members with plates over 2 in thick subject to tensile stresses due to tension or flexure. Charpy V-notch tests are required, as well as special fabrication and inspection procedures. Where feasible, bolted connections are preferred to welded connections for such sections (see Art. 1.17).

In highway bridges, fasteners or welds may be used in field connections wherever they would be permitted in shop connections. In railroad bridges, the American Railway Engineering Association (AREA) recommended practice requires that field connections be made with high-strength bolts. Welding may be used only for minor connections that are not stressed by live loads and for joining deck plates or other components that are not part of the load-carrying structure.

5.2 BOLTS IN COMBINATION WITH WELDS

In new work, ASTM A307 bolts or high-strength bolts used in bearing-type connections should not be considered as sharing the stress in combination with welds. Welds, if used, should be provided to carry the entire stress in the connection. High-strength bolts proportioned for slip-critical connections may be considered as sharing the stress with welds.

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

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

AREMA does not permit the use of plug or slot welds but will accept fillet welds in holes and slots.

FASTENERS

In steel erection, fasteners commonly used include bolts, welded studs, and pins. Properties of these are discussed in the following articles.

5.3 HIGH-STRENGTH BOLTS, NUTS, AND WASHERS

For general purposes, A325 and A490 high-strength bolts may be specified. Each type of bolt can be identified by the ASTM designation and the manufacturer's mark on the bolt head and nut (Fig. 5.1). The cost of A490 bolts is 15 to 20% greater than that of A325 bolts.

Job specifications often require that ''main connections shall be made with bolts conforming to the Specification for Structural Joints Using ASTM A325 and A490 Bolts.'' This

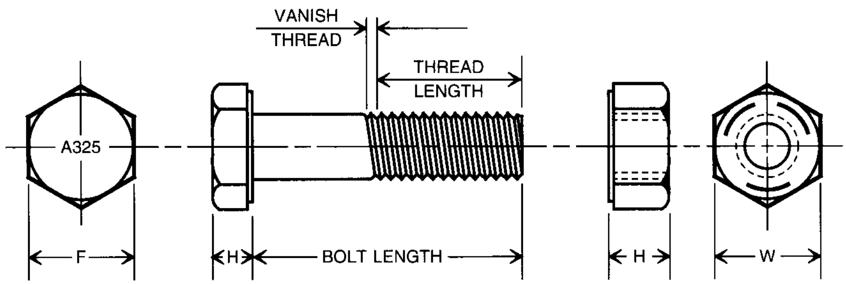


FIGURE 5.1 A325 high-strength structural steel bolt with heavy hex nut; heads are also marked to identify the manufacturer or distributor. Type 1 A325 bolts may additionally be marked with three radial lines 120° apart. Type 3 (weathering steel) bolts are marked as A325 and may also have other distinguishing marks to indicate a weathering grade.

specification, approved by the Research Council on Structural Connections (RCSC) of the Engineering Foundation, establishes bolt, nut, and washer dimensions, minimum fastener tension, and requirements for design and installation.

As indicated in Table 5.1, many sizes of high-strength bolts are available. Most standard connection tables, however, apply primarily to ¾-and ⅞-in bolts. Shop and erection equipment is generally set up for these sizes, and workers are familiar with them.

Bearing versus Slip-Critical Joints. Connections made with high-strength bolts may be slip-critical (material joined being clamped together by the tension induced in the bolts by tightening them) or bearing-type (material joined being restricted from moving primarily by the bolt shank). In bearing-type connections, bolt threads may be included in or excluded from the shear plane. Different stresses are allowed for each condition. The slip-critical connection is the most expensive, because it requires that the faying surfaces be free of paint (some exceptions are permitted), grease, and oil. Hence this type of connection should be used only where required by the governing design specification, e.g., where it is undesirable to have the bolts slip into bearing or where stress reversal could cause slippage (Art. 5.1). Slip-critical connections, however, have the advantage in building construction that when used in combination with welds, the fasteners and welds may be considered to share the stress (Art. 5.2). Another advantage that sometimes may be useful is that the strength of slip-critical connections is not affected by bearing limitations, as are other types of fasteners.

TABLE 5.1 Thread Lengths for High-Strength Bolts

Bolt diamter, in	Nominal thread, in	Vanish thread, in	Total thread, in
½	1.00	0.19	1.19
⅝	1.25	0.22	1.47
¾	1.38	0.25	1.63
⅞	1.50	0.28	1.78
1	1.75	0.31	2.06
1⅛	2.00	0.34	2.34
1¼	2.00	0.38	2.38
1⅜	2.25	0.44	2.69
1½	2.25	0.44	2.69

Threads in Shear Planes. The bearing-type connection with threads in shear planes is frequently used. Since location of threads is not restricted, bolts can be inserted from either side of a connection. Either the head or the nut can be the element turned. Paint is permitted on the faying surfaces.

Threads Excluded from Shear Planes. The bearing-type connection with threads excluded from shear planes is the most economical high-strength bolted connection, because fewer bolts generally are needed for a given capacity. But this type should be used only after careful consideration of the difficulties involved in excluding the threads from the shear planes. The location of the thread runout depends on which side of the connection the bolt is entered and whether a washer is placed under the head or the nut. This location is difficult to control in the shop but even more so in the field. The difficulty is increased by the fact that much of the published information on bolt characteristics does not agree with the basic specification used by bolt manufacturers (American National Standards Institute B18.2.1).

Thread Length and Bolt Length. Total nominal thread lengths and vanish thread lengths for high-strength bolts are given in Table 5.1. It is common practice to allow the last ⅛ in of vanish thread to extend across a single shear plane. In order to determine the required bolt length, the value shown in Table 5.2 should be added to the grip (i.e., the total thickness of all connected material, exclusive of washers). For each hardened flat washer that is used, add 5⁄32 in, and for each beveled washer, add 5⁄16 in. The tabulated values provide appropriate allowances for manufacturing tolerances and also provide for full thread engagement with an installed heavy hex nut. The length determined by the use of Table 5.2 should be adjusted to the next longer ¼-in length.

Washer Requirements. The RCSC specification requires that design details provide for washers in connections with high-strength bolts as follows:

1. A hardened beveled washer should be used to compensate for the lack of parallelism where the outer face of the bolted parts has a greater slope than 1:20 with respect to a plane normal to the bolt axis.

2. For A325 and A490 bolts for slip-critical connections and connections subject to direct tension, hardened washers are required as specified in items 3 through 7 below. For bolts permitted to be tightened only snug-tight, if a slotted hole occurs in an outer ply, a flat hardened washer or common plate washer shall be installed over the slot. For other connections with A325 and A490 bolts, hardened washers are not generally required.

TABLE 5.2 Lengths to be Added to Grip

Nominal bolt size, in	Addition to grip for determination of bolt length, in
½	11⁄16
⅝	⅞
¾	1
⅞	1⅛
1	1¼
1⅛	1½
1¼	1⅝
1⅜	1¾
1½	1⅞

3. When the calibrated wrench method is used for tightening the bolts, hardened washers shall be used under the element turned by the wrench.
4. For A490 bolts tensioned to the specified tension, hardened washers shall be used under the head and nut in steel with a specified yield point less than 40 ksi.
5. A hardened washer conforming to ASTM F436 shall be used for A325 or A490 bolts 1 in or less in diameter tightened in an oversized or short slotted hole in an outer ply.
6. Hardened washers conforming to F436 but at least $\frac{5}{16}$ in thick shall be used, instead of washers of standard thickness, under both the head and nut of A490 bolts more than 1 in in diameter tightened in oversized or short slotted holes in an outer ply. This requirement is not met by multiple washers even though the combined thickness equals or exceeds $\frac{5}{16}$ in.
7. A plate washer or continuous bar of structural-grade steel, but not necessarily hardened, at least $\frac{5}{16}$ in thick and with standard holes, shall be used for an A325 or A490 bolt 1 in or less in diameter when it is tightened in a long slotted hole in an outer ply. The washer or bar shall be large enough to cover the slot completely after installation of the tightened bolt. For an A490 bolt more than 1 in in diameter in a long slotted hole in an outer ply, a single hardened washer (not multiple washers) conforming to F436, but at least $\frac{5}{16}$ in thick, shall be used instead of a washer or bar of structural-grade steel.

The requirements for washers specified in items 4 and 5 above are satisfied by other types of fasteners meeting the requirements of A325 or A490 and with a geometry that provides a bearing circle on the head or nut with a diameter at least equal to that of hardened F436 washers. Such fasteners include ''twist-off'' bolts with a splined end that extends beyond the threaded portion of the bolt. During installation, this end is gripped by a special wrench chuck and is sheared off when the specified bolt tension is achieved.

The RCSC specification permits direct tension-indicating devices, such as washers incorporating small, formed arches designed to deform in a controlled manner when subjected to the tightening force. The specification also provides guidance on use of such devices to assure proper installation (Art. 5.14).

5.4 CARBON-STEEL OR UNFINISHED (MACHINE) BOLTS

''Secondary connections may be made with unfinished bolts conforming to the Specifications for Low-carbon Steel ASTM A307'' is an often-used specification. (Unfinished bolts also may be referred to as **machine, common,** or **ordinary bolts.**) When this specification is used, secondary connections should be carefully defined to preclude selection by ironworkers of the wrong type of bolt for a connection (see also Art. 5.1). A307 bolts have identification marks on their square, hexagonal, or countersunk heads (Fig. 5.2), as do high-strength bolts.

Use of high-strength bolts where A307 bolts provide the required strength merely adds to the cost of a structure. High-strength bolts cost at least 10% more than machine bolts.

A disadvantage of A307 bolts is the possibility that the nuts may loosen. This may be eliminated by use of lock washers. Alternatively, locknuts can be used or threads can be jammed, but either is more expensive than lock washers.

5.5 WELDED STUDS

Fasteners with one end welded to a steel member frequently are used for connecting material. Shear connectors in composite construction are a common application. Welded studs also

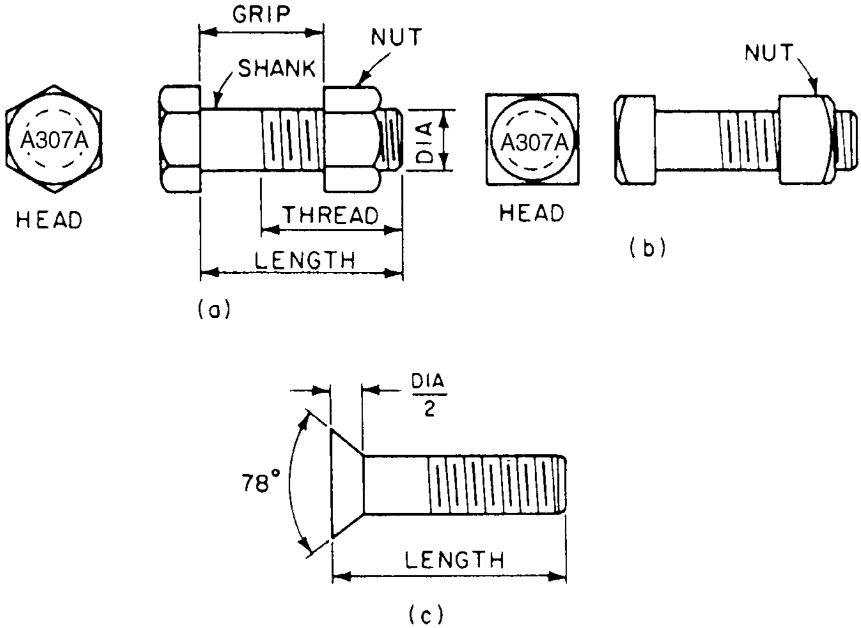


FIGURE 5.2 A307 Grade A carbon-steel bolts; heads are also marked to identify the manufacturer or distributor. (*a*) With hexagonal nut and bolt. (*b*) With square head and nut. (*c*) With countersunk head.

are used as anchors to attach wood, masonry, or concrete to steel. Types of studs and welding guns vary with manufacturers.

Table 5.3 lists approximate allowable loads for Allowable Stress Design for several sizes of threaded studs. Check manufacturer's data for studs to be used. Chemical composition and physical properties may differ from those assumed for this table.

Use of threaded studs for steel-to-steel connections can cut costs. For example, fastening rail clips to crane girders with studs eliminates drilling of the top flange of the girders and may permit a reduction in flange size. In designs with threaded studs, clearance must be provided for stud welds. Usual sizes of these welds are indicated in Fig. 5.3 and Table 5.4. The dimension C given is the minimum required to prevent burn-through in stud welding. Other design considerations may require greater thicknesses.

TABLE 5.3 Allowable Loads (kips) on Threaded Welded Studs

(*ASTM A108, grade 1015, 1018, or 1020*)

Stud size, in	Tension	Single shear
⅝	6.9	4.1
¾	10.0	6.0
⅞	13.9	8.3
1	18.2	10.9

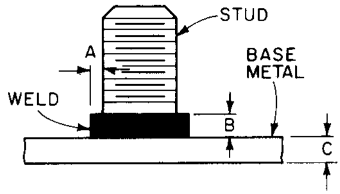


FIGURE 5.3 Welded stud.

5.6 PINS

A pinned connection is used to permit rotation of the end of a connected member. Some aspects of the design of a pinned connection are the same as those of a bolted bearing connection. The pin serves the same purpose as the shank of a bolt. But since only one pin is present in a connection, forces acting on a pin are generally much greater than those on a bolt. Shear on a pin can be resisted by selecting a large enough pin diameter and an appropriate grade of steel. Bearing on thin webs or plates can be brought within allowable values by addition of reinforcing plates. Because a pin is relatively long, bending, ignored in bolts, must be investigated in choosing a pin diameter. Arrangements of plates on the pin affect bending stresses. Hence plates should be symmetrically placed and positioned to minimize stresses.

Finishing of the pin and its effect on bearing should be considered. Unless the pin is machined, the roundness tolerance may not permit full bearing, and a close fit of the pin may not be possible. The requirements of the pin should be taken into account before a fit is specified.

Pins may be made of any of the structural steels permitted by AISC, AASHTO, and AREA specifications, ASTM A108 grades 1016 through 1030, and A668 classes C, D, F, and G.

Pins must be forged and annealed when they are more than 7 in in diameter for railroad bridges. Smaller pins may be forged and annealed or cold-finished carbon-steel shafting. In pins larger than 9 in in diameter, a hole at least 2 in in diameter must be bored full length along the axis. This work should be done after the forging has been allowed to cool to a temperature below the critical range, with precautions taken to prevent injury by too rapid cooling, and before the metal is annealed. The hole permits passage of a bolt with threaded ends for attachment of nuts or caps at the pin ends.

When reinforcing plates are needed on connected material, the plates should be arranged to reduce eccentricity on the pin to a minimum. One plate on each side should be as wide as the outstanding flanges will permit. At least one full-width plate on each segment should

TABLE 5.4 Minimum Weld and Base-Metal Dimensions (in) for Threaded Welded Studs

Stud size, in	*A*	*B* and *C*
5⁄8	1⁄8	1⁄4
3⁄4	3⁄16	5⁄16
7⁄8	3⁄16	3⁄8
1	1⁄4	7⁄16

extend to the far end of the stay plate. Other reinforcing plates should extend at least 6 in beyond the near edge. All plates should be connected with fasteners or welds arranged to transmit the bearing pressure uniformly over the full section.

In buildings, pinhole diameters should not exceed pin diameters by more than $\frac{1}{32}$ in. In bridges, this requirement holds for pins more than 5 in in diameter, but for smaller pins, the tolerance is reduced to $\frac{1}{50}$ in.

Length of pin should be sufficient to secure full bearing on the turned body of the pin of all connected parts. Pins should be secured in position and connected material restrained against lateral movement on the pins. For the purpose, ends of a pin may be threaded, and hexagonal recessed nuts or hexagonal solid nuts with washers may be screwed on them (Fig. 5.4*a*). Usually made of malleable castings or steel, the nuts should be secured by cotter pins in the screw ends or by burred threads. Bored pins may be held by a recessed cap at each end, secured by a nut on a bolt passing through the caps and the pin (Fig. 5.4*b*). In building work, a pin may be secured with cotter pins (Fig. 5.4*c* and *d*).

The most economical method is to drill a hole in each end for cotter pins. This, however, can be used only for horizontal pins. When a round must be turned down to obtain the required fit, a head can be formed to hold the pin at one end. The other end can be held by a cotter pin or threaded for a nut.

Example. Determine the diameter of pin required to carry a 320-kip reaction of a deck-truss highway bridge (Fig. 5.5) using Allowable Stress Design (ASD).

Bearing. For A36 steel, American Association of State Highway and Transportation Officials (AASHTO) specifications permit a bearing stress of 14 ksi on pins subject to rotation, such as those used in rockers and hinges. Hence the minimum bearing area on the pin must equal

$$A = 320/14 = 22.8 \text{ in}^2$$

Assume a 6-in-diameter pin. The bearing areas provided (Fig. 5.5) are

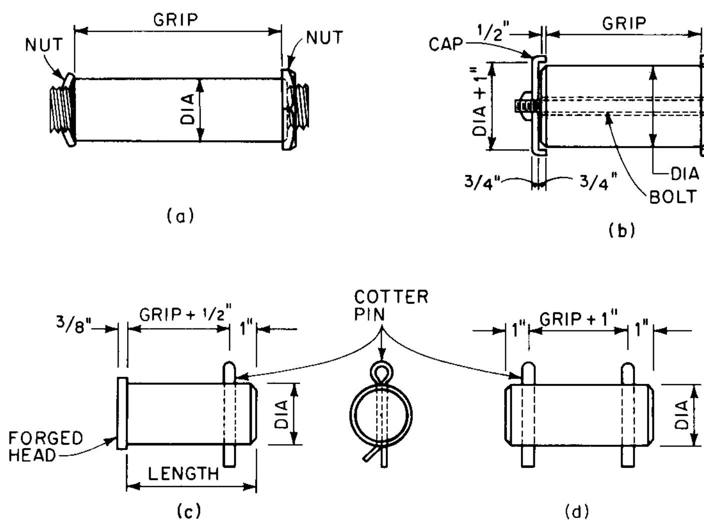


FIGURE 5.4 Pins. (*a*) With recessed nuts. (*b*) With caps and through bolt. (*c*) With forged head and cotter pin. (*d*) With cotter at each end (used in horizontal position).

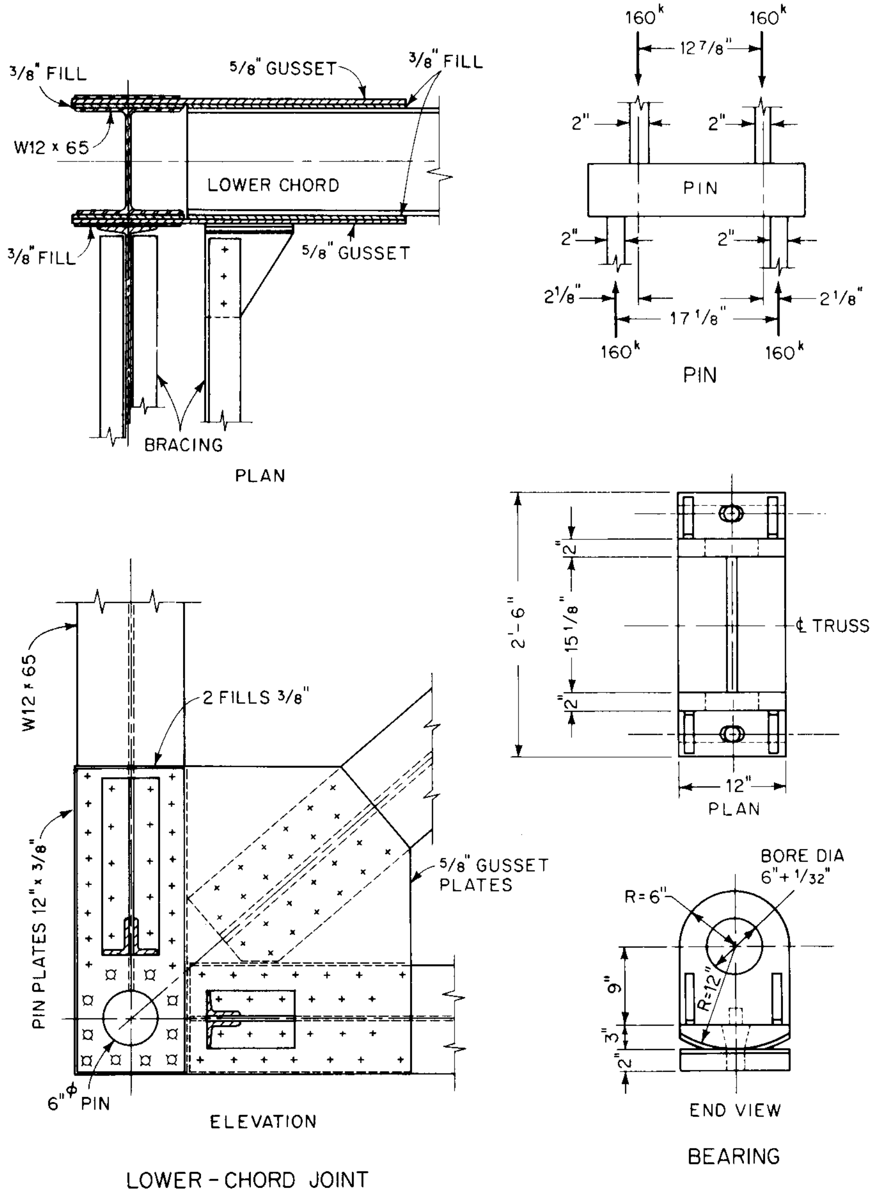


FIGURE 5.5 Pinned bearing for deck-truss highway bridge.

Flanges of W12 × 65	2 × 6 × 0.605	= 7.26
Fill plates	2 × 6 × ⅜	= 4.50
Gusset plates	2 × 6 × ⅝	= 7.50
Pin plates	2 × 6 × ⅜	= <u>4.50</u>
		23.76 in² > 22.8
Bearing plates	2 × 6 × 2	= 24.00 in² > 22.8

The 6-in pin is adequate for bearing.

Shear. For A36 steel, AASHTO specifications permit a shear stress on pins of 14 ksi. As indicated in the loading diagram for the pin in Fig. 5.5, the reaction is applied to the pin at two points. Hence the shearing area equals $2 \times \pi(6)^2/4 = 56.6$. Thus the shearing stress is

$$f_v = \frac{320}{56.6} = 5.65 \text{ ksi} < 14$$

The 6-in pin is adequate for shear.

Bending. For A36 steel, consider an allowable bending stress of 20 ksi. From the loading diagram for the pin (Fig. 5.5), the maximum bending moment is $M = 160 \times 2\frac{1}{8} = 340$ in-kips. The section modulus of the pin is

$$S = \frac{\pi d^3}{32} = \frac{\pi(6)^3}{32} = 21.2 \text{ in}^3$$

Thus the maximum bending stress in the pin is

$$f_b = \frac{340}{21.2} = 16 \text{ ksi} < 20$$

The 6-in pin also is satisfactory in bending.

GENERAL CRITERIA FOR BOLTED CONNECTIONS

Standard specifications for structural steel for buildings and bridges contain general criteria governing the design of bolted connections. They cover such essentials as permissible fastener size, sizes of holes, arrangements of fasteners, size and attachment of fillers, and installation methods.

5.7 FASTENER DIAMETERS

Minimum bolt diameters are ½ in for buildings and railroad bridges. In highway-bridge members carrying calculated stress, ¾-in fasteners are the smallest permitted, in general, but ⅝-in fasteners may be used in 2½-in stressed legs of angles and in flanges of sections requiring ⅝-in fasteners (controlled by required installation clearance to web and minimum edge distance). Structural shapes that do not permit use of ⅝-in fasteners may be used only in handrails.

In general, a connection with a few large-diameter fasteners costs less than one of the same capacity with many small-diameter fasteners. The fewer the fasteners, the fewer the number of holes to be formed and the less installation work. Larger-diameter fasteners are particularly favorable in connections where shear governs, because the load capacity of a fastener in shear varies with the square of the fastener diameter. For practical reasons, however, ¾-and ⅞-in-diameter fasteners are preferred.

Maximum Fastener Diameters in Angles. In bridges, the diameter of fasteners in angles carrying calculated stress may not exceed one-fourth the width of the leg in which they are placed. In angles where the size is not determined by calculated stress, 1-in fasteners may be used in 3½-in legs, ⅞-in fasteners in 3-in legs, and ¾-in fasteners in 2½-in legs. In addition, in highway bridges, ⅝-in fasteners may be used in 2-in legs.

5.8 FASTENER HOLES

Standard specifications require that holes for bolts be 1⁄16 in larger than the nominal fastener diameter. In computing net area of a tension member, the diameter of the hole should be taken 1⁄16 in larger than the hole diameter.

Standard specifications also require that the holes be punched or drilled. Punching usually is the most economical method. To prevent excessive damage to material around the hole, however, the specifications limit the maximum thickness of material in which holes may be punched full size. These limits are summarized in Table 5.5.

In buildings, holes for thicker material may be either drilled from the solid or subpunched and reamed. The die for all subpunched holes and the drill for all subdrilled holes should be at least 1⁄16 in smaller than the nominal fastener diameter.

In highway bridges, holes for material not within the limits given in Table 5.5 should be subdrilled or drilled full size. Holes in all field connections and field splices of main members of trusses, arches, continuous beams, bents, towers, plate girders, and rigid frames should be subpunched, or subdrilled when required by thickness limitations, and subsequently reamed while assembled or drilled full size through a steel template. Holes for floorbeam and stringer field end connections should be similarly formed. The die for subpunched holes and the drill for subdrilled holes should be 3⁄16 in smaller than the nominal fastener diameter.

A contractor has the option of forming, with parts for a connection assembled, subpunched holes and reaming or drilling full-size holes. The contractor also has the option of drilling or punching holes full size in unassembled pieces or connections with suitable numerically controlled drilling or punching equipment. In this case, the contractor may be required to demonstrate, by means of check assemblies, the accuracy of this drilling or punching pro-

TABLE 5.5 Maximum Material Thickness (in) for Punching Fastener Holes*

	AISC	AASHTO	AREMA
A36 steel	d + ⅛†	¾§	⅞
High-strength steels	d + ⅛†	⅝§	¾
Quenched and tempered steels	½‡	½§	

*Unless subpunching or subdrilling and reaming are used.
†d = fastener diameter, in.
‡A514 steel.
§But not more than five thicknesses of metal.

cedure. Holes drilled or punched by numerically controlled equipment are formed to size through individual pieces, but they may instead be formed by drilling through any combination of pieces held tightly together.

In railway bridges, holes for shop and field bolts may be punched full size, within the limits of Table 5.5, in members that will not be stressed by vertical live loads. This provision applies to, but is not limited to, the following: stitch bolts, bracing (lateral, longitudinal, or sway bracing) and connecting material, lacing stay plates, diaphragms that do not transfer shear or other forces, inactive fillers, and stiffeners not at bearing points.

Shop-bolt holes to be reamed may be subpunched. Methods permitted for shop-bolt holes in rolled beams and plate girders, including stiffeners and active fillers at bearing points, depend on material thickness and, in some cases, on strength. In materials not thicker than the nominal bolt diameter less ⅛ in, the holes should be subpunched ⅛ in less in diameter than the finished holes and then reamed to size with parts assembled. In A36 material thicker than ⅞ in (¾ in for high-strength steels), the holes should be subdrilled ¼ in less in diameter than the finished holes and then reamed to size with parts assembled.

A special provision applies to the case where matching shop-bolt holes in two or more plies are required to be reamed with parts assembled. If the assembly consists of more than five plies with more than three plies of main material, the matching holes in the other plies also should be reamed with parts assembled. Holes in those plies should be subpunched ⅛ in less in diameter than the finished hole.

Other shop-bolt holes should be subpunched ¼ in less in diameter than the finished hole and then reamed to size with parts assembled.

Field splices in plate girders and in truss chords should be reamed or drilled full size with members assembled. Truss-chord assemblies should consist of at least three abutting sections. Milled ends of the compression chords should have full bearing.

Field-bolt holes may be subpunched or subdrilled ¼ in less in diameter than finished holes in individual pieces. The subsized holes should then be reamed to size through steel templates with hardened steel bushings. In A36 steel thicker than ⅞ in (¾ in for high-strength steels), field-bolt holes may be subdrilled ¼ in less in diameter than the finished holes and then reamed to size with parts assembled or drilled full size with parts assembled. Field-bolt holes for sway bracing should conform to the requirements for shop-bolt holes.

If numerically controlled equipment is used to punch or drill holes, requirements are similar to those for highway bridges.

5.9 MINIMUM NUMBER OF FASTENERS

In buildings, connections carrying calculated stresses, except lacing, sag bars, and girts, should be designed to support at least 6 kips.

In highway bridges, connections, including angle bracing but not lacing bars and handrails, should contain at least two fasteners. Web shear splices should have at least two rows of fasteners on each side of the joint.

In railroad bridges, connections should have at least three fasteners per plane of connection.

Long Grips. In buildings, if A307 bolts in a connection carry calculated stress and have grips exceeding five diameters, the number of these fasteners used in the connection should be increased 1% for each additional ¹⁄₁₆ in in the grip.

5.10 CLEARANCES FOR FASTENERS

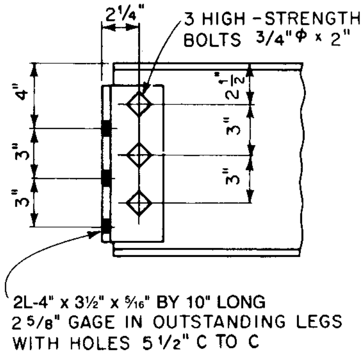


FIGURE 5.6 Staggered holes provide clearance for high-strength bolts.

Designs should provide ample clearance for tightening high-strength bolts. Detailers who prepare shop drawings for fabricators generally are aware of the necessity for this and can, with careful detailing, secure the necessary space. In tight situations, the solution may be staggering of holes (Fig. 5.6), variations from standard gages (Fig. 5.7), use of knife-type connections, or use of a combination of shop welds and field bolts.

Minimum clearances for tightening high-strength bolts are indicated in Fig. 5.8 and Table 5.6.

5.11 FASTENER SPACING

Pitch is the distance (in) along the line of principal stress between centers of adjacent fasteners. It may be measured along one or more lines of fasteners. For example, suppose bolts are staggered along two parallel lines. The pitch may be given as the distance between successive bolts in each line separately. Or it may be given as the distance, measured parallel to the fastener lines, between a bolt in one line and the nearest bolt in the other line.

Gage is the distance (in) between adjacent lines of fasteners along which pitch is measured or the distance (in) from the back of an angle or other shape to the first line of fasteners.

The minimum distance between centers of fasteners should be at least three times the fastener diameter. (The AISC specification, however, permits 2⅔ times the fastener diameter.)

Limitations also are set on maximum spacing of fasteners, for several reasons. In built-up members, **stitch fasteners,** with restricted spacings, are used between components to ensure uniform action. Also, in compression members, such fasteners are required to prevent local buckling. In bridges, **sealing fasteners** must be closely spaced to seal the edges of

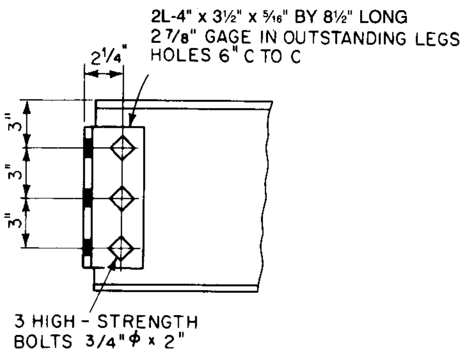


FIGURE 5.7 Increasing the gage in framing angles provides clearance for high-strength bolts.

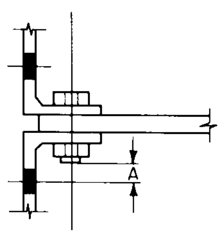


FIGURE 5.8 The usual minimum clearances *A* for high-strength bolts are given in Table 5.6.

TABLE 5.6 Clearances for High-Strength Bolts

Bolt dia, in	Nut height, in	Usual min clearance, in A	Min clearance for twist-off bolts, in A	
			Small tool	Large tool
$\frac{5}{8}$	$\frac{5}{8}$	1	$1\frac{5}{8}$	—
$\frac{3}{4}$	$\frac{3}{4}$	$1\frac{1}{4}$	$1\frac{5}{8}$	$1\frac{7}{8}$
$\frac{7}{8}$	$\frac{7}{8}$	$1\frac{3}{8}$	$1\frac{5}{8}$	$1\frac{7}{8}$
1	1	$1\frac{7}{16}$		$1\frac{7}{8}$
$1\frac{1}{8}$	$1\frac{1}{8}$	$1\frac{9}{16}$		—
$1\frac{1}{4}$	$1\frac{1}{4}$	$1\frac{11}{16}$		—

plates and shapes in contact to prevent penetration of moisture. Maximum spacing of fasteners is governed by the requirements for sealing or stitching, whichever requires the smaller spacing.

For sealing, AASHTO specifications require that the pitch of fasteners on a single line adjoining a free edge of an outside plate or shape should not exceed 7 in or $4 + 4t$ in, where t is the thickness (in) of the thinner outside plate or shape (Fig. 5.9*a*). (See also the maximum edge distance, Art. 5.12). If there is a second line of fasteners uniformly staggered with those in the line near the free edge, a smaller pitch for the two lines can be used if the gage g (in) for these lines is less than $1\frac{1}{2} + 4t$. In this case, the staggered pitch (in) should not exceed $4 + 4t - \frac{3}{4}g$ or 7 in but need not be less than half the requirement for a single line (Fig. 5.9*b*). See AASHTO specifications for requirements for stitch fasteners.

Bolted joints in unpainted weathering steel require special limitations on pitch: 14 times the thickness of the thinnest part, not to exceed 7 in (AISC specification).

5.12 EDGE DISTANCE OF FASTNERS

Minimum distances from centers of fasteners to any edges are given in Tables 5.7 and 5.8.

The AISC specifications for structural steel for buildings have the following provisions for minimum edge distance: The distance from the center of a standard hole to an edge of a connected part should not be less than the applicable value from Table 5.7 nor the value from the equation

$$L_e \leq 2P/F_u t \tag{5.1}$$

where L_e = the distance from the center of a standard hole to the edge of the connected part, in
P = force transmitted by one fastener to the critical connected part, kips
F_u = specified minimum tensile strength of the critical connected part, ksi
t = thickness of the critical connected part, in

Also, L_e should not be less than $1\frac{1}{2}d$ when $F_p = 1.\,2F_u$, where d is the diameter of the bolt (in) and F_p is the allowable bearing stress of the critical connected part (ksi).

The AASHTO specifications for highway bridges require the minimum distance from the center of any bolt in a standard hole to a sheared or flame-cut edge to be as shown in Table 5.8. When there is only a single transverse fastener in the direction of the line of force in a standard or short slotted hole, the distance from the center of the hole to the edge connected part (ASD specifications) should not be less than $1\frac{1}{2}d$ when

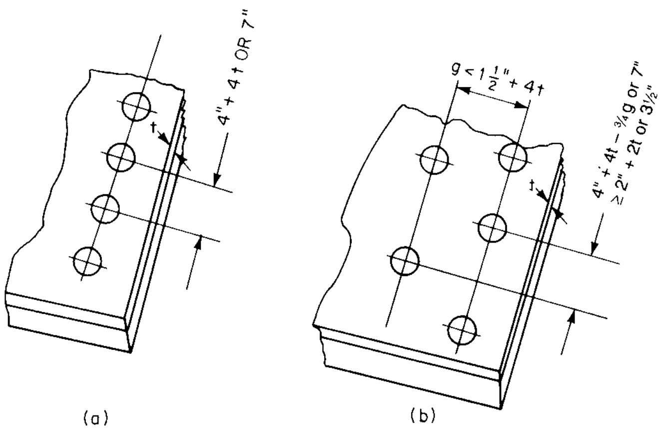


FIGURE 5.9 Maximum pitch of bolts for sealing. (*a*) Single line of bolts. (*b*) Double line of bolts.

$$F_p = \frac{0.5L_eF_u}{d} \tag{5.1a}$$

where F_u = specified minimum tensile strength of conection material, ksi
L_e = clear distance between holes or between hole and edge of material in direction of applied force, in
d = nominal bolt diameter, in

The AREMA *Manual* requirement for minimum edge distance for a sheared edge is given in Table 5.8. The distance between the center of the nearest bolt and the end of the connected part toward which the pressure of the bolt is directed should be not less than $2df_p/F_u$, where

TABLE 5.7 Minimum Edge Distances (in) for Fastener Holes in Steel for Buildings

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	1¾d‡	1¼d‡

* 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.

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

‡ d = fastener diameter in.

From AISC "Specification for Structural Steel Buildings."

TABLE 5.8 Minimum Edge Distances (in) for Fastener Holes in Steel for Bridges

Fastener diameter, in	At sheared or flame-cut edges		In flanges of beams or channels		At other rolled or planed edges	
	Highway	Railroad	Highway	Railroad	Highway	Railroad
½		⅞		⅝		¾
⅝	1⅛	1⅛	⅞	13⁄16	1	15⁄16
¾	1¼	1 5⁄16	1	15⁄16	1⅛	1⅛
⅞	1½	1½	1⅛	1⅛	1¼	1½
Over 1	1¾	1¾*d**	1¼	1¼*d**	1½	1½*d**

* d = fastener diameter, in.

d is the diameter of the bolt (in) and f_p is the computed bearing stress due to the service load (ksi).

Maximum edge distances are set for sealing and stitch purposes. AISC specifications limit the distance from center of fastener to nearest edge of parts in contact to 12 times the thickness of the connected part, with a maximum of 6 in. The AASHTO maximum is 5 in or 8 times the thickness of the thinnest outside plate. (AISC gives the same requirement for unpainted weathering steel.) The AREMA maximum is 6 in or 4 times the plate thickness plus 1.5 in.

5.13 FILLERS

A **filler** is a plate inserted in a splice between a gusset or splice plate and stress-carrying members to fill a gap between them. Requirements for fillers included in the AISC specifications for structural steel for buildings are as follows.

In welded construction, a filler ¼ in or more thick should extend beyond the edge of the splice plate and be welded to the part on which it is fitted (Fig. 5.10). The welds should be

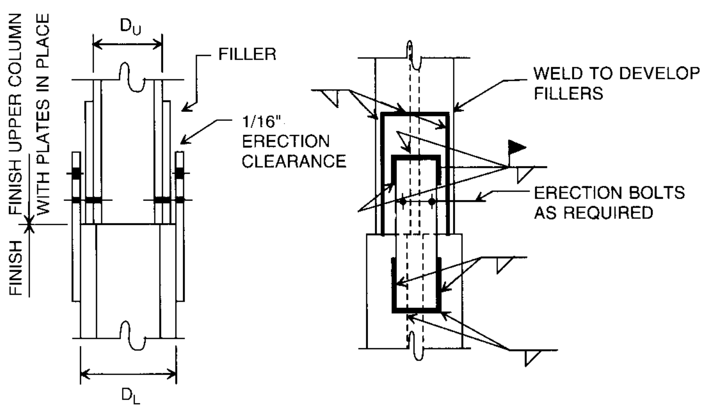


FIGURE 5.10 Typical welded splice of columns when depth D_u of the upper column is nominally 2 in less than depth D_L of the lower column.

able to transmit the splice-plate stress, applied at the surface of the filler, as an eccentric load. The welds that join the splice plate to the filler should be able to transmit the splice plate stress and should have sufficient length to prevent overstress of the filler along the toe of the welds. A filler less than ¼ in thick should have edges flush with the splice-plate edges. The size of the welds should equal the sum of the filler thickness and the weld size necessary to resist the splice plate stress.

In bearing connections with bolts carrying computed stress passing through fillers thicker than ¼ in, the fillers should extend beyond the splice plate (Fig. 5.11). The filler extension should be secured by sufficient bolts to distribute the load on the member uniformly over the combined cross section of member and filler. Alternatively, an equivalent number of bolts should be included in the connection. Fillers ¼ to ¾ in thick need not be extended if the allowable shear stress in the bolts is reduced by the factor $0.4(t - 0.25)$, where t is the total thickness of the fillers but not more than ¾ in.

The AASHTO specifications for highway bridges require the following: Fillers thicker than ¼ in, except in slip critical connections, through which stress-carrying fasteners pass, should preferably be extended beyond the gusset or splice material. The extension should be secured by enough additional fasteners to carry the stress in the filler. This stress should be calculated as the total load on the member divided by the combined cross-sectional area of the member and filler. Alternatively, additional fasteners may be passed through the gusset or splice material without extending the filler. If a filler is less than ¼ in thick, it should not be extended beyond the splice material. Additional fasteners are not required. Fillers ¼ in or more thick should not consist of more than two plates, unless the engineer gives permission.

The AREMA does not require additional bolts for development of fillers in high-strength bolted connections.

5.14 INSTALLATION OF FASTENERS

All parts of a connection should be held tightly together during installation of fasteners. Drifting done during assembling to align holes should not distort the metal or enlarge the holes. Holes that must be enlarged to admit fasteners should be reamed. Poor matching of holes is cause for rejection.

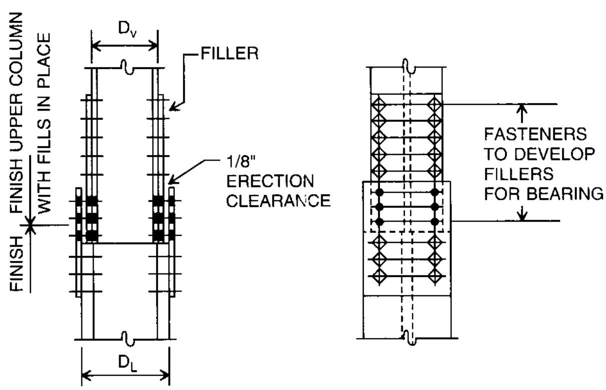


FIGURE 5.11 Typical bolted splice of columns when depth D_u of the upper column is nominally 2 in less than depth D_L of the lower column.

For connections with high-strength bolts, surfaces, when assembled, including those adjacent to bolt heads, nuts, and washers, should be free of scale, except tight mill scale. The surfaces also should be free of defects that would prevent solid seating of the parts, especially dirt, burrs, and other foreign material. Contact surfaces within slip-critical joints should be free of oil, paint, lacquer, and rust inhibitor.

Each high-strength bolt should be tightened so that when all fasteners in the connection are tight it will have the total tension (kips) given in Table 6.18, for its diameter. Tightening should be done by the turn-of-the-nut method or with properly calibrated wrenches.

High-strength bolts usually are tightened with an impact wrench. Only where clearance does not permit its use will bolts be hand-tightened.

Requirements for joint assembly and tightening of connections are given in the ''Specification for Structural Joints Using ASTM A325 or A490 Bolts,'' Research Council on Structural Connections of the Engineering Foundation. The provisions applicable to connections requiring full pretensioning include the following.

Calibrated-wrench Method. When a calibrated wrench is used, it must be set to cut off tightening when the required tension (Table 6.18) has been exceeded by 5%. The wrench should be tested periodically (at least daily on a minimum of three bolts of each diameter being used). For the purpose, a calibrating device that gives the bolt tension directly should be used. In particular, the wrench should be calibrated when bolt size or length of air hose is changed.

When bolts are tightened, bolts previously tensioned may become loose because of compression of the connected parts. The calibrated wrench should be reapplied to bolts previously tightened to ensure that all bolts are tensioned to the prescribed values.

Turn-of-the-nut Method. When the turn-of-the-nut method is used, tightening may be done by impact or hand wrench. This method involves three steps:

1. **Fit-up of connection.** Enough bolts are tightened a sufficient amount to bring contact surfaces together. This can be done with fit-up bolts, but it is more economical to use some of the final high-strength bolts.
2. **Snug tightening of bolts.** All high-strength bolts are inserted and made snug-tight (tightness obtained with a few impacts of an impact wrench or the full effort of a person using an ordinary spud wrench). While the definition of snug-tight is rather indefinite, the condition can be observed or learned with a tension-testing device.
3. **Nut rotation from snug-tight position.** All bolts are tightened by the amount of nut rotation specified in Table 5.9. If required by bolt-entering and wrench-operation clearances, tightening, including by the calibrated-wrench method, may be done by turning the bolt while the nut is prevented from rotating.

Direct-Tension-Indicator Tightening. Two types of direct-tension-indicator devices are available: washers and twist-off bolts. The hardened-steel load-indicator washer has dimples on the surface of one face of the washer. When the bolt is torqued, the dimples depress to the manufacturer's specification requirements, and proper torque can be measured by the use of a feeler gage. Special attention should be given to proper installation of flat hardened washers when load-indicating washers are used with bolts installed in oversize or slotted holes and when the load-indicating washers are used under the turned element.

The twist-off bolt is a bolt with an extension to the actual length of the bolt. This extension will twist off when torqued to the required tension by a special torque gun. A representative sample of at least three bolts and nuts for each diameter and grade of fastener should be tested in a calibration device to demonstrate that the device can be torqued to 5% greater tension than that required in Table 6.18.

When the direct tension indicator involves an irreversible mechanism such as yielding or fracture of an element, bolts should be installed in all holes and brought to the snug-tight

TABLE 5.9 Number of Nut or Bolt Turns from Snug-Tight Condition for High-Strength Bolts*

	Slope of outer faces of bolted parts		
Bolt length (Fig. 5.1)	Both faces normal to bolt axis	One face normal to bolt axis and the other sloped†	Both faces sloped†
Up to 4 diameters	⅓	½	⅔
Over 4 diameters but not more than 8 diameters	½	⅔	⅚
Over 8 diameters but not more than 12 diameters‡	⅔	⅚	1

*Nut rotation is relative to the bolt regardless of whether the nut or bolt is turned. For bolts installed by ½ turn and less, the tolerance should be ±30°. For bolts installed by ⅔ turn and more, the tolerance should be ±45°. This table is applicable only to connections in which all material within the grip of the bolt is steel.

†Slope is not more than 1:20 from the normal to the bolt axis, and a beveled washer is not used.

‡No research has been performed by RCSC to establish the turn-of-the-nut procedure for bolt lengths exceeding 12 diameters. Therefore, the required rotation should be determined by actual test in a suitable tension-measuring device that stimulates conditions of solidly fitted steel.

condition. All fasteners should then be tightened, progressing systematically from the most rigid part of the connection to the free edges in a manner that will minimize relaxation of previously tightened fasteners prior to final twist off or yielding of the control or indicator element of the individual devices. In some cases, proper tensioning of the bolts may require more than a single cycle of systematic tightening.

WELDS

Welded connections often are used because of simplicity of design, fewer parts, less material, and decrease in shop handling and fabrication operations. Frequently, a combination of shop welding and field bolting is advantageous. With connection angles shop welded to a beam, field connections can be made with high-strength bolts without the clearance problems that may arise in an all-bolted connection.

Welded connections have a rigidity that can be advantageous if properly accounted for in design. Welded trusses, for example, deflect less than bolted trusses, because the end of a welded member at a joint cannot rotate relative to the other members there. If the end of a beam is welded to a column, the rotation there is practically the same for column and beam.

A disadvantage of welding, however, is that shrinkage of large welds must be considered. It is particularly important in large structures where there will be an accumulative effect.

Properly made, a properly designed weld is stronger than the base metal. Improperly made, even a good-looking weld may be worthless. Properly made, a weld has the required penetration and is not brittle.

Prequalified joints, welding procedures, and procedures for qualifying welders are covered by AWS D1.1, ‘‘Structural Welding Code—Steel,’’ and AWS D1.5, ‘‘Bridge Welding Code,’’ American Welding Society. Common types of welds with structural steels intended for welding when made in accordance with AWS specifications can be specified by note or by symbol with assurance that a good connection will be obtained.

In making a welded design, designers should specify only the amount and size of weld actually required. Generally, a 5⁄16-in weld is considered the maximum size for a single pass.

The cost of fit-up for welding can range from about one-third to several times the cost of welding. In designing welded connections, therefore, designers should consider the work necessary for the fabricator and the erector in fitting members together so they can be welded.

5.15 WELDING MATERIALS

Weldable structural steels permissible in buildings and bridges are listed with required electrodes in Tables 5.10 and 5.11. Welding electrodes and fluxes should conform to AWS 5.1, 5.5, 5.17, 5.18, 5.20, 5.23, 5.25, 5.26, 5.28, or 5.29 or applicable provisions of AWS D1.1 or D1.5. Weld metal deposited by electroslag or electrogas welding processes should conform to the requirements of AWS D1.1 or D1.5 for these processes. For bridges, the impact requirements in D1.5 are mandatory. Welding processes are described in Art. 2.6.

For welded connections in buildings, the electrodes or fluxes given in Table 5.10 should be used in making complete-penetration groove welds. These welds can be designed with allowable stresses for base metal indicated in Table 6.23. (See Art. 6.14.)

TABLE 5.10 Matching Filler-Metal Requirements for Complete-Penetration Groove Welds in Building Construction

	Welding process			
Base metal*	Shielded metal-arc	Submerged-arc	Gas metal-arc	Flux cored arc
A36†, A53 grade B A500 grades A and B A501, A529, and A570 grades 30 through 50	AWS A5.1 or A5.5§ E60XX E70XX E70XX-X	AWS A5.17 or A5.23§ F6XX-EXXX F7XX-EXXX or F7XX-EXX-XX	 AWS A5.18 ER70S-X	AWS A5.20 or A5.29§ E6XT-X E7XT-X (Except −2, −3, −10, −13, −14, −GS) E7XTX-XX
A572 grade 42 and 50, and A588‡ (4 in. and under)	AWS A5.1 or A5.5§ E7015, E7016, E7018, E7028 E7015-X, E7016-X, E7018-X	AWS A5.17 or A5.23§ F7XX-EXXX F7XX-EXX-XX	AWS A5.18 ER70S-X	AWS A5.20 or A5.29§ E7XT-X (Except −2, −3, −10, −13, −14, −GS) E7XTX-X
A572 grades 60 and 65	AWS A5.5§ E8016-X, E8015-X E8018-X	AWS A5.23§ F8XX-EXX-XX	AWS A5.28§ ER 80S-X	AWS A5.29§ E8XTX-X

* In joints involving base metals of different groups, either of the following filler metals may be used: (1) that which matches the higher strength base metal; or (2) that which matches the lower strength base metal and produces a low-hydrogen deposit. Preheating must be in conformance with the requirements applicable to the higher strength group.

† Only low-hydrogen electrodes may be used for welding A36 steel more than 1 in thick for cyclically loaded structures.

‡ Special welding materials and procedures (e.g., E80XX-X low-alloy electrodes) may be required to match the notch toughness of base metal (for applications involving impact loading or low temperature) or for atmospheric corrosion and weathering characteristics.

§ Filler metals of alloy group B3, B3L, B4, B4L, B5, B5L, B6, B6L, B7, B7L, B8, B8L, or B9 in ANSI/AWS A5.5, A5.23, A5.28, or A5.29 are not prequalified for use in the as-welded condition.

For welded connections in bridges, the electrodes or fluxes given in Table 5.11 should be used in making complete-penetration groove welds. These welds can be designed with allowable stresses for base metal indicated in Table 11.6 or 11.29. (See Art. 11.8 or 11.37.)

Allowable fatigue stresses must be considered where stress fluctuations are present. (See Art. 6.22, 11.10, or 11.38.)

5.16 TYPES OF WELDS

The main types of welds used for structural steel are fillet, groove, plug, and slot. The most commonly used weld is the fillet. For light loads, it is the most economical, because little preparation of material is required. For heavy loads, groove welds are the most efficient, because the full strength of the base metal can be obtained easily. Use of plug and slot welds generally is limited to special conditions where fillet or groove welds are not practical.

More than one type of weld may be used in a connection. If so, the allowable capacity of the connection is the sum of the effective capacities of each type of weld used, separately computed with respect to the axis of the group.

Tack welds may be used for assembly or shipping. They are not assigned any stress-carrying capacity in the final structure. In some cases, these welds must be removed after final assembly or erection.

Fillet welds have the general shape of an isosceles right triangle (Fig. 5.12). The size of the weld is given by the length of leg. The strength is determined by the throat thickness, the shortest distance from the root (intersection of legs) to the face of the weld. If the two legs are unequal, the nominal size of the weld is given by the shorter of the legs. If welds are concave, the throat is diminished accordingly, and so is the strength.

Fillet welds are used to join two surfaces approximately at right angles to each other. The joints may be lap (Fig. 5.13) or tee or corner (Fig. 5.14). Fillet welds also may be used with groove welds to reinforce corner joints. In a skewed tee joint, the included angle of weld deposit may vary up to 30° from the perpendicular, and one corner of the edge to be connected may be raised, up to $^3\!/_{16}$ in. If the separation is greater than $^1\!/_{16}$ in, the weld leg should be increased by the amount of the root opening.

Groove welds are made in a groove between the edges of two parts to be joined. These welds generally are used to connect two plates lying in the same plane (butt joint), but they also may be used for tee and corner joints.

Standard types of groove welds are named in accordance with the shape given the edges to be welded: square, single vee, double vee, single bevel, double bevel, single U, double U, single J, and double J (Fig. 5.15). Edges may be shaped by flame cutting, arc-air gouging, or edge planing. Material up to $^5\!/_8$ in thick, however, may be groove-welded with square-cut edges, depending on the welding process used.

Groove welds should extend the full width of parts joined. Intermittent groove welds and butt joints not fully welded throughout the cross section are prohibited.

Groove welds also are classified as complete-penetration and partial-penetration welds.

In a **complete-penetration weld,** the weld material and the base metal are fused throughout the depth of the joint. This type of weld is made by welding from both sides of the joint or from one side to a backing bar or backing weld. When the joint is made by welding from both sides, the root of the first-pass weld is chipped or gouged to sound metal before the weld on the opposite side, or back pass is made. The throat dimension of a complete-penetration groove weld, for stress computations, is the full thickness of the thinner part joined, exclusive of weld reinforcement.

Partial-penetration welds generally are used when forces to be transferred are small. The edges may not be shaped over the full joint thickness, and the depth of weld may be less than the joint thickness (Fig. 5.15). But even if edges are fully shaped, groove welds made from one side without a backing strip or made from both sides without back gouging

TABLE 5.11 Matching Filler-Metal Requirements for Complete-Penetration Groove Welds in Bridge Construction

(*a*) *Qualified in Accordance with AWS D1.5 Paragraph 5.12*

	Welding process†		
Base metal*	Shielded metal-arc	Submerged-arc	Flux-cored arc with external shielding gas
A36/M270M grade 250	AWS A5.1 or A5.5 E7016, E7018, or E7028, E7016-X, E7018-X	AWS A5.17 F6A0-EXXX F7A0-EXXX	AWS A5.20 E6XT-1,5 E7XT-1,5
A572 grade 50/M270M grade 345 type 1, 2, or 3	AWS A5.1 or A5.5 E7016, E7018, E7028, E7016-X, or E7018-X	AWS A5.17 F7A0-EXXX	AWS A5.20 E7XT-1,5
A588/M270M grade 345W‡ 4-in and under	AWS A5.1 E7016, E7018, E7028 AWS A5.5 E7016-X, E7018-X, E7028-X, E7018-W E7015, 16, 18-C1L, C2L E8016, 18-C1, C2§ E8016, 18-C3§ E8018-W§	AWS A5.17 or A5.23 F7A0-EXXX, F8A0-EXXX§	AWS A5.20 or A5.29 E7XT-1,5 E8XT-1,5-NiX, W
A852/M270M grade 485W‡	AWS A5.5 E9018-M	AWS A5.23 F9A0-EXXX-X	AWS A5.29 E9XT1-X E9XT5-X
A514/M270 grades 690 and 690W Over 2½ in thick	AWS A5.5 E1018-M		

(*b*) *Qualified in accordance with AWS D1.5 Paragraph 5.13*

	Welding process†					
Base metal*	Flux-cored arc, self-shielding	Gas metal-arc	Electrogas (not authorized for tension and stress reversal members)		Submerged-arc	Shielded metal-arc
A36/M270M grade 250	AWS A5.20 E6XT-6,8 E7XT-6,8 AWS A5.29 E6XT8-8 E7XT8-X	AWS A5.18 ER70S-2,3,6,7	AWS A5.25 FES 60-XXXX FES 70-XXXX FES 72-XXXX	AWS A5.26 EG60XXXX EG62XXXX EG70XXXX EG72XXXX		
A572 grade 50/M270M grade 345	AWS A5.20 E7XT-6,8 AWS A5.29 E7XT8-X	AWS A5.18 ER 70S-2,3,6,7	AWS A5.25 FES 70-XXXX FES 72-XXXX	AWS A5.26 EG70XXXX EG72XXXX		

(*b*) *Qualified in accordance with AWS D1.5 Paragraph 5.13* (*continued*)

Base metal*	Welding process† Flux-cored arc, self-shielding	Gas metal-arc	Electrogas (not authorized for tension and stress reversal members)		Submerged-arc	Shielded metal-arc
A588/M270M grade 345W‡ 4 in and under	AWS A5.20 E7XT-6,8 AWS A5.29 E7XT8-NiX§	AWS A5.18 ER70S-2,3,6,7 AWS A5.28 ER80S-NiX	AWS A5.25 FES70-XXXX FES72-XXXX	AWS A5.25 EG70-XXXX EG72-XXXX		
A852/M270 grade 485W‡		As Approved by Engineer				
A514/M270M grades 690 and 690W‡ over 2½ in thick	With external shielding gas AWS A5.29 E100 T5-K3 E101 T1-K7	AWS A5.28 ER100S-1 ER100S-2			AWS A5.23 F10A4-EM2-M2	
A514/M270M grades 690 and 690W 2½ in thick or less	With external shielding gas AWS A5.29 E110T5-K3,K4 E111T1-K4	AWS A5.28 ER110S-1			AWS A5.23 F11A4-EM3-M3	AWS A5.5 E11018-M

* In joints involving base metals of two different yield strengths, filler metal applicable to the lower-strength base metal may be used.

† Electrode specifications with the same yield and tensile properties, but with lower impact-test temperature may be substituted (e.g., F7A2-EXXX may be substituted for F7A0-EXXX).

‡ Special welding materials and procedures may be required to match atmospheric, corrosion and weathering characteristics. See AWS D1.5.

§ The 550MPa filler metals are intended for exposed applications of weathering steels. They need not be used on applications of M270M grade 345W steel that will be painted.

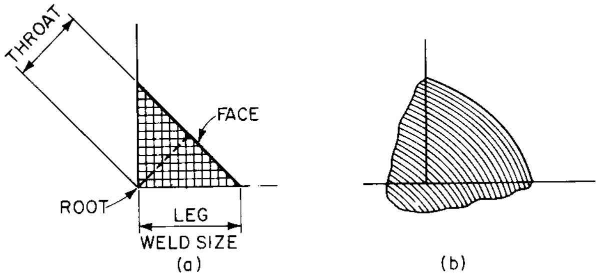


FIGURE 5.12 Fillet weld. (*a*) Theoretical cross section. (*b*) Actual cross section.

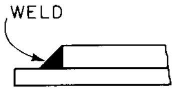


FIGURE 5.13 Welded lap joint.

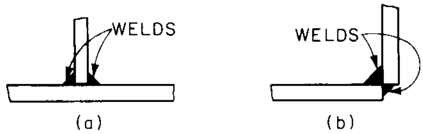


FIGURE 5.14 (*a*) Tee joint. (*b*) Corner joint.

are considered partial-penetration welds. They often are used for splices in building columns carrying axial loads only. In bridges, such welds should not be used where tension may be applied normal to the axis of the welds.

Plug and slot welds are used to transmit shear in lap joints and to prevent buckling of lapped parts. In buildings, they also may be used to join components of built-up members. (Plug or slot welds, however, are not permitted on A514 steel.) The welds are made, with lapped parts in contact, by depositing weld metal in circular or slotted holes in one part. The openings may be partly or completely filled, depending on their depth. Load capacity of a plug or slot completely welded equals the product of hole area and allowable stress. Unless appearance is a main consideration, a fillet weld in holes or slots is preferable.

Economy in Selection. In selecting a weld, designers should consider not only the type of joint but also the type of weld that would require a minimum amount of metal. This would yield a saving in both material and time.

While strength of a fillet weld varies with size, volume of metal varies with the square of the size. For example, a ½-in fillet weld contains four times as much metal per inch of

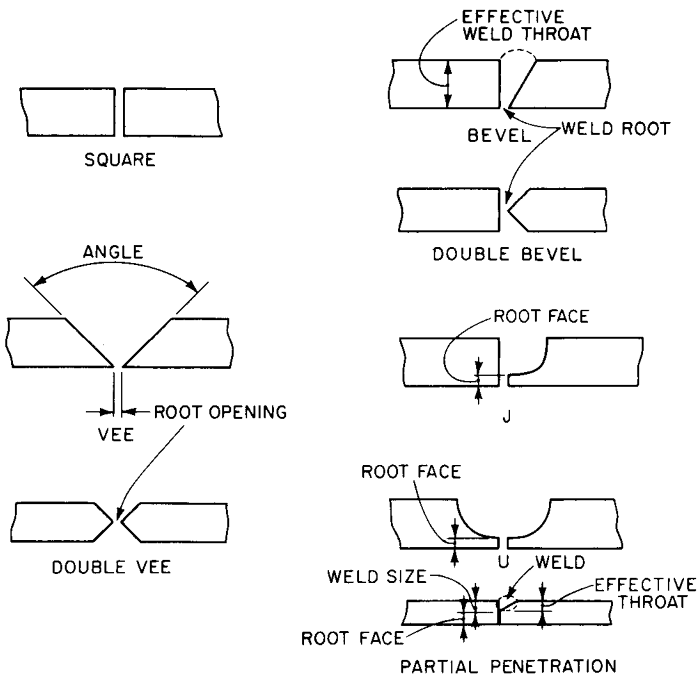


FIGURE 5.15 Groove welds.

length as a ¼-in weld but is only twice as strong. In general, a smaller but longer fillet weld costs less than a larger but shorter weld of the same capacity.

Furthermore, small welds can be deposited in a single pass. Large welds require multiple passes. They take longer, absorb more weld metal, and cost more. As a guide in selecting welds, Table 5.12 lists the number of passes required for some frequently used types of welds.

Double-V and double-bevel groove welds contain about half as much weld metal as single-V and single-bevel groove welds, respectively (deducting effects of root spacing). Cost of edge preparation and added labor of gouging for the back pass, however, should be considered. Also, for thin material, for which a single weld pass may be sufficient, it is uneconomical to use smaller electrodes to weld from two sides. Furthermore, poor accessibility or less favorable welding position (Art. 5.18) may make an unsymmetrical groove weld more economical, because it can be welded from only one side.

When bevel or V grooves can be flame-cut, they cost less than J and U grooves, which require planning or arc-air gouging.

5.17 STANDARD WELDING SYMBOLS

These should be used on drawings to designate welds and provide pertinent information concerning them. The basic parts of a weld symbol are a horizontal line and an arrow:

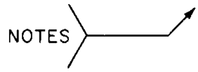
Extending from either end of the line, the arrow should point to the joint in the same manner as the electrode would be held to do the welding.

TABLE 5.12 Number of Passes for Welds

		Single-bevel groove welds (back-up weld not included)		Single-V groove welds (back-up weld not included)		
Weld size,* in	Fillet welds	30° bevel	45° bevel	30° open	60° open	90° open
3/16	1					
¼	1	1	1	2	3	3
5/16	1					
⅜	3	2	2	3	4	6
7/16	4					
½	4	2	2	4	5	7
⅝	6	3	3	4	6	8
¾	8	4	5	4	7	9
⅞		5	8	5	10	10
1		5	11	5	13	22
1⅛		7	11	9	15	27
1¼		8	11	12	16	32
1⅜		9	15	13	21	36
1½		9	18	13	25	40
1¾		11	21			

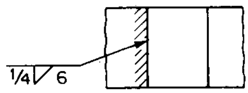
*Plate thickness for groove welds.

Welding symbols should clearly convey the intent of the designer. For the purpose, sections or enlarged details may have to be drawn to show the symbols, or notes may be added. Notes may be given as part of welding symbols or separately. When part of a symbol, the note should be placed inside a tail at the opposite end of the line from the arrow:

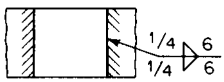


Type and length of weld are indicated above or below the line. If noted below the line, the symbol applies to a weld on the arrow side of the joint, the side to which the arrow points. If noted above the line, the symbol indicates that the other side, the side opposite the one to which the arrow points (not the far side of the assembly), is to be welded.

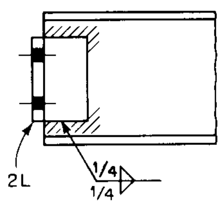
A fillet weld is represented by a right triangle extending above or below the line to indicate the side on which the weld is to be made. The vertical leg of the triangle is always on the left.



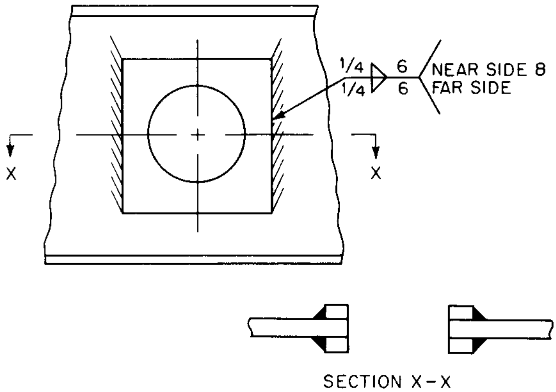
The preceding symbol indicates that a ¼-in fillet weld 6 in long is to be made on the arrow side of the assembly. The following symbol requires a ¼-in fillet weld 6 in long on both sides.



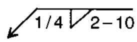
If a weld is required on the far side of an assembly, it may be assumed necessary from symmetry, shown in sections or details, or explained by a note in the tail of the welding symbol. For connection angles at the end of a beam, far-side welds generally are assumed:



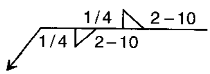
Length of weld is not shown on the symbol in this case because the connection requires a continuous weld the full length of each angle on both sides of the angle. Care must be taken not to omit length unless a continuous full-length weld is wanted. "Continuous" should be written on the weld symbol to indicate length when such a weld is required. In general, a tail note is advisable to specify welds on the far side, even when the welds are the same size.



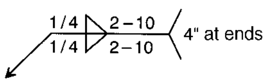
For many members, a stitch or intermittent weld is sufficient. It may be shown as



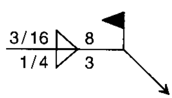
This symbol calls for ¼-in fillet welds on the arrow side. Each weld is to be 2 in long. Spacing of welds is to be 10 in center to center. If the welds are to be staggered on the arrow and other sides, they can be shown as



Usually, intermittent welds are started and finished with a weld at least twice as long as the length of the stitch welds. This information is given in a tail note:



When the welding is to be done in the field rather than in the shop, a triangular flag should be placed at the intersection of arrow and line:



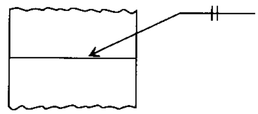
This is important in ensuring that the weld will be made as required. Often, a tail note is advisable for specifying field welds.

A continuous weld all around a joint is indicated by a small circle around the intersection of line and arrow:

Such a symbol would be used, for example, to specify a weld joining a pipe column to a base plate. The all-around symbol, however, should not be used as a substitute for computation of actual weld length required. Note that the type of weld is indicated below the line in the all-around symbol, regardless of shape or extent of joint.

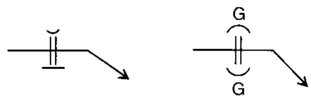
The preceding devices for providing information with fillet welds also apply to groove welds. In addition, groove-weld symbols also must designate material preparation required. This often is best shown on a cross section of the joint.

A square-groove weld (made in thin material) without root opening is indicated by



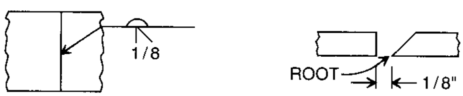
Length is not shown on the welding symbol for groove welds because these welds almost always extend the full length of the joint.

A short curved line below a square-groove symbol indicates weld contour. A short straight line in that position represents a flush weld surface. If the weld is not to be ground, however, that part of the symbol is usually omitted. When grinding is required, it must be indicated in the symbol.



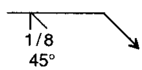
The root-opening size for a groove weld is written in within the symbol indicating the type of weld. For example, a ⅛-in root opening for a square-groove weld is specified by

And a ⅛-in root opening for a bevel weld, not to be ground, is indicated by

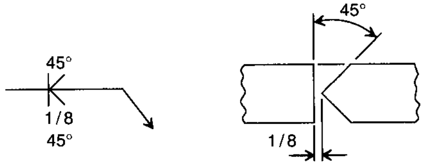


In this and other types of unsymmetrical welds, the arrow not only designates the arrow side of the joint but also points to the side to be shaped for the groove weld. When the arrow has this significance, the intention often is emphasized by an extra break in the arrow.

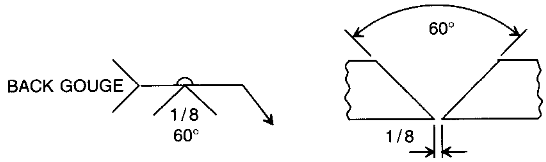
The angle at which the material is to be beveled should be indicated with the root opening:



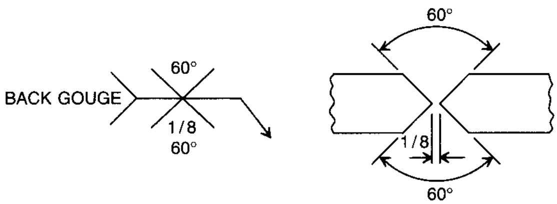
A double-bevel weld is specified by



A single-V weld is represented by



A double-V weld is indicated by



Summary. Standard symbols for various types of welds are summarized in Fig. 5.16. The symbols do not indicate whether backing, spacer, or extension bars are required. These should be specified in general notes or shown in detail drawings. Preparation for J and U welds is best shown by an enlarged cross section. Radius and depth of preparation must be given.

In preparing a weld symbol, insert size, weld-type symbol, length of weld, and spacing, in that order from left to right. The perpendicular leg of the symbol for fillet, bevel, J, and flare-bevel welds should be on the left of the symbol. Bear in mind also that arrow-side and other-side welds are the same size, unless otherwise noted. When billing of detail material discloses the identity of the far side with the near side, the welding shown for the near side also will be duplicated on the far side. Symbols apply between abrupt changes in direction of welding unless governed by the all-around symbol or dimensioning shown.

Where groove preparation is not symmetrical and complete, additional information should be given on the symbol. Also, it may be necessary to give weld-penetration information, as

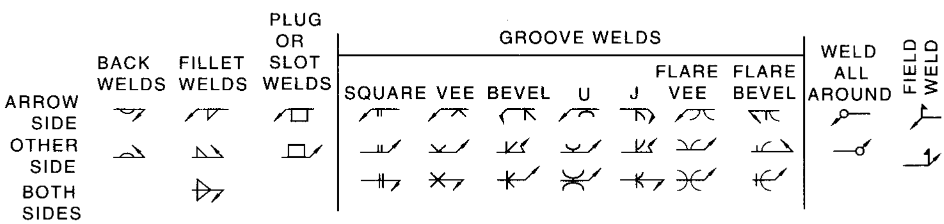


FIGURE 5.16 Summary of welding symbols.

in Fig. 5.17. For the weld shown, penetration from either side must be a minimum of 3⁄16 in. The second side should be back-gouged before the weld there is made.

Welds also may be a combination of different groove and fillet welds. While symbols can be developed for these, designers will save time by supplying a sketch or enlarged cross section. It is important to convey the required information accurately and completely to the workers who will do the job. Actually, it is common practice for designers to indicate what is required of the weld and for fabricators and erectors to submit proposed procedures.

5.18 WELDING POSITIONS

The position of the stick electrode relative to the joint when a weld is being made affects welding economy and quality. In addition, AWS specifications D1.0 and D1.5 prohibit use of some welding positions for some types of welds. Careful designing should eliminate the need for welds requiring prohibited welding positions and employ welds that can be efficiently made.

The basic welding positions are as follows:

Flat, with face of weld nearly horizontal. Electrode is nearly vertical, and welding is performed from above the joint.

Horizontal, with axis of weld horizontal. For groove welds, the face of weld is nearly vertical. For fillet welds, the face of weld usually is about 45° relative to horizontal and vertical surfaces.

Vertical, with axis of weld nearly vertical. (Welds are made upward.)

Overhead, with face of weld nearly horizontal. Electrode is nearly vertical, and welding is performed from below the joint.

Where possible, welds should be made in the flat position. Weld metal can be deposited faster and more easily. Generally, the best and most economical welds are obtained. In a shop, the work usually is positioned to allow flat or horizontal welding. With care in design, the expense of this positioning can be kept to a minimum. In the field, vertical and overhead welding sometimes may be necessary. The best assurance of good welds in these positions is use of proper electrodes by experienced welders.

The AWS specifications require that only the flat position be used for submerged-arc welding, except for certain sizes of fillet welds. Single-pass fillet welds may be made in the flat or the horizontal position in sizes up to 5⁄16 in with a single electrode and up to ½ in with multiple electrodes. Other positions are prohibited.

When groove-welded joints can be welded in the flat position, submerged-arc and gas metal-arc processes usually are more economical than the manual shielded metal-arc process.

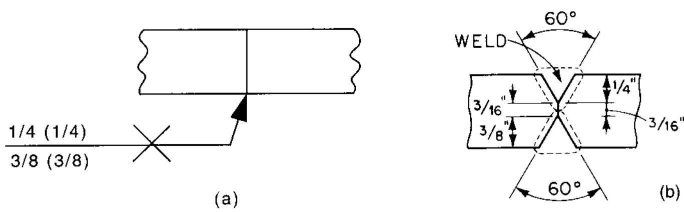


FIGURE 5.17 Penetration information is given on the welding symbol in (*a*) for the weld shown in (*b*). Penetration must be at least 3⁄16 in. Second side must be back-gouged before the weld on that side is made.

GENERAL CRITERIA FOR WELDED CONNECTIONS

5.19 LIMITATIONS ON FILLET-WELD DIMENSIONS

For a given size of fillet weld, cooling rate is faster and restraint is greater with thick plates than with thin plates. To prevent cracking due to resulting internal stresses, specifications set minimum sizes for fillet welds, depending on plate thickness (Table 5.13).

In bridges, seal welds should be continuous. Size should be changed only when required for strength or by changes in plate thickness.

To prevent overstressing of base material at a fillet weld, standard specifications also limit the maximum weld size. They require that allowable stresses in adjacent base material not be exceeded when a fillet weld is stressed to its allowed capacity.

Example. Two angles transfer a load of 120 kips to a ⅜-in-thick plate through four welds (Fig. 5.18). Assume that the welding process is shielded metal-arc using E70XX electrodes and steel is ASTM A36. Use AISC ASD method.

Allowable shear stress in fillet weld $F_v = 0.3 \times$ nominal tensile strength of weld metal $= 0.3 \times 70$ ksi $= 21.0$ ksi. The capacity of 1 in of 5⁄16-in fillet weld $= 0.707(5/16)21.0 = 4.64$ kips. Since there are welds on both sides of plate, the effective thickness required for a 5⁄16-in fillet weld is 0.64 in (Table 5.13). Therefore, the effective capacity of a 5⁄16-in fillet weld is $4.64 \times 0.375/0.64 = 2.7$ kips. For four welds,

$$\text{Length of weld required} = \frac{120}{2.7 \times 4} = 11.1 \text{ in}$$

The minimum size of fillet weld (Table 5.13) should be used, a 3⁄16-in fillet weld with a capacity of $0.707(3/16)21.0 = 2.78$ kips per in. Required minimum plate thickness is 0.38 in (Table 5.13). This weld satisfies the AISC requirement that the length of welds should be at least twice the distance between welds to be fully effective.

TABLE 5.13 Minimum Fillet-Weld Sizes and Plate-Thickness Limits

Sizes of fillet welds,* in			Minimum plate thickness for fillet welds on each side of the plate, in	
Buildings† AWS D1.1	Bridges‡ AWS D1.5	Maximum plate thickness, in§	36-ksi steel	50-ksi steel
⅛¶	—	¼	—	—
3⁄16	—	½	0.38	0.28
¼	¼	¾	0.51	0.37
5⁄16	5⁄16	Over ¾	0.64	0.46

* Weld size need not exceed the thickness of the thinner part joined, but AISC and AWS D1.5 require that care be taken to provide sufficient preheat to ensure weld soundness.

† When low-hydrogen welding is employed, AWS D1.1 permits the thinner part joined to be used to determine the minimum size of fillet weld.

‡ Smaller fillet welds may be approved by the engineer based on applied stress and use of appropriate preheat.

§ Plate thickness is the thickness of the thicker part joined.

¶ Minimum weld size for structures subjected to dynamic loads is 3⁄16 in.

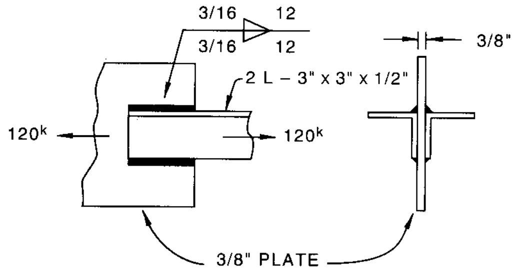


FIGURE 5.18 Welds on two sides of a plate induce stresses in it.

The capacity of a fillet weld per inch of length, with 21.0-ksi allowable stress, can be computed conveniently by multiplying the weld size in sixteenths of an inch by 0.928, since $0.707 \times 21/16 = 0.928$. For example, the capacity of 1 in of 5⁄16-in fillet weld is $0.928 \times 5 = 4.64$, as in the preceding example.

A limitation also is placed on the maximum size of fillet welds along edges. One reason is that edges of rolled shapes are rounded, and weld thickness consequently is less than the nominal thickness of the part. Another reason is that if weld size and plate thickness are nearly equal, the plate corner may melt into the weld, reducing the length of weld leg and the throat. Hence standard specifications require the following: **Along edges of material less than ¼ in thick, maximum size of fillet weld may equal material thickness. But along edges of material ¼ in or more thick, the maximum size should be 1⁄16 in less than the material thickness.**

Weld size may exceed this, however, if drawings definitely show that the weld is to be built out to obtain full throat thickness. AWS D1.1 requires that the minimum effective length of a fillet weld be at least four times the nominal size, or else the weld must be considered not to exceed 25% of the effective length. AWS D1.5 requires that the minimum effective length of a fillet weld be at least four times the nominal size or 1½ in, whichever is greater.

Suppose, for example, a ½-in weld is only 1½ in long. Its effective size is $1\frac{1}{2}/4 = \frac{3}{8}$ in.

Subject to the preceding requirements, intermittent fillet welds may be used in buildings to transfer calculated stress across a joint or faying surfaces when the strength required is less than that developed by a continuous fillet weld of the smallest permitted size. Intermittent fillet welds also may be used to join components of built-up members in buildings. But such welds are prohibited in bridges, in general, because of the requirements for sealing edges to prevent penetration of moisture and to avoid fatigue failures.

Intermittent welds are advantageous with light members, where excessive welding can result in straightening costs greater than the cost of welding. Intermittent welds often are sufficient and less costly than continuous welds (except girder fillet welds made with automatic welding equipment).

Weld lengths specified on drawings are effective weld lengths. They do not include distances needed for start and stop of welding.

To avoid the adverse effects of starting or stopping a fillet weld at a corner, welds extending to corners should be returned continuously around the corners in the same plane for a distance of at least twice the weld size. This applies to side and top fillet welds connecting brackets, beam seats, and similar connections, on the plane about which bending moments are computed. End returns should be indicated on design and detail drawings.

Fillet welds deposited on opposite sides of a common plane of contact between two parts must be interrupted at a corner common to both welds.

If longitudinal fillet welds are used alone in end connections of flat-bar tension members, the length of each fillet weld should at least equal the perpendicular distance between the welds. The transverse spacing of longitudinal fillet welds in end connections should not exceed 8 in unless the design otherwise prevents excessive transverse bending in the connections.

5.20 LIMITATIONS ON PLUG AND SLOT WELD DIMENSIONS

In material $\frac{5}{8}$ in or less thick, the thickness of plug or slot welds should be the same as the material thickness. In material more than $\frac{5}{8}$ in thick, the weld thickness should be at least half the material thickness but not less than $\frac{5}{8}$ in.

Diameter of hole for a plug weld should be at least the depth of hole plus $\frac{5}{16}$ in, but the diameter should not exceed minimum diameter + $\frac{1}{8}$ in, or $2\frac{1}{4}$ times the thickness of the weld metal, whichever is greater.

Thus the hole diameter in $\frac{3}{4}$-in plate could be a minimum of $\frac{3}{4} + \frac{5}{16} = 1\frac{1}{16}$ in. Depth of metal would be at least $\frac{5}{8}$ in $> (1.0625/2.25 = 0.5$ in$) > (\frac{1}{2} \times \frac{3}{4} = \frac{3}{8}$ in$)$.

Plug welds may not be spaced closer center to center than four times the hole diameter.

Length of slot for a slot weld should not exceed 10 times the part thickness. Width of slot should be at least depth of hole plus $\frac{5}{16}$ in, but the width should not exceed the minimum diameter $+\frac{1}{8}$ in or $2\frac{1}{4}$ times the weld thickness.

Thus, width of slot in $\frac{3}{4}$-in plate could be a minimum of $\frac{3}{4} + \frac{5}{16} = 1\frac{1}{16}$ in. Weld metal depth would be at least $\frac{5}{8}$ in $> (1.0625/2.25 = 0.5$ in$) > (\frac{1}{2} \times \frac{3}{4} = \frac{3}{8}$ in$)$. If the minimum depth is used, the slot could be up to $10 \times \frac{3}{4} = 7\frac{1}{2}$ in long.

Slot welds may be spaced no closer than four times their width in a direction transverse to the slot length. In the longitudinal direction, center-to-center spacing should be at least twice the slot length.

5.21 WELDING PROCEDURES

Welds should be qualified and should be made only by welders, welding operators, and tackers qualified as required in AWS D1.1 for buildings and AWS D1.5 for bridges.

Welding should not be permitted under any of the following conditions:

When the ambient temperature is below 0°F

When surfaces are wet or exposed to rain, snow, or high wind.

When welders are exposed to inclement conditions.

Surfaces and edges to be welded should be free from fins, tears, cracks, and other defects. Also, surfaces at and near welds should be free from loose scale, slag, rust, grease, moisture, and other material that may prevent proper welding. AWS specifications, however, permit mill scale that withstands vigorous wire brushing, a light film of drying oil, or antispatter compound to remain. But the specifications require all mill scale to be removed from surfaces on which flange-to-web welds are to be made by submerged-arc welding or shielded metal-arc welding with low-hydrogen electrodes.

Parts to be fillet-welded should be in close contact. The gap between parts should not exceed $\frac{3}{16}$ in. If it is $\frac{1}{16}$ in or more, fillet-weld size should be increased by the amount of separation. The separation between faying surfaces for plug and slot welds, and for butt joints landing on a backing, should not exceed $\frac{1}{16}$ in. Parts to be joined at butt joints should be carefully aligned. Where the parts are effectively restrained against bending due to eccentricity in alignment, an offset not exceeding 10% of the thickness of the thinner part joined, but in no case more than $\frac{1}{8}$ in, is permitted as a departure from theoretical alignment.

When correcting misalignment in such cases, the parts should not be drawn in to a greater slope than ½ in in 12 in.

For permissible welding positions, see Art. 5.18. Work should be positioned for flat welding, whenever practicable.

In general, welding procedures and sequences should avoid needless distortion and should minimize shrinkage stresses. As welding progresses, welds should be deposited so as to balance the applied heat. Welding of a member should progress from points where parts are relatively fixed in position toward points where parts have greater relative freedom of movement. Where it is impossible to avoid high residual stresses in the closing welds of a rigid assembly, these welds should be made in compression elements. Joints expected to have significant shrinkage should be welded before joints expected to have lesser shrinkage, and restraint should be kept to a minimum. If severe external restraint against shrinkage is present, welding should be carried continuously to completion or to a point that will ensure freedom from cracking before the joint is allowed to cool below the minimum specified preheat and interpass temperature.

In shop fabrication of cover-plated beams and built-up members, each component requiring splices should be spliced before it is welded to other parts of the member. Up to three subsections may be spliced to form a long girder or girder section.

With too rapid cooling, cracks might form in a weld. Possible causes are shrinkage of weld and heat-affected zone, austenite-martensite transformation, and entrapped hydrogen. Preheating the base metal can eliminate the first two causes. Preheating reduces the temperature gradient between weld and adjacent base metal, thus decreasing the cooling rate and resulting stresses. Also, if hydrogen is present, preheating allows more time for this gas to escape. Use of low-hydrogen electrodes, with suitable moisture control, also is advantageous in controlling hydrogen content.

High cooling rates occur at arc strikes that do not deposit weld metal. Hence arc strikes outside the area of permanent welds should be avoided. Cracks or blemishes resulting from arc strikes should be ground to a smooth contour and checked for soundness.

To avoid cracks and for other reasons, standard specifications require that under certain conditions, before a weld is made the base metal must be preheated. Tables 5.14 and 5.15 list typical preheat and interpass temperatures. The tables recognize that as plate thickness, carbon content, or alloy content increases, higher preheats are necessary to lower cooling rates and to avoid microcracks or brittle heat-affected zones.

Preheating should bring to the specified preheat temperature the surface of the base metal within a distance equal to the thickness of the part being welded, but not less than 3 in, of the point of welding. This temperature should be maintained as a minimum interpass temperature while welding progresses.

Preheat and interpass temperatures should be sufficient to prevent crack formation. Temperatures above the minimums in Tables 5.14 and 5.15 may be required for highly restrained welds.

For A514, A517, and A852 steels, the maximum preheat and interpass temperature should not exceed 400°F for thicknesses up to 1½ in, inclusive, and 450°F for greater thicknesses. Heat input during the welding of these quenched and tempered steels should not exceed the steel producer's recommendation. Use of stringer beads to avoid overheating is advisable.

Peening sometimes is used on intermediate weld layers for control of shrinkage stresses in thick welds to prevent cracking. It should be done with a round-nose tool and light blows from a power hammer after the weld has cooled to a temperature warm to the hand. The root or surface layer of the weld or the base metal at the edges of the weld should not be peened. Care should be taken to prevent scaling or flaking of weld and base metal from overpeening.

When required by plans and specifications, welded assemblies should be stress-relieved by heat treating. (See AWS D1.1 and D1.5 for temperatures and holding times required.) Finish machining should be done after stress relieving.

Tack and other temporary welds are subject to the same quality requirements as final welds. For tack welds, however, preheat is not mandatory for single-pass welds that are

TABLE 5.14 Requirements of AWS D1.1 for Minimum Preheat and Interpass Temperature (°F) for welds in Buildings*

Thickness of thickest part at point of welding, in	Shielded metal-arc with other than low-hydrogen electrodes	Shielded metal-arc with low-hydrogen electrodes; submerged-arc, gas, metal-arc or flux-cored arc	Shielded metal-arc with low-hydrogen electrodes; submerged-arc, gas metal-arc or flux-cored arc
	ASTM A36†, A53 grade B, A501, A529 A570 all grades	ASTM A36†, A53 grade B, A501, A529 A570 all grades, A572 grades 42, 50, A588	ASTM A572 grades 60 and 65
To ¾	0‡	0‡	50
Over ¾ to 1½	150	50	150
Over 1½ to 2½	225	150	225
Over 2½	200	225	300

*In joints involving combinations of base metals, preheat as specified for the higher-strength steel being welded.
†Use only low-hydrogen electrodes when welding A36 steel more than 1 in thick for dynamically loaded structures.
‡When the base-metal temperature is below 32°F, the base metal should be preheated to at least 70°F and the minimum temperature maintained during welding.

TABLE 5.15 Requirements of AWS D1.5 for Minimum Preheat and Interpass Temperatures (°F) for Welds in Bridges* (Shielded metal-arc, submerged-arc, gas metal-arc or flux-cored arc.)

Thickness of thickest part at point of welding, in	ASTM A36/M270M grade 250, A572 grade 50/M270M grade M345 A588/M270M grade 345			A514/M270M grades 690 and 690W / ASTM A852/M270M grade 485W		
		Fracture critical			Fracture critical	
	General	A36, A572	A588	General	ASTM A852/ M270M grade 485W	ASTM A514/ M270M grades 690 and 690W
To ¾	50	100	100	50	100	**
Over ¾ to 1½	70	150	200	125	200	**
Over 1½ to 2½	150	200	300	175	300	**
Over 2½	225	300	350	225	350	**

*In joints involving combinations of base metals, preheat as specified for the higher-strength steel being welded. When the base-metal temperature is below 32°F, preheat the base metal to at least 70°F, and maintain this minimum temperature during welidng.
**For fracture critical members in this steel, the temperatures (min./max., °F) are as follows for the indicated thickness range: ¼ to ⅜ in, 100/150; + ⅜ to ½ in, 100/325; +½ to ¾ in, 200/400; +¾ to 1 in, 150/400; +1 to 2 in, 200/400; +2 in, 300/460.

remelted and incorporated into continuous submerged-arc welds. Also, defects such as undercut, unfilled craters, and porosity need not be removed before final submerged-arc welding. Welds not incorporated into final welds should be removed after they have served their purpose, and the surface should be made flush with the original surface.

Before a weld is made over previously deposited weld metal, all slag should be removed, and the weld and adjacent material should be brushed clean.

Groove welds should be terminated at the ends of a joint in a manner that will ensure sound welds. Where possible, this should be done with the aid of weld tabs or runoff plates. AWS D1.5 requires removal of weld tabs after completion of the weld in bridge construction. AWS D1.1 does not require removal of weld tabs for statically loaded structures but does require it for dynamically loaded structures. The ends of the welds then should be made smooth and flush with the edges of the abutting parts.

After welds have been completed, slag should be removed from them. The metal should not be painted until all welded joints have been completed, inspected, and accepted. Before paint is applied, spatter, rust, loose scale, oil, and dirt should be removed.

AWS D1.1 and D1.5 present details of techniques acceptable for welding buildings and bridges, respectively. These techniques include handling of electrodes and fluxes and maximum welding currents

5.22 WELD QUALITY

A basic requirement of all welds is thorough fusion of weld and base metal and of successive layers of weld metal. In addition, welds should not be handicapped by craters, undercutting, overlap, porosity, or cracks. (AWS D1.1 and D1.5 give acceptable tolerances for these defects.) If craters, excessive concavity, or undersized welds occur in the effective length of a weld, they should be cleaned and filled to the full cross section of the weld. Generally, all undercutting (removal of base metal at the toe of a weld) should be repaired by depositing weld metal to restore the original surface. Overlap (a rolling over of the weld surface with lack of fusion at an edge), which may cause stress concentrations, and excessive convexity should be reduced by grinding away of excess material (see Figs. 5.19 and 5.20). If excessive

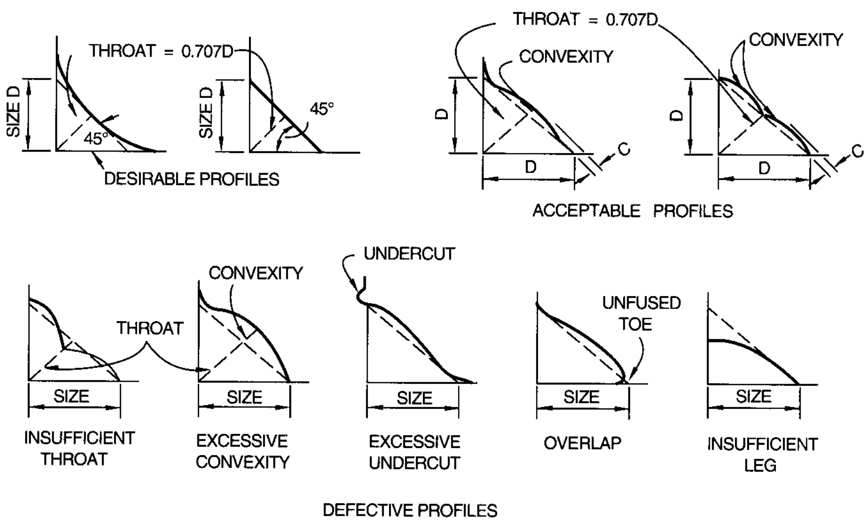


FIGURE 5.19 Profiles of fillet welds.

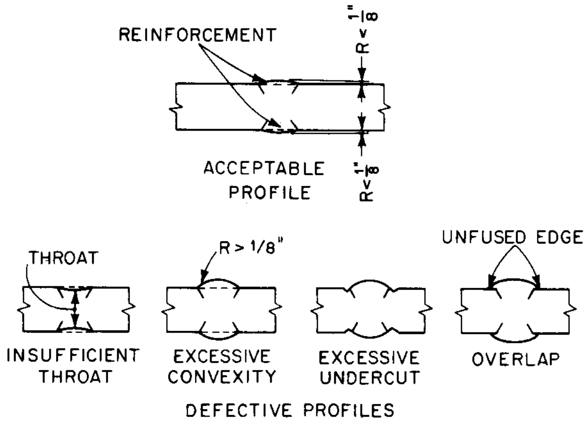


FIGURE 5.20 Profiles of groove welds.

porosity, excessive slag inclusions, or incomplete fusion occur, the defective portions should be removed and rewelded. If cracks are present, their extent should be determined by acid etching, magnetic-particle inspection, or other equally positive means. Not only the cracks but also sound metal 2 in beyond their ends should be removed and replaced with the weld metal. Use of a small electrode for this purpose reduces the chances of further defects due to shrinkage. An electrode not more than $\frac{5}{32}$ in in diameter is desirable for depositing weld metal to compensate for size deficiencies.

AWS D1.1 limits convexity C to the values in Table 5.16. AWS D1.5 limits C to 0.06 in plus 7% of the measured face of the weld.

Weld-quality requirements should depend on the job the welds are to do. Excessive requirements are uneconomical. Size, length, and penetration are always important for a stress-carrying weld and should completely meet design requirements. Undercutting, on the other hand, should not be permitted in main connections, such as those in trusses and bracing, but small amounts might be permitted in less important connections, such as those in platform framing for an industrial building. Type of electrode, similarly, is important for stress-carrying welds but not so critical for many miscellaneous welds. Again, poor appearance of a weld is objectionable if it indicates a bad weld or if the weld will be exposed where aesthetics is a design consideration, but for many types of structures, such as factories, warehouses, and incinerators, the appearance of a good weld is not critical. A sound weld is important. But a weld entirely free of porosity or small slag inclusions should be required only when the type of loading actually requires this perfection.

Welds may be inspected by one or more methods: visual inspection; nondestructive tests, such as ultrasonic, x-ray, dye penetration, and magnetic particle; and cutting of samples from

TABLE 5.16 AWS D1.1 Limits on Convexity of Fillet Welds

Measured leg size or width of surface bead, in	Maximum convexity, in
$\frac{5}{16}$ or less	$\frac{1}{16}$
Over $\frac{5}{16}$ but less than 1	$\frac{1}{8}$
1 or more	$\frac{3}{16}$

finished welds. Designers should specify which welds are to be examined, extent of the examination, and methods to be used.

5.23 WELDING CLEARANCE AND SPACE

Designers and detailers should detail connections to ensure that welders have ample space for positioning and manipulating electrodes and for observing the operation with a protective hood in place. Electrodes may be up to 18 in long and ⅜ in in diameter.

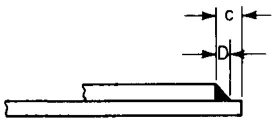


FIGURE 5.21 Minimum landing for a fillet weld.

In addition, adequate space must be provided for deposition of the required size of fillet weld. For example, to provide an adequate landing c (in) for the fillet weld of size D (in) in Fig. 5.21, c should be at least $D + 5/16$. In building-column splices, however, $c = D + 3/16$ often is used for welding splice plates to fillers.

DESIGN OF CONNECTIONS

Overview. Connection design is both an art and a science. The science involves equilibrium, limit states, load paths, and the lower bound theorem of limit analysis. The art involves determination of the most efficient load paths which is necessary because most connections are statically indeterminate. The lower bound theorem of limit analysis can be stated as follows. If a distribution of forces within a structure (or connection, which is localized structure) can be found which is in equilibrium with the external load and which satisfies the limit states, then the externally applied load is less than or at most equal to the load which would cause connection failure. In other words, any solution for a connection which satisfies equilibrium and the limit states yields a safe connection, and this represents the science of connection design. Finding the internal force distribution (or load path) that maximizes the external load at which a connection fails represents the art of connection design. This maximized external load is also the true failure load when the internal force distribution results in satisfaction of compatibility (no gaps and tears) within the connection in addition to satisfying equilibrium and the limit states. Strictly speaking, the lower bound theorem applies only to yield limit states in ductile structures. Therefore, limit states involving stability and fracture must be considered to preclude these modes of failure.

General Procedure. Determine the external (applied) loads, also called required strengths, and their lines of action. Make a preliminary layout, preferably to scale. The connection should be as compact as possible to conserve material and to minimize interferences with utilities, equipment, and access. Decide on where bolts and welds will be used and select bolt type and size. Decide on a load path through the connection. For a statically determinate connection, there is only one, but for indeterminate connections there are many possibilities. Use judgment, experience, and published information to arrive at the best load path. Provide sufficient strength, stiffness, and ductility, using the limit states identified for each part of the load path, to give the connection sufficient design strength, so that it is adequate to carry the given loads. Complete the preliminary layout, check specification required spacings, and finally, check to ensure that the connection can be fabricated and erected economically. The examples of this chapter will demonstrate this procedure.

Economic Considerations. For any given connection situation, it is usually possible to arrive at several satisfactory solutions. Where there is a possibility of using bolts or welds, let the economics of fabrication and erection play a role in the choice. Different fabricators and erectors in different parts of the country have their preferred ways of working, and as long as the principles of connection design are followed to achieve a safe connection, local preferences should be accepted. Some additional considerations which will result in more economical connections are as follows:

1. For shear connections, provide the actual loads and allow the use of single plate and single angle shear connections. Do not specify full depth connections or rely on the AISC uniform load tables.
2. For moment connections, provide the actual moments and the actual shears. Also, provide a ‘‘breakdown’’ of the total moment, with the gravity moment and lateral moment due to wind or seismic loads listed separately. This is needed to do a proper check for column web doubler plates. If stiffeners are required, allow the use of fillet welds in place of complete joint penetration welds. To avoid the use of stiffeners, consider redesigning with a heavier column to eliminate them.
3. For bracing connections, in addition to providing the brace force, also provide the beam shear and axial transfer force. The transfer force, sometimes called ‘‘drag’’ or ‘‘drag through’’ force, is the axial force that must be transferred to the opposite side of the column. The transfer force is not necessarily the beam axial force obtained from a computer analysis of the structure. A misunderstanding of transfer forces can lead to both uneconomic and unsafe connections.

5.24 MINIMUM CONNECTIONS

In buildings, connections carrying calculated stresses, except for lacing, sag bars, and girts, should be designed to support at least 6 kips.

In highway bridges, connections, except for lacing bars and handrails, should contain at least two fasteners or equivalent weld. The smallest angle that may be used in bracing is $3 \times 2\frac{1}{2}$ in. In railroad bridges, the minimum number of fasteners per plane of connection is three.

5.25 HANGER CONNECTIONS

In buildings, end connections for hangers should be designed for the full loads on the hangers. In trusses, however, the AISC specification requires that end connections should develop not only the design load but also at least 50% of the effective strength of the members. This does not apply if a smaller amount is justified by an engineering analysis that considers other factors, including loads from handling, shipping, and erection. This requirement is intended only for shop-assembled trusses where such loads may be significantly different from the loads for which the trusses were designed.

In highway bridges, connections should be designed for the average of the calculated stress and the strength of the members. But the connections should be capable of developing at least 75% of the strength of the members.

In railroad bridges, end connections of main tension members should have a strength at least equal to that of the members. Connections for secondary and bracing members should develop at least the average of the calculated stress and the strength of the members. But bracing members used as ties or struts to reduce the unsupported length of a member need not be connected for more than the flexural strength of that member.

When a connection is made with fasteners, the end fasteners carry a greater load than those at the center of the connection. Because of this, AISC and AASHTO reduce fastener strength when the length of a connection exceeds 50 in.

5.25.1 Bolted Lap Joints

Tension members serving as hangers may be connected to supports in any of several ways. One of the most common is use of a lap joint, with fasteners or welds.

Example—AISC ASD. A pair of A36 steel angles in a building carry a 60-kip vertically suspended load (Fig. 5.22). Size the angles and gusset plate, and determine the number of ⅞-in-diameter A325N (threads included) bolts required.

Bolts. Capacity of one bolt in double shear (i.e., two shear planes) is $r_v = 2 \times 21 \times 0.4418 = 18.6$ kips. Thus $60/18.6 = 3.23$ or 4 bolts are required.

Angles. Gross area required $= 60/21.6 = 2.78$ in^2. Try two angles $3 \times 3 \times 1/4$ with area $= 2.88$ in^2. Then net area $A_n = 2.88 - 1 \times 0.25 \times 2 = 2.38$ in^2, and effective net area $A_e = UA_n = 0.85 \times 2.38 = 2.02$ in^2. (The U factor accounts for shear lag, because only one leg of an angle is bolted.) The allowable load based on fracture of the net area is $2.02 \times 0.5 \times 58 = 58.6$ kips < 60 kips. The angles are not adequate. Try two angles $3 \times 3 \times 5/16$ with area $= 3.55$ in^2. The net area is $A_n = 3.55 - 1 \times 0.3125 \times 2 = 2.92$ in^2, $A_e = 0.85 \times 2.92 = 2.48$ in^2, and the allowable load based on net area fracture is $2.48 \times 0.5 \times 58 = 71.9$ kips > 60 kips—OK.

Gusset. Assume that the gusset is 10 in wide at the top bolt. Thus the net width is $10 - 1 = 9$ in, and since, for $U = 0.85$, this is greater than $0.85 \times 10 = 8.5$, use 8.5 in for the effective net width. For the gross area used to determine the yield limit state, the effective width of the plate at the top bolt can be determined using the Whitmore section described in the AISC ASD and LRFD manuals. For this example, the Whitmore section width is $2(9 \times \tan 30°) = 10.4$ in. Because this is greater than the actual width of 10 in, use 10 in. Therefore, thickness required $= 60/(10 \times 21.6) = 0.28$ in. For the effective net

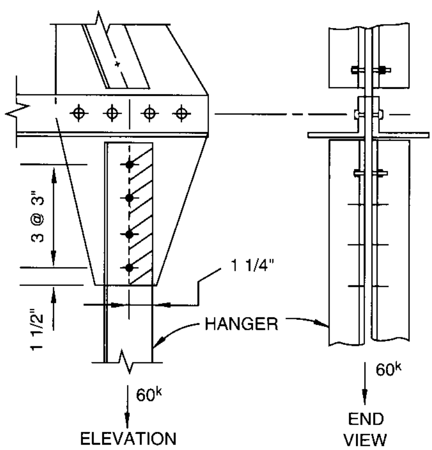


FIGURE 5.22 Hanger supported by gusset-plate connection with bolts.

area (fracture limit state), thickness required = 60/(8.5 × 29) = 0.24 in. Yield controls at 0.28 in. Use a 5⁄16-in gusset.

Additional checks required are for bearing and angle-leg tearout (block shear fracture).

Bearing. For a bolt spacing of 3 in and an edge distance of 1½ in, the allowable bearing stress is $1.2F_u = 1.2 \times 58 = 69.6$ ksi, and the allowable bearing load is $69.6 \times 0.3125 \times 0.875 \times 4 = 76.1$ kips > 60 kips—OK.

Tearout. The cross-hatched region of the angle leg in Fig. 5.22 can tear out as a block. To investigate this, determine the tearout capacity of the angles: The shear area is

$$A_v = (10.5 - 3.5 \times 0.9375) \times 0.3125 \times 2 = 4.51 \text{ in}^2$$

The tension area is

$$A_t = (1.25 - 0.5 \times 0.9375) \times 0.3125 \times 2 = 0.49 \text{ in}^2$$

The tearout resistance is

$$P_{to} = 4.51 \times 0.3 \times 58 + 0.49 \times 0.5 \times 58 = 92.7 \text{ kips} > 60 \text{ kips—OK}$$

5.25.2 Welded Lap Joints

For welded lap joints, standard specifications require that the amount of lap be five times the thickness of the thinner part joined, but not less than 1 in. Lap joints with plates or bars subjected to axial stress should be fillet-welded along the end of both lapped parts, unless the deflection of the lapped parts is sufficiently restrained to prevent the joint from opening under maximum loading.

Welded end connections have the advantage of avoiding deductions of hole areas in determining net section of tension members.

If the tension member consists of a pair of angles required to be stitched together, ring fills and fully tensioned bolts can be used (Fig. 5.23*a*). With welded connections, welded stitch bars (Fig. 5.23*b*) should be used. Care should be taken to avoid undercutting at the toes of the angles at end-connection welds and stitch welds. In building design end-connection welds may be placed equally on the toe and heel of the angles, ignoring the small eccentricity. The welds should be returned around the end of each angle.

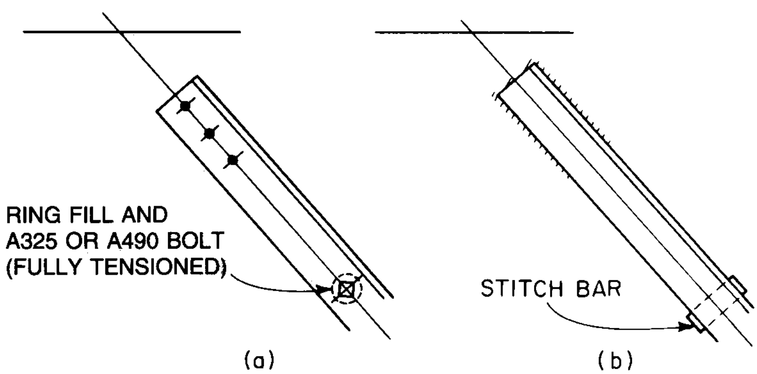


FIGURE 5.23 Pair of steel angles stitched together (*a*) with ring fill and high-strength bolts, (*b*) with stitch bar and welds.

Welded connections have the disadvantage of requiring fitting. Where there are several identical pieces, however, jigs can be used to reduce fit-up time.

Example—AISC ASD. Suppose the hanger in the preceding example is to be connected to the gusset plate by a welded lap joint with fillet welds (Fig. 5.24). Since the two angles will now have no holes, no net-area fracture-limit-state checks need be made. As before, the gross area required is 60/21.6 = 2.78 in^2. Try two angles 3 × 3 × ¼, with gross area = 2.88 in^2. Because only one leg of the angle is connected, shear lag is a consideration here, just as it was in the bolted case. Thus the effective area is 0.85 × 2.88 = 2.45 in^2, and the capacity is 2.45 × 29.0 = 71.0 kips > 60 kips—OK. Note that the fracture allowable stress of $0.5F_u$ is used here. The limit state within the confines of the connection is fracture, not yield. Yield is the limit state in the angles outside the connection where the stress distribution in the angles becomes uniform.

The maximum-size fillet weld that can be used along the edge of the ¼-in material is 3⁄16 in. To provide space for landing the fillet welds along the back and edge of each angle, the minimum width of gusset plate should be 3 + 2(3⁄16 + 5⁄16) = 4 in, if 3⁄16-in welds are used. A preliminary sketch of the joint (Fig. 5.24) indicates that with a minimum width of 4 in, the gusset plate will be about 6 in wide at the ends of the angles, where the load will reach 60 kips on transfer from the welds. For a 6-in width, the required thickness of plate is 60/(22 × 6) = 0.46 in. Use a ½-in plate.

For a ½-in plate, the minimum-size fillet weld is 3⁄16 in. Since the maximum size permitted also is 3⁄16 in, use a 3⁄16-in fillet weld. If an E70XX electrode is used to make the welds, the allowable shear is 21 ksi. The capacity of the welds then is 0.707 × 3⁄16 × 21 = 2.78 kips per in. Length of weld required equals 60(2 × 2.78) = 10.8 in. Hence supply a total length of fillet welds of at least 11 in, with at least 5½ in along the toe and 5½ in along the heel of each angle.

To check the ½-in-thick gusset plate, divide the capacity of the two opposite welds by the allowable shear stress: Gusset thickness required is 2 × 2.78/14.5 = 0.38 in < 0.50 in—OK. One rule of thumb for fillet welds on both faces opposite each other is to make the gusset thickness twice the weld size. However, this rule is too conservative in the present case because the gusset cannot fail through one section under each weld. A better way is to check for gusset tearout. The shear tearout area is A_v = 6 × 0.5 × 2 = 6.0 in^2, and the tension tearout area is A_t = 3 × 0.5 × 1 = 1.5 in^2. Thus the tearout capacity is P_{t0} = 6 × 0.3 × 58 + 1.5 × 0.5 × 58 = 148 kips > 60 kips—OK. This method recognizes a true limit state, whereas matching fillet-weld size to gusset-plate thickness does not.

5.25.3 Fasteners in Tension

As an alternative to lap joints, with fasteners or welds in shear, hangers also may be supported by fasteners in tension. Permissible tension in such fasteners equals the product of the reduced cross-sectional area at threads and allowable tensile stress. The stress is based

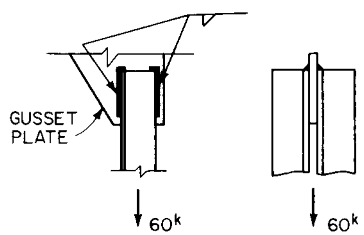


FIGURE 5.24 Hanger supported by gusset-plate connection with welds.

on bolts with hexagonal or square heads and nuts. Flattened-or countersunk-head fasteners, therefore, should not be used in joints where they will be stressed in tension.

Bolts designed for tension loads usually have a deliberately applied pretension. The tension is maintained by compression in the connected parts. A tensile force applied to a fastener relieves the compression in the connected parts without increasing the tension in the fastener. Unless the tensile force is large enough to permit the connected parts to separate, the tension in the fastener will not exceed the pretension.

Generally, the total force on a fastener in tension equals the average force on all the fasteners in the joint plus force due to eccentricity, if present. Sometimes, however, the configuration of the joint produces a prying effect on the fasteners that may be serious and should be investigated.

Figure 5.25*a* shows a connection between the flange of a supporting member and the flange of a T shape (tee, half wide-flange beam, or pair of angles with plate between), with bolts in tension. The load P is concentrically applied. If the prying force is ignored, the average force on any fastener is P/n, where n is the total number of fasteners in the joint. But, as indicated in Fig. 5.25*b*, distortion of the T flange induces an additional prying force Q in the fastener. This force is negligible when the connected flanges are thick relative to the fastener gages or when the flanges are thin enough to be flexible.

Note that in Fig. 5.25*b* though only the T is shown distorted either flange may distort enough to induce prying forces in the fasteners. Hence theoretical determination of Q is extremely complex. Research has shown, however, that the following approach from the AISC "Manual of Steel Construction—ASD" gives reliable results: Let α = ratio of moment M_2 at bolt line to moment δM_1, at stem line where δ = ratio of net area (along the line of bolts) to gross area (at face of stem or angle leg), and α' = value of α for which the required thickness is a minimum or allowable tension per bolt is a maximum.

$$\alpha' = \frac{1}{\delta(1 + \rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \tag{5.2}$$

The allowable tensile load (kips) per fastener, including the effect of prying action, is given by

$$T_a = B\left(\frac{t}{t_c}\right)^2 (1 + \delta\alpha') \tag{5.3}$$

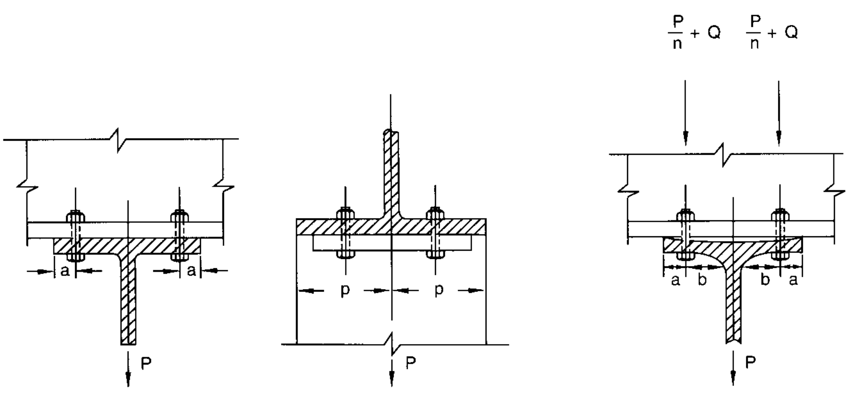


FIGURE 5.25 Two connections with bolts (*a*) develop prying action (*b*), imposing forces Q on the bolts.

where $t_c = \sqrt{\dfrac{8Bb'}{pF_y}}$

B = allowable bolt tension, including the effect of shear, if any, kips
t = thickness of thinnest connected flange or angle leg, in
p = length of flange or angle leg tributary to a bolt, in
F_y = flange or angle yield stress, ksi
$a' = a + d/2$
a = distance from center of fastener to edge of flange or angle leg, in (not to exceed $1.25b$)
$b' = b - d/2$
b = distance from center of fastener to face of tee stem or angle leg, in
$\rho = b'/a'$
$\delta = 1 - d'/p$
d = bolt diameter, in
d' = bolt-hole diameter, in

If $\alpha' > 1$, $\alpha' = 1$ should be used to calculate T_a. If $\alpha' < 0$, $T_a = B$. This approach is also used in the AISC LRFD Manual as will be seen in later examples.

Example—AISC ASD A tee stub hanger is to transfer a 60-kip load to the flange of a wide-flange beam through four 1-in-diameter A325 bolts in tension (Fig. 5.26). The tee stub is half of a W18 × 60 made of A572 grade 50 steel, with $F_y = 50$ ksi, $b = 1.792$ in, $a = 1.777$ in $< (1.25 \times 1.792 = 2.24$ in). Other needed dimensions are given in Fig. 5.26. Check the adequacy of this connection.

$$B = 44 \times 0.7854 = 34.6 \text{ kips}$$

$$t = 0.695 \text{ in}$$

$$p = 4.5 \text{ in}$$

$$a' = 1.775 + 0.5 = 2.277 \text{ in}$$

$$b' = 1.792 - 0.5 = 1.292 \text{ in}$$

$$d' = 1.0625 \text{ in}$$

$$\rho = 1.292/2.277 = 0.567$$

Substitution in Eq. (5.2) yields

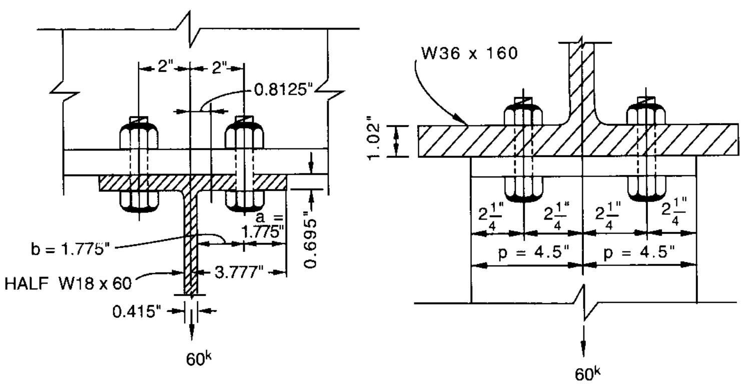


FIGURE 5.26 Example of hanger with bolts in tension.

$$t_c = \sqrt{\frac{8 \times 34.6 \times 1.292}{4.5 \times 50}} = 1.261 \text{ in}$$

With $\delta = 1 - 1.0625/4.5 = 0.7639$ and $t_c = 1.261$,

$$\alpha' = \frac{1}{0.7639 \times 1.568}\left[\left(\frac{1.261}{0.695}\right)^2 - 1\right] = 1.915$$

Since $\alpha' > 1$, use $\alpha' = 1$ to compute the allowable tensile load per fastener, including prying action:

$$T_a = 34.6\left(\frac{0.695}{1.261}\right)^2 1.7639 = 18.5 \text{ kips}$$

Since the load per bolt $T = 60/4 = 15$ kips is less than the allowable load per bolt of 18.5 kips, the connection is satisfactory

This method checks both the bolts and the tee flange or angle leg. No further checks are required. The prying force Q is not explicitly calculated but instead is built into the formulas. The prying force can be calculated as follows:

$$\alpha = \frac{1}{\delta}\left[\frac{T}{B}\left(\frac{t_c}{t}\right)^2 - 1\right] \tag{5.4}$$

where T is the load per bolt (kips). And

$$Q = B\alpha\rho\delta\left(\frac{t}{t_c}\right)^2 \tag{5.5}$$

Substitution of the connection dimensions and $T = 60/4 = 15$ kips gives

$$\alpha = \frac{1}{0.7639}\left[\frac{15}{34.6}\left(\frac{1.261}{0.695}\right)^2 - 1\right] = 0.559$$

$$Q = 34.6 \times 0.559 \times 0.567 \times 0.7639\left(\frac{0.695}{1.261}\right)^2 = 2.55 \text{ kips}$$

and the total load per bolt is $15 + 2.55 = 17.55$ kips. Note that the previous conclusion that the connection is satisfactory is confirmed by this calculation; that is,

$$T_a = 18.5 \text{ kips} > T + Q = 17.55 \text{ kips}$$

Example—AASHTO ASD. AASHTO requires that Q be estimated by the empirical formula

$$Q = \left[\frac{3b}{8a} - \frac{t^3}{20}\right]T \tag{5.6}$$

where all quantities are as defined above, except that b is measured from the center of the bolt to the toe of the fillet of the flange or angle leg; there is no restriction on a, and, considering the data in the preceding example, $B = 39.5 \times 0.7854 = 31.0$ kips. Hence

$$Q = \left[\frac{3 \times 1.1875}{8 \times 1.777} - \frac{0.695^3}{20}\right] 15 = 3.51 \text{ kips}$$

Since $T + Q = 15 + 3.51 = 18.5$ kips < 31.0 kips, the bolts are OK. The tee flange, however, should be checked independently. This can be done as follows.

Assume that the prying force $Q = 3.51$ acts at a distance a from the bolt line. The moment at the bolt line then is

$$M_b = 3.51 \times 1.777 = 6.24 \text{ kip-in}$$

and that at the toe of the fillet, with $T + Q = 18.5$ kips, is

$$M_f = 3.51 \times (1.777 + 1.1875) - 18.5 \times 1.1875 = -11.6 \text{ kip-in}$$

The maximum moment in the flange is thus 11.6 kip-in, and the bending stress in the flange is

$$f_b = \frac{11.6 \times 6}{4.5 \times 0.695^2} = 32.0 < 0.75 \times 50 = 37.5 \text{ ksi—OK}$$

5.25.4 Welded Butt Joints

A hanger also may be connected with a simple welded butt joint (Fig. 5.27). The allowable stress for the complete-penetration groove weld is the same as for the base metal used.

Examples—AISC ASD. A bar hanger carries a 60-kip load and is supported through a complete-penetration groove weld at the edge of a tee stub.

For A36 steel with allowable tensile stress of 22 ksi, the bar should have an area of $60/22 = 2.73$ in^2. A bar $5\frac{1}{2} \times \frac{1}{2}$ with an area of 2.75 in^2 could be used. The weld strength is equal to that of the base metal. Hence no allowance need be made for its presence.

Example—AASHTO ASD. Suppose, however, that the hanger is to be used in a highway bridge and the load will range from 30 kips in compression (minus) to 60 kips in tension (plus). The design then will be governed by the allowable stresses in fatigue. These depend on the stress range and the number of cycles of load the structure will be subjected to. Design is to be based on 100,000 cycles. Assuming a redundant load path and the detail of Fig. 5.27, load category C applies with an allowable stress range $SR = 35.5$ ksi, according

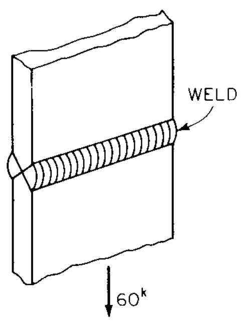


FIGURE 5.27 Groove weld for hanger connection.

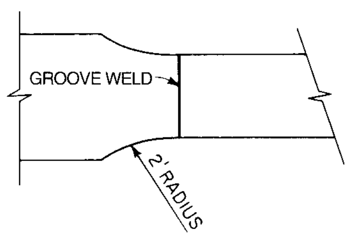


FIGURE 5.28 Recommended taper for unequal-width plates at groove-welded splice.

to AASHTO. For A36 steel, the maximum tensile stress is 20 ksi. At 60-kip tension, the area required is $60/20 = 3$ in^2. A bar $5 \times 5/8$ could be tried. It has an area of 3.125 in^2. Then the maximum tensile stress is $60/3.125 = 19.2$ ksi, and the maximum compressive stress is $30/3.125 = 9.6$ ksi. Thus the stress range is $19.2 - (-9.6) = 28.8$ ksi < 35.5 ksi—OK. The $5 \times 5/8$ bar is satisfactory.

As another example, consider the preceding case subjected to 2 million cycles. The allowable stress range is now 13 ksi. The required area is $[60 - (-30)]/13 = 6.92$ in^2. Try a bar $5 \times 1\frac{1}{2}$ with area 7.5 in^2. The maximum tensile stress is $60/7.5 = 8.0$ ksi (<20 ksi), and the maximum compressive stress is $30/7.5 = 4.0$ ksi. Thus the actual stress range is $8.0 - (-4.0) = 12.0$ ksi < 13.0 ksi—OK. The $5 \times 1\frac{1}{2}$ bar is satisfactory.

5.26 TENSION SPLICES

Design rules for tension splices are substantially the same as those for hanger connections. In buildings, splices should develop the strength required by the stresses at point of splice. For groove welds, however, the full strength of the smaller spliced member should be developed.

In highway bridges, splices should be designed for the larger of the following: 75% of the strength of the member or the average of the calculated stress at point of splice and the strength of the member there. Where a section changes size at a splice, the strength of the smaller section may be used in sizing the splice. In tension splices, the strength of the member should be calculated for the net section.

In railroad bridges, tension splices in main members should have the same strength as the members. Splices in secondary and bracing members should develop the average of the strength of the members and the calculated stresses at the splices.

When fillers are used, the requirements discussed in Art. 5.13 should be satisfied.

In groove-welded tension splices between parts of different widths or thicknesses, a smooth transition should be provided between offset surfaces or edges. The slope with respect to the surface or edge of either part should not exceed 1:2.5 (equivalent to about 5:12 or 22°). Thickness transition may be accomplished with sloping weld faces, or by chamfering the thicker part, or by a combination of the two methods.

Splices may be made with complete-penetration groove welds, preferably without splice plates. The basic allowable unit stress for such welds is the same as for the base metal joined. For fatigue, however, the allowable stress range F_{sr} for base metal adjacent to continuous flange-web fillet welds may be used for groove-welded splices only if

1. The parts joined are of equal thickness.
2. The parts joined are of equal widths or, if of unequal widths, the parts are tapered as indicated in Fig. 5.28, or, except for A514 and A517 steels, tapered with a uniform slope not exceeding 1:2.5.
3. Weld soundness is established by radiographic or ultrasonic testing.
4. The weld is finished smooth and flush with the base metal on all surfaces by grinding in the direction of applied stress, leaving surfaces free from depressions. Chipping may be used if it is followed by such grinding. The grinding should not reduce the thickness of the base metal by more than $1/32$ in or 5% of the thickness, whichever is smaller.

Groove-welded splices that do not conform to all these conditions must be designed for reduced stress range assigned to base metal adjacent to groove welds.

For bolted flexural members, splices in flange parts between field splices should be avoided. In any one flange, not more than one part should be spliced at the same cross section.

Fatigue need not be considered when calculating bolt stresses but must be taken into account in design of splice plates.

Example—AASHTO ASD. A plate girder in a highway bridge is to be spliced at a location where a 12-in-wide flange is changed to a 16-in-wide flange (Fig. 5.29). Maximum bending moments at the splice are +700 kip-ft (tension) and −200 kip-ft (compression). Steel is A36. The girder is redundant and is subjected to not more than 2 million cycles of stress. The connection is slip-critical; bolts are A325SC, ⅞ in in diameter, in standard holes. The web has nine holes for ⅞-in bolts. The tension-flange splice in Fig. 5.29 is to be checked.

According to AASHTO specifications for highway bridges, members loaded primarily in bending should be designed for stresses computed for the gross section. If, however, the areas of flange holes exceed 15% of the flange area, the excess should be deducted from the gross area. With two bolt holes, net width of the flange is $12 - 2 = 10$ in. With four holes, the net width along a zigzag section through the holes is $12 - 4 + 2 \times 3^2/(4 \times 3) = 9.5$ in, with the addition of $s^2/4g$ for two chains, where s = bolt pitch = 3 in and g = gage = 3 in. The four-hole section, with less width, governs. In this case, the ratio of hole area to flange area is $(12 - 9.5)/12 = 0.21 > 0.15$. The area reduction for the flange is $12(0.21 - 0.15)0.875 = 0.63$ in², and reduced flange area is $12 \times 0.875 - 0.63 = 9.87$ in². With moment of inertia I_{gw} of the web equal to $0.3125(48)^3/12 = 2880$ in⁴, the effective gross moment inertia of the girder is

$$I_g = 9.87\left(\frac{48.875}{2}\right)^2 = 2 + 2880 = 14{,}670 \text{ in}^4$$

AASHTO requires that the design tensile stress on the net section not exceed $0.5F_u$ = 29 ksi. The net moment of inertia with flange area = $9.5 \times 0.875 = 8.3125$ in² and nine web holes is

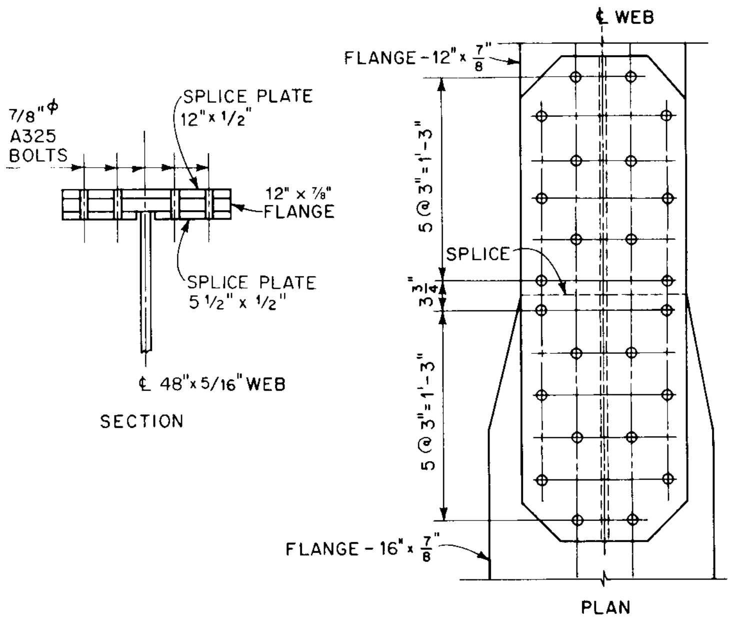


FIGURE 5.29 Tension-flange splice for highway-bridge plate girder.

$$I_n = 2880 + 2 \times 8.3125 \left(\frac{48.875}{2}\right)^2 - (2 \times 1 \times\ 0.3125(5^2 + 10^2 + 15^2 + 20^2)$$

$$= 2880 + 9928 - 469 = 12{,}339 \text{ in}^4$$

The net moment of inertia of the web is

$$I_{nw} = 2880 - 469 = 2411 \text{ in}^4$$

Bending stresses in the girder are computed as follows:

Gross section: $f_b = \dfrac{700 \times 12 \times 24.875}{14{,}670} = 14.2 \text{ ksi} < 20 \text{ ksi—OK}$

Net section: $f_b = 14.2 \times 9.87/8.3125 = 16.9 \text{ ksi} < 29 \text{ ksi—OK}$

Stress range: $f_{sr} = \dfrac{[700 - -200)]12 \times 24.875}{14{,}670} = 18.3 \text{ ksi} \approx 18 \text{ ksi—OK}$

1. The average of the calculated design stress at the point of splice and the allowable stress of the member at the same point

2. 75% of the allowable stress in the member

From criterion 1, the design stress $F_{d_1} = (14.2 + 20)/2 = 17.1$ ksi, and from criterion 2, the design stress $F_{d_2} = 0.75 \times 20 = 15$ ksi. Therefore, design for $F_d = 17.1$ ksi on the gross section.

The required splice-plate sizes are determined as follows:

$$\text{Design moment} = \frac{700 \times 17.1}{14.2} = 843 \text{ kip-ft}$$

$$\text{Moment in flange} = 843 \left(\frac{14{,}670 - 2880}{14{,}670}\right) = 678 \text{ kip-ft}$$

$$\text{Flange force} = \frac{678 \times 12}{48.875} = 166 \text{ kips}$$

$$\text{Area required} = \frac{166}{20} = 8.3 \text{ in}^2$$

This area should be split approximately equally to inside and outside plates. Try an outside plate ½ × 12 with area of 6 in^2 and two inside plates ½ × 5½ each with area of 5.5 in^2. Total plate area is 11.5 in^2 > 8.3 in^2—OK. Net width of the outside plate with two holes deducted is $12 - 2 = 10$ in and of the inside plates with two holes deducted is $11 - 2 = 9$ in. The net width of the outside plate along a zigzag section through four holes is $12 - 4 + 2 \times 3^2/(4 \times 3) = 9.5$ in. The net width of the inside plate with four holes is $11 - 4 + 2 \times 3^2/(4 \times 3) = 8.5$ in. The zigzag section with four holes controls. The net area is $0.5(9.5 + 8.5) = 9.0$ in^2. Hence the stress in the plates is $166/9.0 = 18.4$ ksi < 29 ksi—OK.

Since the gross area of the splice plates exceeds that of the flange, there is no need to check fatigue of the plates.

Capacity of the bolts in this slip-critical connection, with class A surfaces and 12 A325 ⅞-in-diameter bolts in standard holes, is computed as follows: Allowable shear stress is $F_u = 15.5$ ksi. The capacity of one bolt thus is $r_v = 15.5 \times 3.14(0.875)^2/4 = 9.32$ kips. Shear capacity of 12 bolts in double shear is $2 \times 12 \times 9.32 = 224$ kips > 166 kips—OK.

The bearing capacity of the bolts, with an allowable stress of $1.1F_u = 1.1 \times 65 = 71.5$ ksi, is $71.5 \times 0.875 \times 0.875 \times 12 = 657$ kips > 166 kips—OK.

5.27 COMPRESSION SPLICES

The requirements for strength of splice given in Art. 5.26 for tension splices apply also to compression splices.

Compression members may be spliced with complete-penetration groove welds. As for tension splices, with such welds, it is desirable that splice plates not be used.

Groove-welded compression splices may be designed for the basic allowable stresses for base metal. Fatigue does not control if the splice will always be in compression.

Groove-welded compression splices should be made with a smooth transition when the offset between surfaces at either side of the joint is greater than the thickness of the thinner part connected. The slope relative to the surface or edge of either part should not exceed 1 :2½ (5:12, or 22°). For smaller offsets, the face of the weld should be sloped 1:2½ from the surface of the thinner part or sloped to the surface of the thicker part if this requires a lesser slope.

At bolted splices, compression members may transmit the load through the splice plates or through bearing.

Example—AASHTO ASD. Investigate the bolted flange splice in the highway-bridge girder of the example in Art. 5.26 for use with the compression flange.

The computations for the bolts are the same for the compression splice as for the tension splice. Use 12 bolts, for sealing, in a staggered pattern (Fig. 5.29).

The splice-plate net area for the tension flange is OK for the compression flange because it is subject to the smaller moment 200 kip-ft rather than 700 kip-ft. The gross area, however, must carry the force due to the 700-kip-ft moment that causes compression in the compression splice. By calculations in Art. 5.26, the splice plates are OK for the maximum compressive stress of 20 ksi and for stress range. Therefore, the splice plates for the compression splice can be the same as those required for tension.

To avoid buckling of the splice plates, the ends of compression members in bolted splices should be in close contact, whether or not the load is transmitted in bearing, unless the splice plates are checked for buckling. For the compression splice, the unsupported length of plate L is 3.75 in, and the effective length is $0.65L = 2.44$ in. The slenderness ratio of the plate then is $2.44\sqrt{12}/0.5 = 16.90$. Hence the allowable compression stress is

$$F_a = 16{,}980 - 0.53(16.90)^2 = 16{,}829 \text{ psi} = 16.8 \text{ ksi}$$

The compressive stress in the inside splice plates due to the loads is $166/11.5 = 14.4$ ksi < 16.8 ksi—OK.

Columns in Buildings. Ends of compression members may be milled to ensure full bearing at a splice. When such splices are fabricated and erected under close inspection, the designer may assume that stress transfer is achieved entirely through bearing. In this case, splice material and fasteners serve principally to hold the connected parts in place. But they also must withstand substantial stresses during erection and before floor framing is placed. Consequently, standard specifications generally require that splice material and fasteners not only be arranged to hold all parts in place but also be proportioned for 50% of the computed stress. The AISC specification, however, exempts building columns from this requirement. But it also requires that all joints with stress transfer through bearing be proportioned to resist any tension that would be developed by specified lateral forces acting in conjunction with 75% of the calculated dead-load stress and no live load.

When fillers are used, the requirements discussed in Art. 5.13 should be satisfied.

In multistory buildings, changes in column sizes divide the framing vertically into tiers. Joints usually are field bolted or welded as successive lengths are erected. For convenience in connecting beam and girder framing, column splices generally are located 2 to 3 ft above floor level. Also, for convenience in handling and erection, columns usually are erected in two- or three-story lengths.

To simplify splicing details while securing full bearing at a change in column size, wide-flange shapes of adjoining tiers should be selected with the distance between the inner faces of the flanges constant, for example, note that the *T* distances given in the AISC "Steel Construction Manuals" for W14 × 43 and heavier W14 sections are all 11 or 11¼ in. If such sections are not used, or if upper and lower members are not centered, full bearing will not be obtained. In such cases, stress transfer may be obtained with filler plates attached to the flanges of the smaller member and finished to bear on the larger member. Enough fasteners must be used to develop in single shear the load transmitted in bearing. Instead of fillers, however, a butt plate may be interposed in the joint to provide bearing for the upper and lower sections (Fig. 5.30). The plate may be attached to either shaft with tack welds or clip angles. Usually, it is connected to the upper member, because a plate atop the lower one may interfere with beam and girder erection. The butt plate should be thick enough to resist the bending and shear stresses imposed by the eccentric loading. Generally, a 1½-in-thick plate can be used when a W8 section is centrally seated over a W10 and a 2-in-thick plate between other sections. Plates of these thicknesses need not be planed as long as they provide satisfactory bearing. Types and sizes of welds are determined by the loads.

For direct load only, on column sections that can be spliced without plates, partial-penetration bevel or J welds may be used. Figure 5.31 illustrates a typical splice with a partial-penetration single-bevel groove weld (AWS prequalified, manual, shielded metal-arc welded joint BTC-P4). Weld sizes used depend on thickness of the column flange. AISC recommends the partial-penetration groove welds in Table 5.17.

A similar detail may be used for splices subjected to bending moments. A full moment splice can be made with complete-penetration groove welds (Fig. 5.32). When the bevel is formed, a shoulder of at least ⅛ in should remain for landing and plumbing the column.

When flange plates are not used, column alignment and stability may be achieved with temporary field-bolted lugs. These usually are removed after the splice is welded, to meet architectural requirements.

To facilitate column erection, holes often are needed to receive the pin of a lifting hitch. When splice plates are used, the holes can be provided in those plates (Fig. 5.34). When columns are spliced by groove welding, without splice plates, however, a hole must be provided in the column web, or auxiliary plates, usually temporary, must be attached for

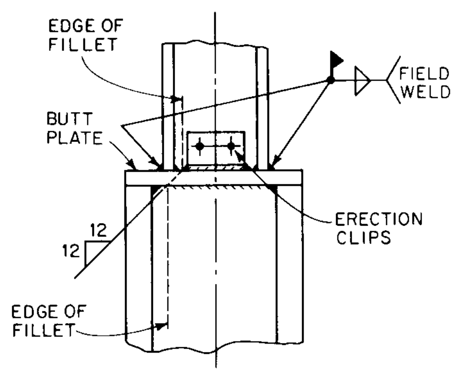


FIGURE 5.30 Column splice with butt plate.

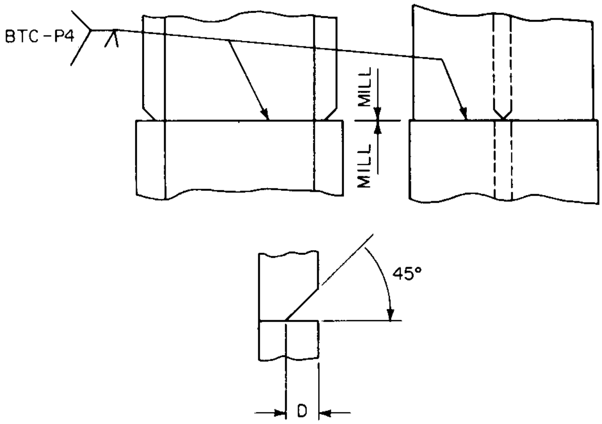


FIGURE 5.31 Column splice with partial-penetration groove welds.

lifting. The lifting-lug detail in Fig. 5.33 takes advantage of the constant distance between inside faces of flanges of W14 sections, and allows for welding the flanges and web of the splice, as shown in Fig. 5.32.

Bolted column splices generally are made with flange plates. Fillers are inserted (Fig. 5.34) when the differences in column sizes are greater than can be absorbed by erection clearance.

To provide erection clearance when columns of the same nominal depth are spliced, a 1⁄16-in filler often is inserted under each splice plate on the lower column. Or with the splice plates shop fastened to the lower column, the fastener holes may be left open on the top line below the milled joint until the upper member is connected. This detail permits the erector to spring the splice plates apart enough for placement of the upper shaft.

For the detail in Fig. 5.34, the usual maximum gage for the flanges should be used for G_1 and G_2 (4, 5½, or 11½ in). The widths of fillers and splice plates are then determined by minimum edge distances (1¼ in for ¾-in fasteners, 1½ in for ⅞-in fasteners, and 1¾-in for 1-in fasteners).

Fill plates not in bearing, as shown in Fig. 5.34, may be connected for shipping, with two fasteners or with welds, to the upper column. Fillers milled to carry load in bearing or thick fills that will carry load should be attached with sufficient fasteners or welds to transmit that load (Art. 5.13).

TABLE 5.17 Recommended Sizes of Partial-Penetration Groove Welds for Column Splices

Thickness of thinner flange, in	Effective weld size, in
Over ½ to ¾ inclusive*	¼
Over ¾ to 1½ inclusive	5⁄16
Over 1½ to 2¼ inclusive	⅜
Over 2¼ to 6 inclusive	½
Over 6	⅝

*Up to ½ in, use splice plates.

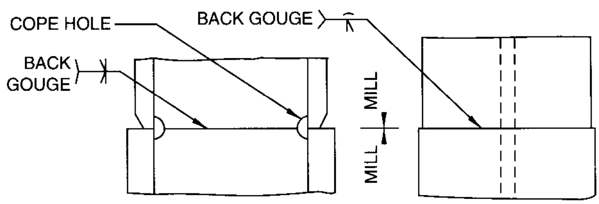


FIGURE 5.32 Column splice with complete-penetration groove welds to resist bending moments.

When fasteners are used for a column splice, it is good practice to space the holes so that the shafts are pulled into bearing during erection. If this is not done, it is possible for fasteners in the upper shaft to carry the entire load until the fasteners deform and the joint slips into bearing.

Usually, for direct load only, thicknesses used for splice plates are as follows:

Column weight lb/ft	Splice-plate thickness, in
Up to 132	3⁄8
Up to 233	1⁄2
Over 233	3⁄4

Four fasteners are used in each half of a 3⁄8-in splice plate and six fasteners in each half of a 1⁄2-in or heavier splice plate.

If, however, the joint carries tension or bending moments, the plate size and number of fasteners must be designed to carry the load. An equivalent amount of weld may be used instead of fasteners.

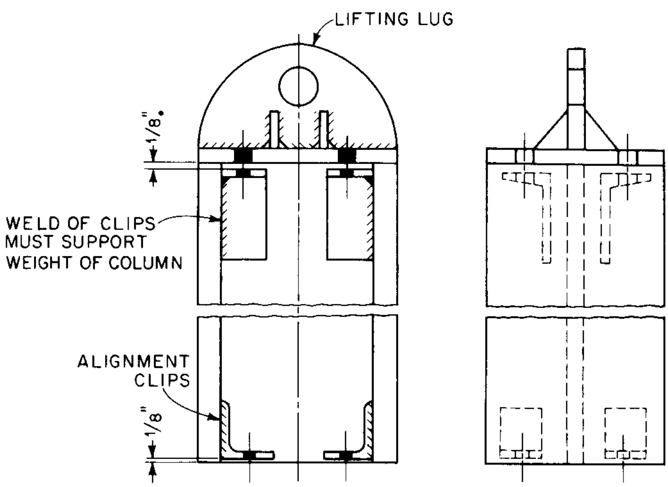


FIGURE 5.33 Lifting lug facilitates column erection.

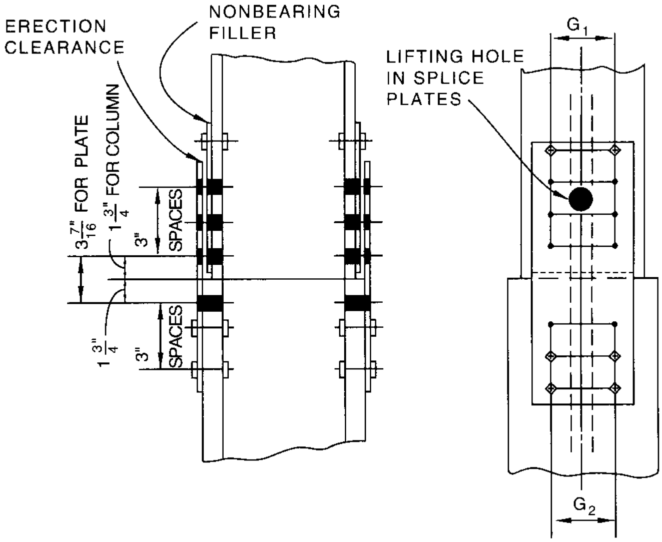


FIGURE 5.34 Column splice with flange plates. A hole in each plate is used for erection of the lower column.

A combination of shop-welded and field-welded connection material is usually required for moment connections. Splice plates are shop welded to the top of the lower shaft and field welded to the bottom of the upper shaft. For field alignment, a web splice plate can be shop bolted or welded to one shaft and field bolted to the other. A single plate nearly full width of the web with two to four fasteners, depending on column size, allows the erector to align the shafts before the flange plates are welded.

5.28 COLUMN BASE PLATES

AISC ASD Approach. The lowest columns of a structure usually are supported on a concrete foundation. To prevent crushing of the concrete, base plates are inserted between the steel and concrete to distribute the load. For very heavy loads, a grillage, often encased in concrete, may be required. It consists of one or more layers of steel beams with pipe separators between them and tie rods through the pipe to prevent separation.

The area (in^2) of base plate required may be computed from

$$A = \frac{P}{F_p} \tag{5.7}$$

where P = load, kips
F_p = allowable bearing pressure on support, ksi

The allowable pressure depends on strength of concrete in the foundation and relative sizes of base plate and concrete support area. If the base plate occupies the full area of the support, $F_p = 0.35f'_c$ where f'_c is the 28-day compressive strength of the concrete. If the base plate covers less than the full area, $F_p = 0.35f'_c\sqrt{A_2/A_1} \leq 0.70f'_c$, where A_1 is the base-plate area ($B \times N$), and A_2 is the full area of the concrete support.

Eccentricity of loading or presence of bending moment at the column base increases the pressure on some parts of the base plate and decreases it on other parts. To compute these effects, the base plate may be assumed completely rigid so that the pressure variation on the concrete is linear.

Plate thickness may be determined by treating projections m and n of the base plate beyond the column as cantilevers. The cantilever dimensions m and n are usually defined as shown in Fig. 5.35. (If the base plate is small, the area of the base plate inside the column profile should be treated as a beam.) Yield-line analysis shows that an equivalent cantilever dimension n' can be defined as $n' = \frac{1}{4}\sqrt{db_f}$, and the required base plate thickness t_p can be calculated from

$$t_p = 2l\sqrt{\frac{f_p}{F_y}}$$

where l = max (m, n, n'), in
$f_p = P/(BN) \leq F_p$, ksi
F_y = yeild strength of base plate, ksi
P = column axial load, kips

For columns subjected only to direct load, the welds of column to base plate, as shown in Fig. 5.35, are required principally for withstanding erection stresses. For columns subjected to uplift, the welds must be proportioned to resist the forces.

Base plates are tied to the concrete foundation with hooked anchor bolts embedded in the concrete. When there is no uplift, anchor bolts commonly used are ¾ in in diameter, about 1 ft 6 in long plus a 3-in hook.

Instead of welds, anchor bolts may be used to tie the column and base plate to the concrete foundation. With anchor bolts up to about 1¼ in in diameter, heavy clip angles may be fastened to the columns to transfer overturning or uplift forces from the column to the anchor bolts. For large uplift forces, stiffeners may be required with the anchor bolts (Fig. 5.36).

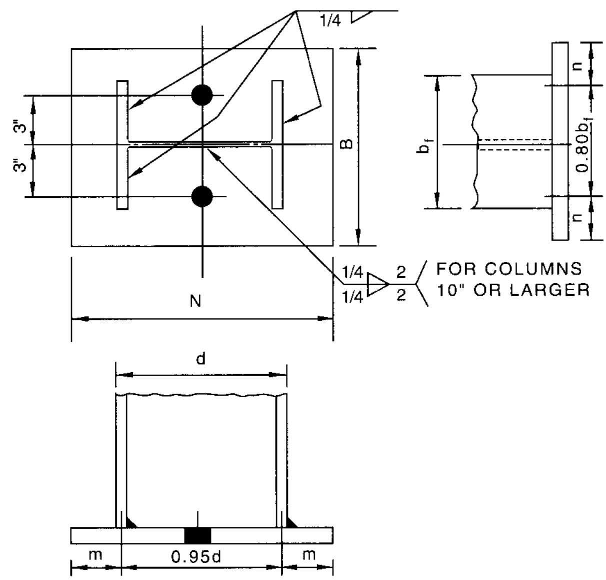


FIGURE 5.35 Column welded to a base plate.

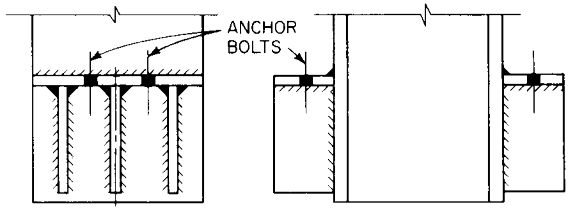


FIGURE 5.36 Column base with stiffeners for anchor bolts.

The load is transferred from column to bolts through the stiffener-plate welds. The cap plate should be designed for bending.

Example—Base Plate with Concentric Column Load, AISC ASD. A W10 × 54 column of A36 steel carries a concentric load of 120 kips. The foundation concrete has a 28-day specified strength $f'_c = 3000$ psi. Design a base plate of A36 steel to occupy the full concrete area.

Since $A_2/A_1 = 1$, the allowable bearing stress on the concrete is $F_p = 0.35 \times 3000 = 1050$ psi $= 1.05$ ksi. The required base-plate area is $120/1.05 = 114$ in^2. Since the column size is $d \times b_f = 10.125 \times 10$, try a base plate 12×12; area $= 144$ in^2. Then, from Fig. 5.35, $m = (12 - 0.95 \times 10.125)/2 = 1.19$ in, $n = (12 - 0.80 \times 10)/2 = 2.00$ in, and $n' = \sqrt{10.125 \times 10}/4 = 2.52$ in. Hence, $f_p = 120/144 = 0.83$ ksi < 1.05 ksi—OK. The plate thickness required, from Eq. (5.8), with $F_y = 36$ ksi, is

$$t_p = 2 \times 2.52\sqrt{0.83/36} = 0.77 \text{ in}$$

Use a ⅞-in-thick plate.

Example—Base Plate for Column with Bending Moment, AISC ASD. A W10 × 54 column of A36 steel carries a concentric load of 120 kips and a bending moment of 29 ft-kips (Fig. 5.37). For the foundation concrete, $f'_c = 3000$ psi. Design a base plate of A36 steel to occupy the full concrete support area.

As a first trial, select a plate 18 in square. Area provided is 324 in^2. The allowable bearing pressure is $0.35f'_c = 1050$ psi.

The bearing pressure due to the concentric load is $120/324 = 0.370$ ksi. The maximum pressures due to the moment are

$$\pm\frac{M}{BN^2/6} = \pm\frac{29 \times 12}{18 \times 18^2/6} = \pm 0.358 \text{ ksi}$$

Hence the maximum total pressure is $0.370 + 0.358 = 0.728$ ksi < 1.050. The minimum pressure is $0.370 - 0.358 = 0.012$ ksi. Thus the plate area is satisfactory.

The projection of the plate beyond the ends of the flanges is

$$n = (18 - 0.80 \times 10)/2 = 5 \text{ in}$$

By Eq. (5.8), the thickness required for this cantilever is

$$t_p = 2 \times 5\sqrt{0.370/36} = 1.01 \text{ in}$$

The projection of the plate in the perpendicular direction (Fig. 5.37) is

$$m = (18 - 0.95 \times 10.125)/2 = 4.19 \text{ in}$$

Over the distance m, the decrease in bearing pressure is $4.19\,(0.728 - 0.012)/18 = 0.142$

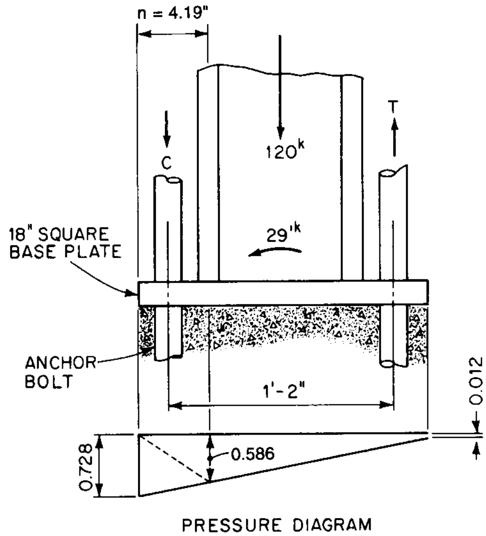


FIGURE 5.37 Example of column base-plate design.

ksi. Therefore, the pressure under the flange is $0.728 - 0.142 = 0.561$ ksi. The projection acts as a cantilever with a trapezoidal load varying from 0.728 to 0.561 ksi. The bending moment in this cantilever at the flange is

$$M = 18\frac{(4.19)^2 0.561}{6} + \frac{(4.19)^2 0.728}{3} = 106 \text{ in-kips}$$

The thickness required to resist this moment is

$$t_p = \sqrt{\frac{6 \times 106}{18 \times 27}} = 1.14 \text{ in}$$

Use a 1¼-in-thick plate.

The overturning moment of 29 ft-kips induces forces T and C in the anchor bolts equal to $29/1.167 = 24.9$ kips. If two bolts are used, the force per bolt is $24.9/2 = 12.5$ kips. Then, with an allowable stress of 20 ksi, each bolt should have a nominal area of

$$12.5/20 = 0.625 \text{ in}^2$$

Use two 1-in-diameter bolts, each with a nominal area of 0.785 in^2.

The circumference of a 1-in bolt is 3.14 in. With an allowable bond stress of 160 psi, length of the bolts should be

$$L = \frac{12{,}500}{160 \times 3.14} = 25 \text{ in}$$

Also, because of the overturning moment, the weld of the base plate to each flange should be capable of resisting a force of $29 \times {}^{12}\!/_{10} = 34.8$ kips. Assume a minimum E70XX weld of $^5\!/_{16}$-in capacity equal to $0.707 \times 21 \times {}^5\!/_{16} = 4.63$ kips per in. The length of weld required then is

$$L = \frac{34.8}{4.63} = 7.5 \text{ in}$$

A weld along the flange width would be more than adequate.

Pressure under Part of Base Plate. The equivalent eccentricity of the moment acting at the base of a column equals the moment divided by the concentric load: $e = M/P$. When the eccentricity exceeds one-sixth the length of base plate (P lies outside the middle third of the plate), part of the plate no longer exerts pressure against the concrete. The pressure diagram thus extends only part of the length of the base plate. If the pressure variation is assumed linear, the pressure diagram is triangular. For moderate eccentricities (one anchor bolt required on each side of the column), the design procedure is similar to that for full pressure over the base plate. For large eccentricities (several anchor bolts required on each side of the column), design may be treated like that of a reinforced-concrete member subjected to bending and axial load.

Example—AISC ASD. A W10 × 54 column of A36 steel carries a concentric load of 120 kips and a bending moment of 45 ft-kips (Fig. 5.38). For the foundation concrete, $f'_c = 3000$ psi. Design a base plate of A36 steel to occupy the full concrete support area.

As a first trial, select a plate 20 in square. The eccentricity of the load then is $45 \times {}^{12}\!/_{120} = 4.50$ in $>$ (${}^{20}\!/_{6} = 3.33$ in). Therefore, there will be pressure over only part of the plate. Assume this length to be d.

For equilibrium, the area of the triangular pressure diagram must equal the 120-kip vertical load:

$$120 = \tfrac{1}{2}Bdf_p = 10df_p \tag{5.9}$$

where B = width of plate = 20 in and f_p = maximum bearing pressure (ksi). The allowable bearing pressure is $0.35f'_c = 0.35 \times 3000 = 1050$ psi $= 1.050$ ksi.

For equilibrium also, the sum of the moments about the edge of the base plate at the point of maximum pressure must be zero:

$$120 \times 10 - 45 \times 12 - \tfrac{1}{6}Bd^2 f_p = 0$$

Substitution of $B = 20$ and rearrangement of terms yields

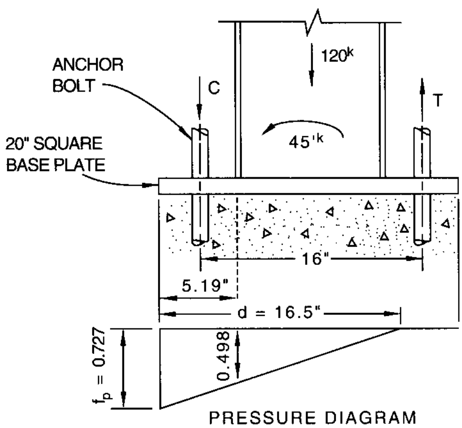


FIGURE 5.38 Loading produces pressure over part of a column base.

$$660 = 3.33d^2 f_p \tag{5.10}$$

Dividing Eq. (5.9) by (5.10) eliminates f_p and solution of the result gives $d = 16.5$ in. For this value of d, Eq. (5.9) gives $f_p = 0.727$ ksi < 1.050. The size of the base plate is satisfactory.

The projection of the plate beyond the flange is $(20 - 0.95 \times 10.125)12 = 5.19$ in. The pressure under the flange is $0.727(16.5 - 5.19)/16.5 = 0.498$ ksi. The projection acts as a cantilever with a trapezoidal load varying from 0.727 to 0.498 ksi. The bending moment in this cantilever at the flange is

$$M = 20\left[\frac{(5.19)^2 0.498}{6} + \frac{(5.19^2 0.727}{3}\right] = 175.3 \text{ in-kips}$$

The thickness required to resist this moment is

$$t = \sqrt{\frac{6 \times 175.3}{20 \times 27}} = 1.40 \text{ in}$$

Use a 1½-in-thick plate.

The overturning moment induces forces T and C in the anchor bolts equal to $45/1.33 = 33.7$ kips. If two bolts are used, the force per bolt is $33.7/2 = 16.85$ kips. Then, with an allowable stress of 20 ksi, each bolt should have a nominal area of $16.85/20 = 0.843$ in^2. Use two 1⅛-in-diameter bolts, each with a nominal area of 0.994 in^2.

The circumference of a 1⅛-in bolt is 3.53 in. With an allowable bond stress of 160 psi, length of the bolts should be

$$L = \frac{16{,}850}{160 \times 3.53} = 30 \text{ in}$$

Assume a minimum weld of 5⁄16 in for connecting the base plate to each flange. The weld has a capacity of 4.63 kips per in. Welds 10 in long on each flange provide a resisting moment of $4.63 \times 10 \times 10 = 463$ in-kips. Since the moment to be resisted is $45 \times 12 = 540$ in-kips, welds along the inside faces of the flanges must resist the 77-in-kip difference. With a smaller moment arm than the outer welds because of the ⅝-in flange thickness, the length of weld per flange required is

$$L = \frac{77}{4.63(10 - 2 \times 0.625)} = 1.9 \text{ in}$$

The outer-face welds should be returned at least 1 in along the inside faces of the flanges on opposite sides of the web.

Setting of Base Plates. The welds of flanges to base plate in the preceding examples usually are made in the shop for small plates. Large base plates often are shipped separately, to be set before the columns are erected. When a column is set in place, the base must be leveled and then grouted. For this purpose, the footing must be finished to the proper elevation. The required smooth bearing area may be obtained with a steel leveling plate about ¼ in thick. It is easy to handle, set to the proper elevation, and level. Oversized holes punched in this plate serve as templates for setting anchor bolts. Alternatively, columns may be set with wedges or shims instead of a leveling plate.

Large base plates may be set to elevation and leveled with shims or with leveling screws (Fig. 5.39). In those cases, the top of concrete should be set about 2 in below the base plate to permit adjustments to be made and grout to be placed under the plate, to ensure full bearing. One or more large holes may be provided in the plate for grouting.

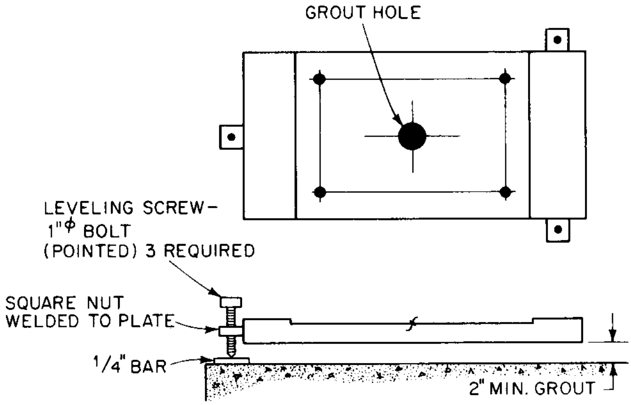


FIGURE 5.39 Large base plate installed with leveling screws.

Planing of Base Plates. Base plates, except thin rolled-steel bearing plates and surfaces embedded in grout, should be planed on all bearing surfaces. Also, all bearing surfaces of rolled-steel bearing plates over 4 in thick should be planed. Rolled-steel bearing plates over 2 in but not more than 4 in thick may be planed or straightened by pressing. Rolled-steel bearing plates 2 in or less in thickness, when used for column bases, and the bottom surfaces of grouted plates need not be planed. Design thickness of a plate is that after milling. So finishing must be taken into account in ordering the plate.

5.29 BEAM BEARING PLATES

AISC ASD Approach. Beams may be supported directly on concrete or masonry if the bearing pressure is within the allowable. The flanges, however, will act as cantilevers, loaded by the bearing pressure f_p (ksi). The maximum bending stress (ksi) may be computed from

$$f_b = \frac{3f_p(B/2 - k)^2}{t^2} \tag{5.11}$$

where B = flange width, in
k = distance from bottom of beam to web toe of fillet, in
t = flange thickness, in

The allowable bending stress in this case is $0.75F_y$, where F_y is the steel yield stress (ksi).

In the absence of building code regulations, the allowable bearing stress F_p (psi) may be taken as 400 for sandstone and limestone, 250 for brick, $0.35f'_c$ for concrete when the full area of the support is covered by the bearing steel, and $0.35f'_c\sqrt{A_2/A_1} \leq 0.70f'_c$ if the base plate covers less than the full area, where f'_c is the 28-day compressive strength of the concrete.

When the bearing pressure under a beam flange exceeds the allowable, a bearing plate should be inserted under the flange to distribute the beam load over the concrete or masonry. The beam load may be assumed to be uniformly distributed to the bearing plate over an area of $2kN$, where k is the distance (in) from bottom of beam to web toe of fillet and N is the length of plate (Fig. 5.40).

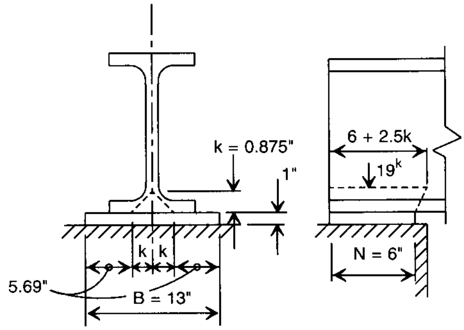


FIGURE 5.40 Beam seated on bearing plate on concrete or masonry support.

Example—AISC ASD. A W12 × 26 beam of A36 steel with an end reaction of 19 kips rests on a brick wall (Fig. 5.40). Length of bearing is limited to 6 in. Design a bearing plate of A36 steel.

With allowable bearing pressure of 0.250 ksi, plate area required is $19/0.250 = 76$ in^2. Since the plate length is limited to 6 in, the plate width should be at least $^{76}/_{6} = 12.7$ in. Use a 6 × 13 in plate, area 78 in^2. The actual bearing pressure then will be $f_p = {}^{19}/_{78} = 0.243$ ksi.

For a W12 × 26 the distance $k = 0.875$ in (see beam dimensions in AISC manual). The plate projection acting as a cantilever then is $l = B/2 - k = {}^{13}/_{2} - 0.875 = 5.62$ in. As for a column base (Art. 5.28), the required thickness can be computed for a bearing pressure of 0.243 ksi from Eq. (5.8):

$$t_p = 2 \times 5.62 \sqrt{0.243/36} = 0.923 \text{ in}$$

Use a 1-in-thick bearing plate.

The beam web must be checked for web yielding over the bearing plate, over a length of $N + 2.5k = 8.188$ in. The allowable bearing stress is $0.66F_y = 23.6$ ksi for A36 steel. For a web thickness of 0.230 in, the web stress due to the 19-kip reaction of the beam is

$$f = \frac{19}{8.188 \times 0.230} = 10.1 \text{ ksi} < 23.6\text{—OK}$$

In addition to web yielding, web crippling must be checked. The allowable web crippling load is given by

$$R_{cp} = 34t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \frac{F_y t_f}{t_w} \tag{5.12}$$

Substitution of plate and beam dimensions, including beam depth $d = 12.22$ in and flange thickness t_f of 0.380 in yields

$$R_{cp} = 34 \times 0.230^2 \left[1 + 3 \left(\frac{6}{12.22} \right) \left(\frac{0.230}{0.380} \right)^{1.5} \right] \sqrt{\frac{36 \times 0.380}{0.230}} = 23.5 \text{ kips}$$

Since 23.5 kips > 19 kips, the beam is OK for avoiding web crippling.

Beams usually are not attached to bearing plates. The plates are shipped separately and grouted in place before the beams are erected. Wall-bearing beams usually are anchored to the masonry or concrete. Government anchors (Fig. 5.41) generally are preferred for this purpose.

5.30 SHEAR SPLICES

In buildings, splices of members subjected principally to shear should develop the strength required by the stresses at point of splice. For groove welds, however, the full strength of the smaller spliced member should be developed.

In highway bridges, shear splices should be designed for the larger of the following: 75% of the strength of the member or the average of the calculated stress at point of splice and the strength of the member there. Splices of rolled flexural members, however, may be designed for the calculated maximum shear multiplied by the ratio of splice design moment to actual moment at the splice.

In railroad bridges, shear splices in main members should have the same strength as the members. Splices in secondary members should develop the average of the strength of the members and the calculated stresses at the splices.

When fillers are used, the requirements discussed in Art. 5.13 should be satisfied.

Shear splices may be made with complete-penetration groove welds, preferably without splice plates. Design rules for groove welds are practically the same as for compression splices (Art. 5.27), though values of allowable stresses are different.

Shear splices are most often used for splicing girder webs. In such applications, splice plates should be symmetrically arranged on opposite sides of the web. For bridges, they should extend the full depth of the girder between flanges. Bolted splices should have at least two rows of fasteners on each side of the joint.

Generally, it is desirable to locate flange and web splices at different sections of a girder. But this is not always practical. Long continuous girders, for example, often require a field splice, which preferably is placed at a section with low bending stresses, such as the dead-load inflection points. In such cases it could be troublesome and costly to separate flange and web splices.

Sometimes, a web splice can be placed where it is required to transmit only shear. Usually, however, a web splice must be designed for both shear and moment. Even in a web splice subjected to pure shear, moment is present if splice plates are used, because of the eccentricity of the shear. For example, as indicated in Fig. 5.42, a group of fasteners on one side of the joint transmits the shear V (kips) from the web to the splice plates. This shear acts through the center of gravity O of the fastener group. On the other side of the joint, a similar

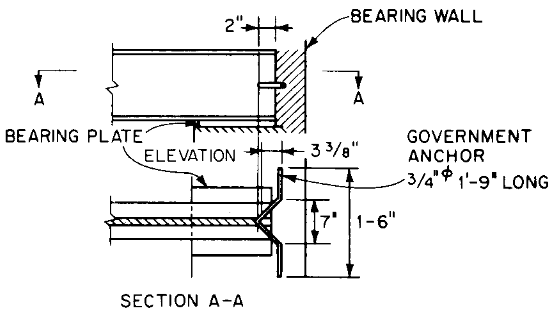


FIGURE 5.41 Wall-bearing beam anchored with government anchor.

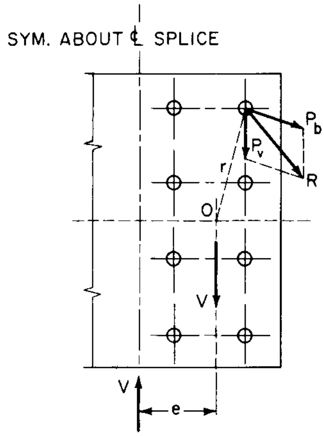


FIGURE 5.42 Bolted splice for a girder web.

group of fasteners transmits the shear from the splice plates to the web on that side. This shear acts through the center of gravity of that fastener group. The two transmitted shears, then, form a couple $2Ve$, where e is half the distance between the shears. This moment must be taken by the fasteners in both groups. In the design of a splice, however, it generally is simpler to work with only one side of the joint. Hence, when the forces on one fastener group are computed, the fasteners should be required to carry the shear V plus a moment Ve. (In a symmetrical splice, e is the distance from O to the splice.)

The classical elastic method assumes that in resisting the moment, each fastener in the group tends to rotate about O. As a consequence:

The reaction P_b of each fastener acts normal to the radius vector from O to the fastener (Fig. 5.42).

The magnitude of P_b is proportional to the distance r from O.

The resisting moment provided by each fastener is proportional to its distance from O.

The applied moment equals the total resisting moment, the sum of the resisting moments of the fasteners in the group,

These consequences result in the relationship

$$M = \frac{P_b J}{c} \qquad P_b = \frac{Mc}{J} \tag{5.13}$$

where M = applied moment, in-kips
P_b = load due to M on outermost fastener in group, kips
J = sum of squares of distances of fasteners in group from center of gravity of group, in^2 (analogous to polar moment of inertia)
c = distance of outermost fastener from center of gravity, in

Hence, when the moment applied to a fastener group is known, the maximum stress in the fasteners can be computed from Eq. (5.13).

This stress has to be added vectorially to the shear on the fastener (Fig. 5.42). The shear (kips) is given by

$$P_v = \frac{V}{n} \tag{5.14}$$

where n is the number of fasteners in group. The resultant stress must be less than the allowable capacity of the fastener.

Depending on the fastener pattern, the largest resultant stress does not necessarily occur in the outermost fastener. Vectorial addition of shear and bending stresses may have to be performed for the most critical fasteners in a group to determine the maximum.

In computing the fastener stresses, designers generally find that the computations are simpler if the forces and distances are resolved into their horizontal and vertical components. Advantage can be taken of the fact that

$$J = I_X + I_y \tag{5.15}$$

where I_y = sum of squares of distances measured horizontally from centrr of gravity to fasteners, in^2

I_x = sum of squares of distances measured vertically from center of gravity to fasteners, in^2

I_x and I_y are analogous to moment of inertia.

Fatigue need not be considered when calculating bolt stresses but should be taken into account in designing splice plates subjected to bending moments.

Example—AASHTO ASD. The 48 × ⁵⁄₁₆ in A36 steel web of a plate girder in a highway bridge is to be field spliced with ⅞-in-diameter A325 bolts (Fig. 5.43). Maximum shear is 95 kips, and maximum bending moments are 700 and − 200 ft-kips. The moment of inertia I_g of the gross section of the girder is 14,670 in^4. Design the web splice for A36 steel.

With an allowable stress of 12 ksi, the shear capacity of the web is

$$V_c = 12 \times 48 \times 5/16 = 180 \text{ kips}$$

One design criterion, then, is $0.75V_c = 0.75 \times 180 = 135$ kips. A second design criterion is the average shear:

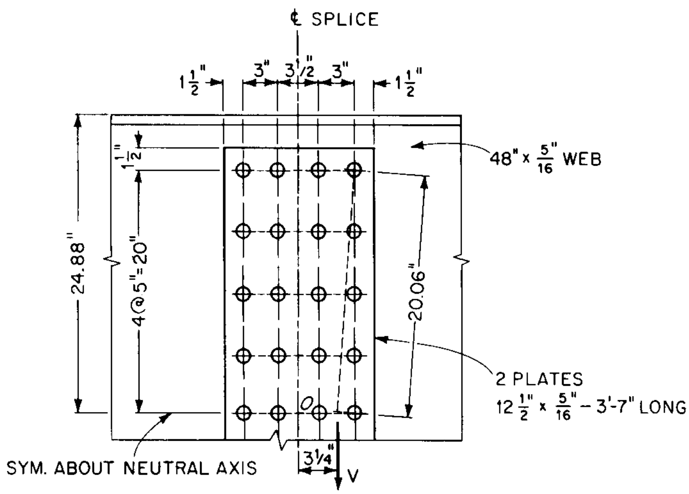


FIGURE 5.43 Example of design of a girder-web splice.

$$V_{av} = \tfrac{1}{2}(95 + 180) = 137.5 \text{ kips} > 135 \text{ kips}$$

This governs for the bolts in the web splice.

The gross moment of inertia of the web is 2,880 in^4. From this should be subtracted the moment of inertia of the holes. With the bolts arranged as shown in Fig. 5.43 and area per hole = 1 × ⁵⁄₁₆ = 0.313 in^2,

$$I_x = 2[(5)^2 + (10)^2 + (15)^2 + (20)^2]0.313 = 1500 \times 0.313 = 470 \text{ in}^4$$

Hence the net moment of inertia of the web is 2880 − 470 = 2410 in^4. Allowable bending stress for the girder is 20 ksi.

The maximum bending stress for a moment of 700 ft-kips and I_g = 14,670 in^4 is, for the gross section,

$$f_b = \frac{700 \times 12 \times 24{,}875}{14{,}670} = 14.2 \text{ ksi} < 20 \text{ ksi—OK}$$

and the average stress for design of the splice is F_b = (14.2 + 20)12 = 17.1 ksi > 0.75 × 20. Hence the girder design moment is 700 × 17.1/14.2 = 843 ft-kips. The net moment carried by the web then is 843 × 2880/14,670 = 166 ft-kips, to which must be added the moment due to the 3.25-in eccentricity of the shear. The total web moment thus is 166 × 12 + 137.5 × 3.25 = 2439 in-kips.

For determination of the maximum stress in the bolts due to this moment, J is needed for use in Eq. (5.13). It is obtained from Eq. (5.15) and bolt-hole calculations:

$$\begin{aligned} I_x &= 4(5^2 + 10^2 + 15^2 + 20^2) &&= 3000 \\ I_y &= 18(1.5)^2 &&= \underline{\quad 41} \\ & && J = 30.41 \text{ in}^2 \end{aligned}$$

By Eq. (5.13) then, the load on the outermost bolt due to moment is

$$P_m = \frac{2{,}439 \times 20.06}{3041} = 1609 \text{ kips}$$

The vertical component of this load is

$$P_v = \frac{1609 \times 1.5}{20.06} = 1.20 \text{ kips}$$

And the horizontal component is

$$P_h = \frac{1609 \times 20}{20.06} = 16.04 \text{ kips}$$

By Eq. (5.14), the load per bolt due to shear is P_v = 137.5/18 = 7.63 kips. Consequently, the total load on the outermost bolt is the resultant

$$R = \sqrt{(1.20 + 7.63)^2 + (16.04)^2} = 18.31 \text{ kips}$$

The allowable capacity of a ⅞-in A325 bolt in double shear is

$$2 \times 0.601 \times 15.5 = 18.6 \text{ kips} > 18.31 \text{ kips}$$

The bolts and bolt arrangement are satisfactory.

For determination of the maximum stress in the splice plates, note that they are 43 in deep and must carry the moment $M_w = 2439$ in-kips. Try two plates, each 5⁄16 in thick. For the gross section, the moment of inertia is

$$I_{gp} = \frac{1}{12} \times 0.3125 \times 43^3 \times 2 = 4141 \text{ in}^4$$

and for the net section,

$$I_{np} = 4141 - 0.3125 \times 1(5^2 + 10^2 + 15^2 + 20^2)4 = 3203 \text{ in}^4$$

The stress on the gross section is

$$f_b = \frac{2439}{4141} \times 215 = 12.7 \text{ ksi} < 20 \text{ ksi—OK}$$

and that on the net is

$$f_b = \frac{2439}{3203} \times 215 = 16.4 \text{ ksi} < 29 \text{ ksi—OK}$$

For the stress range, the moment in the web is $[700 - (-200)]2880/14{,}670 = 177$ ft-kips. The eccentric moment is $[95 - (-2/7 \times 95)]1.75 = 214$ in-kips. Hence the stress range is

$$f_{sr} = \frac{(177 \times 12 + 214)21.5}{4141} = 12.1 \text{ ksi} < 18 \text{ ksi—OK}$$

The assumed web plates are satisfactory.

Alternative Splices. Bolted splices may be made in several different ways when moment is not to be transmitted across the joint. Figure 5.44 shows two examples.

Field-bolted splices generally are more economical than field-welded splices. Groove welds may be more economical, however, if they are made in the shop. Field welding is complicated by the need to control the welding sequence to minimize residual stresses. This often requires that flange-to-web welds on each side of the joint be omitted during fabrication and made after the splice welds. Also, the web interferes with the groove welding of the flange. To avoid the risk of a poor weld, the web must be coped (Fig. 5.45). This provides space for welding, backup bars, cleaning, and other necessary operations.

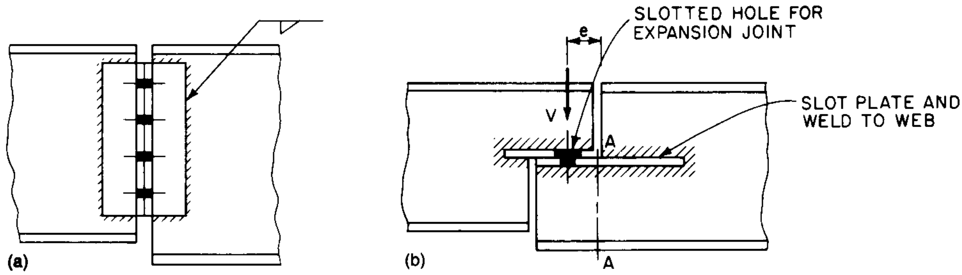


FIGURE 5.44 Girder splice. (*a*) With web plates, shop welds, and field bolts. (*b*) Designed to transmit only shear. Nevertheless, section *A–A* should be designed to resist bending moment *Ve* due to shear.

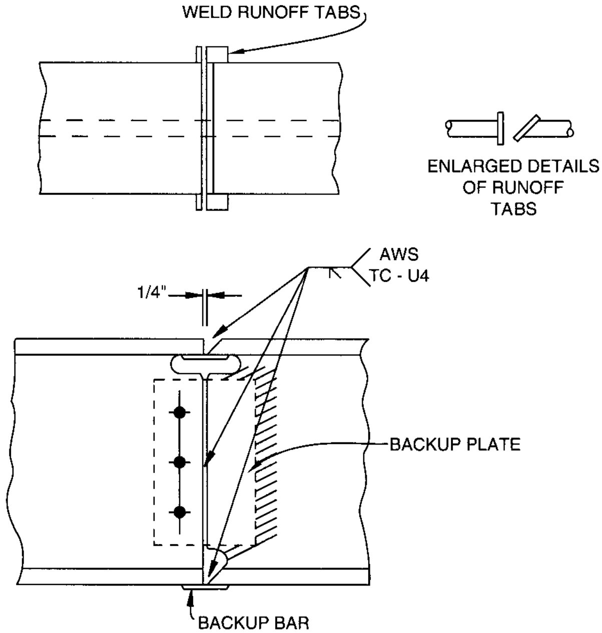


FIGURE 5.45 Welded girder splice. Web plate serves both as an erection connection and as backup for the web groove weld.

5.31 BRACKET CONNECTIONS

Brackets are projections that carry loads. The connection of a bracket to a support has to transmit both shear and moment. Fasteners or welds may be used for the purpose. Connections may be made with fasteners or welds subjected only to shear or to combined shear and tension.

Figure 5.46 shows a plate bracket connected with bolts in single shear. Design of this type of connection is similar to that of the web splice in Fig. 5.42. Each fastener is subjected to a shear load and a load due to moment. The load due to moment on any fastener acts normal to the radius vector from the center of gravity O of the fastener group to the fastener. The maximum load due to moment is given by Eq. (5.13). The resultant of this load and the shear load must be less than the allowable capacity of the fastener. Vectorial addition of the loads due to bending and shear must be performed for the critical fasteners to determine the maximum resultant.

As for a web splice, computations usually are simpler if distances and forces are resolved into horizontal and vertical components, and if J for Eq. (5.13) is obtained from Eq. (5.15).

Example—Bolted Bracket—AISC LRFD. Investigate the bracket connection in Fig. 5.46. The A36 steel bracket is to be connected with ⅞-in-diameter A325N bolts to a building column. The bracket carries a 48-kip factored load 15 in from the center of the column web.

The moment arm is 15 in. The moment on the connection is $48 \times 15 = 720$ in-kips.

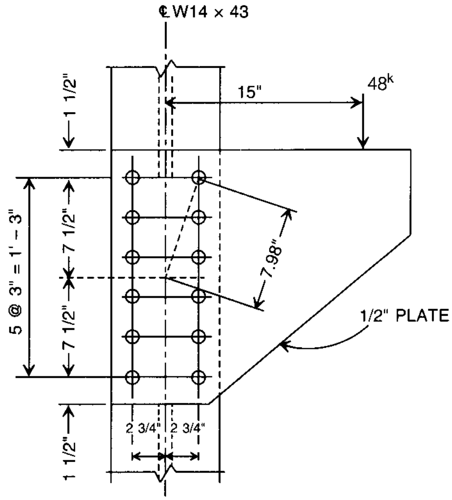


FIGURE 5.46 Bracket with bolts in single shear.

Elastic Method. J is obtained from Eq. (5.15):

$$I_x = 4[(1.5)^2 + (4.5)^2 + (7.5)^2] = 315$$

$$I_y = 2 \times 6(2.75)^2 = \underline{\ 91\ }$$

$$J = 406 \text{ in}^2$$

By Eq. (5.13), the load on the outermost bolts due to moment is

$$P_m = \frac{720 \times 7.98}{406} = 14.2 \text{ kips}$$

The vertical component of this load is

$$P_v = \frac{14.2 \times 2.75}{7.98} = 4.89 \text{ kips}$$

And the horizontal component is

$$P_h = \frac{14.2 \times 7.50}{7.98} = 13.3 \text{ kips}$$

By Eq. (5.14), the load per bolt due to shear is $48/12 = 4.00$ kips. Consequently, the total load on the outermost bolt is the resultant:

$$R = \sqrt{(4.89 + 4.00)^2 + (13.3)^2} = 16.0 \text{ kips}$$

The design strength of a ⅞ in bolt is $0.75 \times 0.6013 \times 48 = 21.6$ kips > 16.0 kips. The connection is satisfactory.

An ultimate-strength method that gives an accurate estimate of the strength of eccentrically loaded bolt groups may be used instead of the preceding procedure. The method assumes that fastener groups rotate about an instantaneous center. (This point coincides with the centroid of a group only when the moment arm l becomes very large.) Tables in the AISC ASD and LRFD manuals are based on this method.

Inelastic Method. Refer to Table 8-20 of the AISC LRFD Manual, Vol. II, Connections. With the moment arm $e_x = 15$ in, bolt spacing $s = 3$ in, and number of bolts per row $n = 6$, interpolate between $e_x = 14$ in and $e_x = 16$ in to find the coefficient $C = (3.99 + 3.55)/2 = 3.77$. The connection strength then is $\phi R = 3.77 \times 21.6 = 81.4$ kips > 48 kips, and is OK. The coefficient C indicates that, because of the eccentric loading, the 12 bolts of the connection are only as effective as 3.77 bolts in direct (concentric) shear. To compare methods, an equivalent C for the elastic method could be calculated as the ratio of the applied load to the resultant force on the outermost bolt, $48/16 = 3.00$. Thus, for this example, the inelastic method gives a capacity $3.77/3.00 = 1.26$ times that of the elastic method.

The plate should be checked for bending and shear. For the gross section, required bending strength is

$$f_b = \frac{48(15 - 2.75 - 0.5)}{0.5(18)^2/4} = 13.9 \text{ ksi}$$

The strength of the gross section is

$$F_b = 0.9 \times 36 = 32.4 \text{ ksi} > 13.9 \text{ ksi}$$

The plate is satisfactory for bending (yielding) of the gross section. For bending of the net section, from the AISC LRFD manual, the section modulus is $S_{net} = 18 \text{ in}^3$ and the required strength is

$$f_b = \frac{48(15 - 2.75)}{18} = 32.7 \text{ ksi}$$

Fracture, rather than yielding, however, is the limit state for the net section. The fracture strength of a net section in tension is $0.75F_uA_e$, where F_u is the specified tensile strength and A_e is the effective net area. Since bending induces a tensile stress over the top half of the bracket, and because the yield limit on the gross section has already been checked, it is reasonable to assume that the fracture (rupture) design strength of the bracket net section is $0.75 \times 58 = 43.5$ ksi > 32.7 ksi. The bracket is satisfactory for fracture of the net section.

The shear on the gross section is

$$f_v = \frac{48}{0.5 \times 18} = 5.33 < (0.9 \times 0.6 \times 36 = 19.4 \text{ ksi})\text{—OK}$$

The shear on the net section is

$$f_v = \frac{48}{0.5(18 - 6 \times 0.9375)} = 8.0 < (0.75 \times 0.6 \times 58 = 26.1 \text{ ksi})\text{—OK}$$

The plate is satisfactory for shear.

A plate bracket such as the one in Fig. 5.46 also can be connected to a support with fillet welds in shear. Design of the welds for such a connection can be performed by the classical elastic method, which is analogous to that for fasteners. For example, for the bracket in Fig. 5.47, the shear due to the 48-kip factored load induces a shear (kips per in) equal to the load divided by the total length of weld (weld A + weld B + weld C). The moment due to the load tends to rotate the welds about their center of gravity O. As a consequence:

The force at any point of a weld acts normal to the radius vector from O to the point. In Fig. 5.47, P_b, the force due to the moment of the 48-kip load about O, is normal to the radius vector OB.

The magnitude of P_b (kips per in) is proportional to the distance r from O.

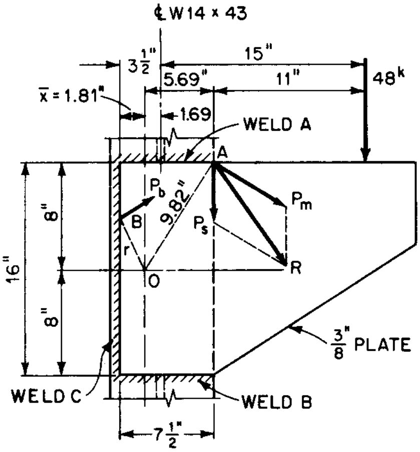


FIGURE 5.47 Bracket with fillet welds in shear.

The resisting moment per inch of weld is proportional to the square of the distance from O.

The applied moment equals the total resisting moment, the sum of the resisting moments of all the welds in the group.

These consequences result in the relationship

$$P_m = \frac{Mc}{J} \quad \text{or} \quad M = \frac{P_m J}{c} \tag{5.16}$$

where M = applied moment, in-kips
P_m = load due to M on the point of a weld most distant from the center of gravity of the weld group, kips per in
J = sum of squares of distances of unit weld lengths from center of gravity of group, in^3 (analogous to polar moment of inertia)
c = distance of outermost point from center of gravity, in

Hence, when the moment applied to a weld group is known, the maximum stress in the welds can be computed from Eq. (5.16). This stress has to be added vectorially to the shear on the weld. The resultant stress must be less than the allowable capacity of the weld.

Depending on the weld pattern, the largest resultant stress does not necessarily occur at the outermost point of the weld group. Vectorial addition of shear and bending stresses may have to be performed for the most critical points in a group to determine the maximum.

Computation of weld stresses generally is simplified if the forces and distances are resolved into their horizontal and vertical components. Advantage can be taken of the fact that

$$J = I_x + I_y \tag{5.17}$$

where I_y = sum of squares of distances measured horizontally from center of gravity of weld group to unit lengths of welds, in^3
I_x = sum of squares of distances measured vertically from center of gravity of weld group to unit lengths of welds, in^3

I_x and I_y are analogous to moment of inertia.

Example—Welded Bracket Connection—AISC LRFD. Investigate the bracket connection in Fig. 5.47. The A36 steel bracket is to be connected with fillet welds made with E70XX electrodes to a building column. The bracket carries a 48-kip factored load 15 in from the center of the column web.

Elastic Method. Because of symmetry, the center of gravity$O$ of the weld group is located vertically halfway between top and bottom of the 16-in-deep plate. The horizontal location of O relative to the vertical weld is obtained by dividing the moments of the weld lengths about the vertical weld by the total length of welds:

$$\bar{x} = \frac{2 \times 7.5 \times 7.5/2}{2 \times 7.5 + 16} = \frac{56.2}{31} = 1.81 \text{ in}$$

J is obtained from Eq. (5.17):

$$\begin{array}{llr} \text{Welds } A \text{ and } B\text{: } 2 \times 7.5(8)^2 & = & 961 \\ \text{Weld } C\text{: } \quad \dfrac{(16)^3}{12} & = & \underline{341} \\ \qquad I_x & = & 1302 \text{ in}^3 \end{array} \qquad \begin{array}{llr} \text{Welds } A \text{ and } B\text{: } \dfrac{2(7.5)^3}{12} & = & 70 \\ 2 \times 7.5\left(\dfrac{7.5}{2} - 1.81\right)^2 & = & 56 \\ \text{Weld } C\text{: } 16(1.81)^2 & = & \underline{53} \\ I_y & = & 179 \text{ in}^3 \end{array}$$

$$J = 1301 + 179 = 1480 \text{ in}^3$$

By Eq. (5.16), the stress on the most distant point A in the weld group due to moment is

$$P_m = \frac{48(15 + 1.69)9.82}{1480} = 5.32 \text{ kips per in}$$

The vertical component of this load is

$$P_v = \frac{5.32 \times 5.69}{9.82} = 3.08 \text{ kips per in}$$

And the horizontal component is

$$P_h = \frac{5.32 \times 8}{9.82} = 4.33 \text{ kips per in}$$

The shear load on the welds is $48/(2 \times 7.5 + 16) = 1.55$ kips per in. Consequently, the total load on the outermost point is the resultant is

$$R = \sqrt{(1.55 + 3.08)^2 + (4.33)^2} = 6.34 \text{ kips per in}$$

For a design stress of $0.75 \times 0.60 \times 70 = 31.5$ ksi, the weld size required is

$$D = \frac{6.34}{0.707 \times 31.5} = 0.285 \text{ in}$$

Use a 5⁄16-in weld.

Instead of the elastic method, the following inelastic method based on the instantaneous center of rotation can be used. The tables for eccentrically loaded weld groups in the AISC manuals—ASD and LRFD—are based on this method.

Inelastic Method. From Fig. 5.47 and the AISC LRFD Manual, Vol. II, $l = 16$ in, $kl = 7.5$ in, and $al = 11 + 7.5 - 1.81 = 16.69$ in, from which $a = 16.69/16 = 1.04$ and $k = 7.5/16 = 0.469$. By interpolation in Table 8-42, coefficient $C = 1.177$. Hence the required weld size in number of sixteenths of an inch is

$$D = \frac{48}{1.177 \times 16} = 2.55$$

Use a $\frac{3}{16}$-in fillet weld. (In comparison, the elastic analysis requires a $\frac{5}{16}$-in fillet weld.)

For a $\frac{5}{16}$-in fillet weld, the minimum plate thickness is $\frac{3}{8}$ in, to permit inspection of the weld size. Try a $\frac{5}{16}$-in-thick plate and check for shear and bending. For shear,

$$f_v = 48/(0.375 \times 16) = 8.00 \text{ ksi} < 19.4 \text{ ksi—OK}$$

For bending,

$$f_b = \frac{48 \times 11}{0.375(16)^2/4} = 22.0 \text{ ksi} < 32.4 \text{ ksi—OK}$$

In the above, $\phi_v F_n = 0.90 \times 0.60 \times 36 = 19.4$ ksi and $\phi_b\, F_b = 0.90 \times 36 = 32.4$ ksi.

Fasteners in Shear and Tension. Bolted brackets also may be attached to supports with the fasteners subjected to combined shear and tension. For the bracket in Fig. 5.48, for example, each bolt is subjected to a shear (kips) of

$$P_v = \frac{P}{n} \tag{5.18}$$

where P = load, kips, on the bracket
n = total number of fasteners

In addition, the moment is resisted by the upper fasteners in tension and the pressure of the lower part of the bracket against the support. The neutral axis is usually located above the bottom of the connection by about $\frac{1}{6}$ of the connection length, but its exact location requires a trial-and-error approach. However, an alternative is to use a conservative plastic distribution, which is valid for both ASD and LRFD design methods. For this distribution, each bolt

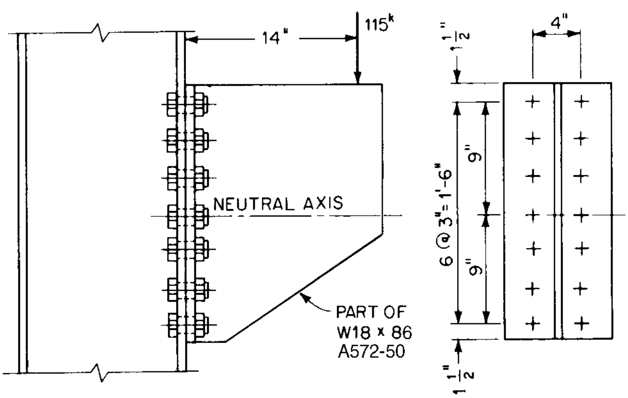


FIGURE 5.48 Bracket with bolts in combined shear and tension.

above the neutral axis is assumed to carry an axial tensile force, T, and each bolt below the neutral axis an axial compressive force $-T$. Thus, the neutral axis falls at the centroid of the bolt group. There is no axial force on the bolts located at the neutral axis.

In bearing-type connections, allowable stresses or design strengths are determined from interaction equations for tension and shear. However, in slip-critical connections, since the shear load is carried by friction at the faying surface, the reduction in friction resistance above the neutral axis of the bolt group (due to the tensile force from bending) is compensated for by an increase in friction resistance below the neutral axis (due to the compressive force from bending). Thus an interaction equation is not required in this case, but both the shear and tensile stresses must be less than those allowable. Also, since slip is a serviceability limit state, the strength-limit state of bearing also must be checked. In addition, the tension forces on the fasteners and the bending of the flanges must be checked for prying (Sec. 5.25.3).

Example—Bracket with Bolts in Tension and Shear—AISC LRFD. Investigate the slip-critical connection in Fig. 5.48. The A36 steel bracket is to be connected with ⅞-in-diameter A325 bolts to a flange of a building column. The bracket carries a 115-kip factored load 14 in from the flange.

First consider the connections as **slip-critical** with A325SC-A-N ⅞-in diameter bolts in standard $^{15}/_{16}$ in holes. The bolt notation indicates slip-critical (*SC*), surface class (*A*), with threads not excluded from the shear planes (*N*). The design strength in shear in this case is

$$\phi r_v = \phi 1.13 \mu T_n = 1.0 \times 1.13 \times 0.33 \times 39 = 14.5 \text{ kips}$$

The design strength for tension is

$$\phi rt = \phi F_t A_b = 0.75 \times 90 \times 0.6013 = 40.6 \text{ kips}$$

The required design strength per bolt in shear is simply $V = 115/14 = 8.21$ kips < 14.5 kips, OK. For the required design strength in tension, take moments about the neutral axis with a force T in each bolt: $2T(3 + 6 + 9) = 115 \times 14$. Solve for $T = 22.4$ kips < 40.6 kips, OK. The bolts are satisfactory for a slip-critical connection.

Next consider as a **bearing** type connection with A325N ⅞-in diameter bolts in standard $^{15}/_{16}$ in holes. The design strength in shear in this case is

$$\phi r_v = \phi F_v A_b = 0.75 \times 48 \times 0.6013 = 21.6 \text{ kips}$$

$V = 8.21$ kips < 21.6 kips, OK. The design strength in pure tension is the same as for the slip-critical condition, 40.6 kips. The design tensile strength in the presence of shear is calculated from the following interaction equation:

$$\phi B = \phi A_b[117 - 2.5(V/A_b)] \leq \phi A_b(90) \tag{5.19}$$

where ϕ = resistance factor, 0.75
A_b = area of bolt, in^2
V = shear force per bolt, kips

For this case, $\phi B = 0.75 \times 0.6013[117 - 2.5\,(8.21/0.6013)] = 37.3$ kips, and $\phi A_b(90) = 0.75 \times 0.6013 \times 90 = 40.6$ kips. Therefore, $\phi B = 37.3$ kips > 22.4 kips required, OK. The bolts are satisfactory for a bearing type connection. This type of connection would be used unless there was a specific requirement for a slip-critical connection.

Next, check bolts and bracket flange for bending and prying action using the notation from Art. 5.25.3:

$$b = (4 - 0.480)/2 = 1.76 \text{ in}$$

$$a = (11.090 - 4)/2 = 3.545 \text{ in, but } a \leq 1.25b = 1.25 \times 1.76 = 2.20 \text{ in; } a = 2.20 \text{ in}$$

$$b' = b - d/2 = 1.76 - 0.875/2 = 1.32 \text{ in}$$

$$a' = a + d/2 = 2.20 + 0.875/2 = 2.64 \text{ in}$$

$$\rho = b'/a' = 1.32/2.64 = 0.500$$

$$\delta = 1 - d'/p = 1 - 0.9375/3 = 0.69$$

A check shows that the W18 × 36 bracket would be unsatisfactory if of A36 steel. Try A572–50 steel.

$$t_c = \sqrt{\frac{4.44\phi Bb'}{pF_y}} = \sqrt{\frac{4.44 \times 40.6 \times 1.32}{3 \times 50}} = 1.260 \text{ in}$$

$$\alpha' = \frac{1}{\delta(1 + \rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] = \frac{1}{0.69 \times 1.500}\left[\left(\frac{1.260}{0.770}\right)^2 - 1\right] = 1.62$$

Because $\alpha' > 1$, use $\alpha' = 1$ to calculate the design strength, ϕT.

$$\phi T = \phi B\left(\frac{t}{t_c}\right)^2 (1 + \delta\alpha') = 40.6\left(\frac{0.770}{1.260}\right)^2 (1.69) = 25.6 \text{ kips} > 22.4 \text{ kips}$$

Bolts and bracket flange are OK for bending and prying action.

Check web of W18 × 86 bracket for limit state of shear on gross section:

$$\phi_s R_{gv} = \phi_s 0.60 F_y A_g = 0.90 \times 0.60 \times 50 \times 0.480 \times 21 = 272 \text{ kips} > 115 \text{ kips, OK.}$$

Check flange of W18 × 86 bracket for limit state of shear on net section:

$$\phi_s R_{nv} = \phi_s 0.60 F_u A_n = 0.75 \times 0.60 \times 65 \times 0.770 \times (21 - 7 \times 1.00) \times 2 = 630 \text{ kips} > 115 \text{ kips, OK.}$$

Check web of W18 × 86 bracket for bending limit state:

$$\phi_b M_n = \phi_b M_p = 0.90 \times 0.480 \times (21)^2 \times (1/4) \times 50 = 2381 \text{ in-kips} > 115 \times 14 = 1610 \text{ in-kips, OK.}$$

Check flange of W18 × 86 bracket for bearing limit state:

$$\phi R_n = \phi 2.4 dtF_u = 0.75 \times 2.4 \times 0.875 \times 0.770 \times 65 \times 14 \text{ bolts} = 1104 \text{ kips} > 115 \text{ kips OK}$$

For the column flange, a similar check shows that bearing is OK. Bending in the column flange must also be considered. In this case, the A572-50 column flange is assumed to be 0.770 in. or thicker, and therefore, is considered OK without calculations. See Art. 5.36 for applicable method for thinner flanges where calculations are required.

Finally, the stability of the column web should be considered. First, if the loading conditions are such that the load can move out of the plane of the bracket web, a top stiffener plate should be used to prevent such motion. Then use the procedure of Salmon and Johnson to assess the web stability under the compressive stresses. In some cases, if the web is too slender, a thicker web or an edge stiffener may be required. (C. G. Salmon and J. E. Johnson, *Steel Structure—Design and Behavior,* Harper Collins, New York.)

Welds in Tension and Shear. Brackets also can be connected to supports with fillet welds in combined shear and tension. For the bracket in Fig. 5.49, for example, the welds carry a shear (kips per in) of

$$P_v = \frac{P}{L} \tag{5.20}$$

where P = load on bracket, kips
L = total length of welds, in

In addition, the moment is resisted by the upper portion of the welds in tension and the pressure of the lower part of the bracket against the support. Both the elastic and inelastic methods assume that the weld carries all the load (bearing pressure between the bracket and the lower part of the support is neglected). The neutral axis is taken at the center of gravity of the weld group. The intensity of load at any point in the weld is proportional to the distance from the neutral axis, and the resisting moment per inch is proportional to that distance. For the elastic method, tension (kips per in) at the most heavily stressed point of the welds may be computed from

$$P_m = \frac{M}{S} \tag{5.21}$$

where M = moment on weld groups, in-kips
S = section modulus of weld group about its neutral axis, in^3

Example—Brackets with Welds in Tension and Shear—AISC LRFD. An A36 steel plate bracket is to be connected with fillet welds on two sides of the plate to a building column (Fig. 5.49). The bracket carries a 115-kip factored load 14 in from the column flange. The welds are to be made with E70XX electrodes.

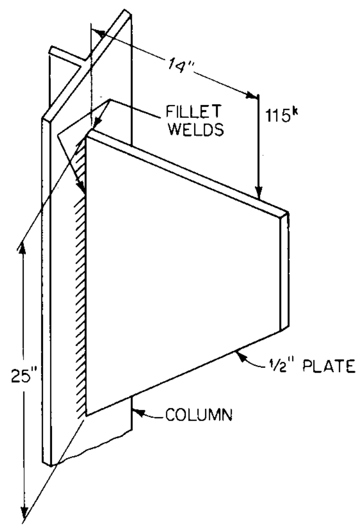


FIGURE 5.49 Bracket with welds in combined shear and tension.

Elastic Method. Assume a plate ½ in thick. With design stress of $0.90 \times 36 = 32.4$ ksi, the length of plate at the support should be at least

$$L = \sqrt{\frac{115 \times 14 \times 6}{0.5 \times 32.4}} = 24.4 \text{ in}$$

To keep down the size of welds, use a plate 25 × ½ in.

By Eq. (5.20), the shear on the two welds is

$$P_v = \frac{115}{2 \times 25} = 2.30 \text{ kips per in}$$

By Eq. (5.21), the maximum tensile stress in the welds is

$$P_m = \frac{115 \times 14}{2(25)^2/6} = 7.73 \text{ kips per in}$$

The resultant of the shear and tensile stresses is

$$R = \sqrt{(2.30)^2 + (7.73)^2} = 8.06 \text{ kips per in}$$

With design stress of $0.75 \times 0.60 \times 70 = 31.5$ ksi, the weld size required is

$$D = \frac{8.06}{0.707 \times 31.5} = 0.362 \text{ in}$$

Use ⅜-in fillet welds.

The plate should be checked for shear and bending as well as stability. For shear, the stress is

$$f_v = 115/(0.5 \times 25) = 9.2 \text{ ksi} < 0.9 \times 0.6 \times 36 = 19.4 \text{ ksi—OK}$$

For bending, the stress is

$$f_b = \frac{115 \times 4}{0.5(25)^2/6} = 30.9 \text{ ksi} < 32.4 \text{ ksi—OK}$$

For the stability check, see the preceding example discussion on that subject.

Inelastic Method. Determine the required weld size for the bracket in Fig. 5.49 by the inelastic method.

From the AISC LRFD manual Vol. II Table 8-38 (special case), with $k = 0$, $l = 25$, and $al = 14$, from which $a = 0.56$, the coefficient C is determined by interpolation to be 1.59. Thus the weld size required, in number of sixteenths of an inch, is $D = 115/(1.59 \times 25) = 2.89$. Use $^3/_{16}$-in fillet welds.

Comparing the results for the two calculation procedures, the elastic method was too conservative and resulted in a weld size twice that for the inelastic method. Another option is to use a simple plastic method, similar to that used for the bolts in Fig. 5.48. This method, which does not rely on the AISC tables for eccentrically loaded welds, provides more reasonable results than the elastic method.

Plastic Method. In Eq. 5.21, replace the elastic section modulus S with the plastic section modulus, Z. Then, proceeding as before, make the following calculations.

$$P_v = \frac{P}{L} = \frac{115}{2 \times 25} = 2.30 \text{ kips per in}$$

$$P_m = \frac{M}{Z} = \frac{115 \times 4}{2(25)^2/4} = 5.15 \text{ kips per in}$$

$$R = \sqrt{(2.30)^2 + (5.15)^2} = 5.64 \text{ kips per in}$$

$$D = \frac{5.64}{0.707 \times 31.5} = 0.253 \text{ kips per in}$$

This indicates the use of a ¼-in fillet weld, assuming an overstress of 1% is acceptable. Thus, the plastic method is conservative compared to the inelastic method (3⁄16-in fillet weld) but more reasonable than the elastic method (⅜-in fillet weld).

5.32 CONNECTIONS FOR SIMPLE BEAMS

End connections of beams to their supports are classified as simple-beam, fully restrained, and partially restrained connections.

Simple, or **conventional, connections** are assumed free to rotate under loads. They are designed to carry shear only. The AISC specifications for structural steel buildings require that connections of this type have adequate inelastic rotation capacity to avoid overstressing the fasteners or welds.

Fully restrained **(rigid-frame) connections,** transmitting bending moment as well as shear, are used to provide complete continuity in a frame (Art. 5.33).

Partially restrained **(semirigid) connections** provide end restraint intermediate between the rigid and flexible types (Art. 5.33).

For simple connections, design drawings should give the end reactions for each beam. If no information is provided, the detailer may design the connections for one-half the maximum allowable total uniform load on each beam.

Simple connections are of two basic types: framed (Fig. 5.50*a*) and seated (Fig. 5.50*b*). A **framed connection** transfers the load from a beam to a support through one or two connection angles, or a shear plate attached to the supporting member, or a tee attached to either the supporting or supported member. A **seated connection** transfers the load through

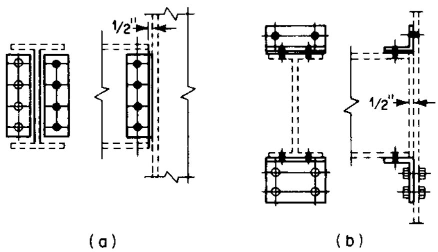


FIGURE 5.50 Simple-beam connections. (*a*) Framed. (*b*) Seated.

a seat under the beam bottom flange. A top, or cap, angle should be used with seated connections to provide lateral support. It may be attached to the beam top flange, as shown in Fig. 5.50*b*, or to the top portion of the web. With both framed and seated connections, the beam end is stopped ½ in short of the face of the supporting member, to allow for inaccuracies in beam length.

5.32.1 Framed Connections

These generally are more economical of material than seated connections. For example, in a symmetrical, bolted, framed connection, the fasteners through the web are in double shear. In a seated connection, the fasteners are in single shear. Hence framed connections are used where erection clearances permit, e.g., for connections to column flanges or to girders with flanges at the same level as the beam flanges. Seated connections, however, usually are more advantageous for connections to column webs because placement of beams between column flanges is easier. Seats also are useful in erection because they provide support for beams while field holes are aligned and fasteners are installed. Furthermore, seated connections may be more economical for deep beams. They require fewer field bolts, though the total number of shop and field fasteners may be larger than those required for a framed connection of the same capacity.

The AISC manual lists capacities and required design checks for beam connections for buildings. Design is facilitated when this information can be used. For cases where such connections are not suitable, beam connections can be designed by the principles and methods given for brackets in Art. 5.31.

Vertical fastener spacing in framed connections is standardized at 3 in. The top gage line also is set 3 in below the beam top, when practicable Closer spacing may be used however, as long as AISC specification restrictions on minimum spacing are met.

To ensure adequate stiffness and stability, the length of the connection material in a framed connection should be at least half the distance *T* between flange-web fillets of the beam.

Distance between inner gage lines of outstanding legs or flange of connection material is standardized at 5½ in, but sometimes a shorter spacing is required to meet AISC specification requirements for minimum edge distance.

Thickness of connection material may be determined by shear on a vertical section, availability of material of needed thicknesses, or the bearing value for the nominal fastener diameter.

When a beam frames into a girder with tops of both at the same level, the top of the beam generally is notched, or coped, to remove enough of the flange and web to clear the girder flange. Depth of cut should be sufficient to clear the web fillet (*k* distance for a rolled section). Length of cope should be sufficient to clear the girder flange by ½ to ¾ in. A fillet with smooth transition should be provided at the intersection of the horizontal and vertical cuts forming the cope.

For beams framing into column flanges, most fabricators prefer connections attached to the columns in the shop. Then the beams require punching only. Thus less handling and fewer operations are required in the shop. Furthermore, with connection material attached to the columns, erectors have more flexibility in plumbing the steel before field bolts are tightened or field welds made.

Some of the standardized framed connections in the AISC manual are arranged to permit substitution of welds for bolts. For example, welds *A* in Fig. 5.51*a* replace bolts for the web connections. Welds *B* replace bolts in the outstanding legs (Fig. 5.51*b*). Angle thickness must be at least the weld size plus ¹⁄₁₆ in and a minimum of ⁵⁄₁₆. Holes may be provided for erection bolts in legs that are to be field welded. When bolts are used in outstanding legs, the bearing capacity of supporting material should be investigated.

Welds *A* are eccentrically loaded. They receive the load from the beam web and the connection transmits the load to the support at the back of the outstanding legs. Hence there

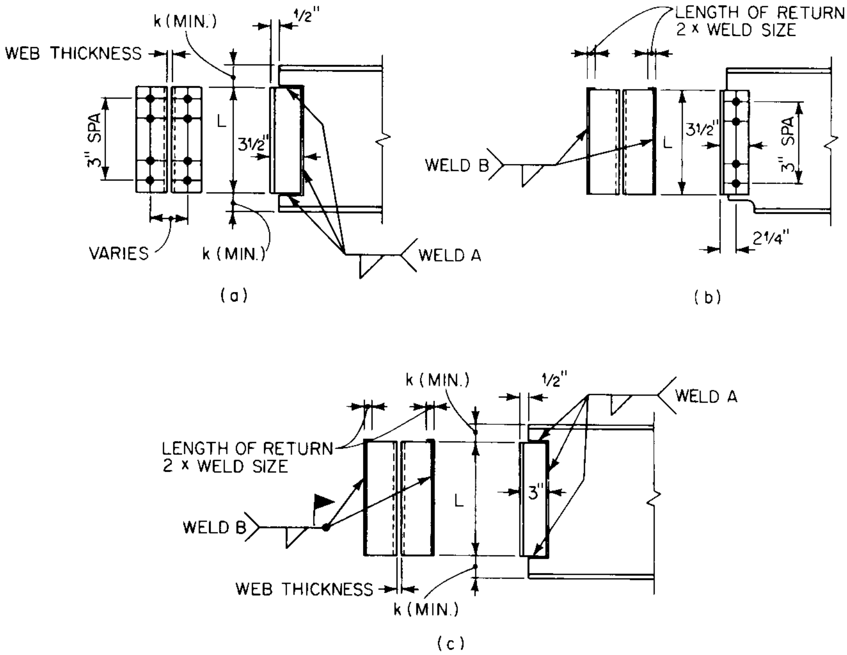


FIGURE 5.51 Welded framed connections. (*a*) Welds replace bolts of the standardized connection at the web. (*b*) Welds replace bolts of the standardized connection for outstanding legs of framing angles. (*c*) All-welded standardized connection.

is an eccentricity of load equal to the distance from the back of the outstanding legs to the center of gravity of welds *A*. Therefore, when the connections in Fig. 5.51 are used, the combination of vertical shear and moment on welds *A* should be taken into account in design, unless the tables in the AISC manual are used.

For the connection in Fig. 5.51*b*, the welds usually are made in the shop. Consequently, the beam bottom must be coped to permit the beam to be inserted between the angles in the field.

Welds *B* also are eccentrically loaded. The beam reaction is transmitted from the center of web to the welds along the toes of the outstanding legs. This moment, too, should be taken into account in design. To prevent cracking, the vertical welds at the top of the angles should be returned horizontally for a distance of twice the weld size.

Standardized framed connections of the type shown in Fig. 5.51*c* were developed especially for welding of both the web legs and outstanding legs of the connection angles.

5.32.2 Seated Connections

These may be unstiffened, as shown in Fig. 5.50*b* and Fig. 5.53*a*, or stiffened, as shown in Fig. 5.52 and Fig. 5.53*b*. A stiffened seat usually is used when loads to be carried exceed the capacities of the outstanding leg of standardized unstiffened seats. Tables in the AISC manuals facilitate the design of both types of connections.

The primary use for seated connections is for beams framing to column webs. In this case, the seat is inside the column flange toes or nearly so, and is not an architectural

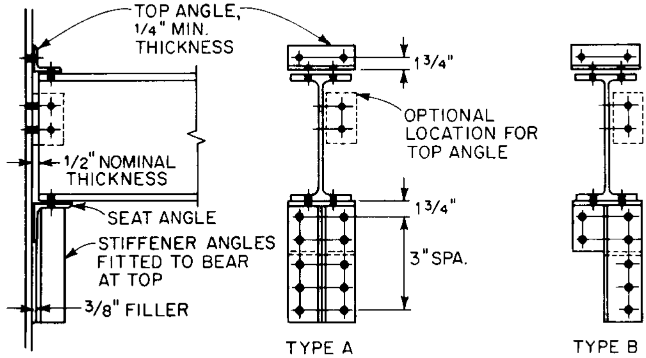


FIGURE 5.52 Standardized stiffened-seat connections with bolts.

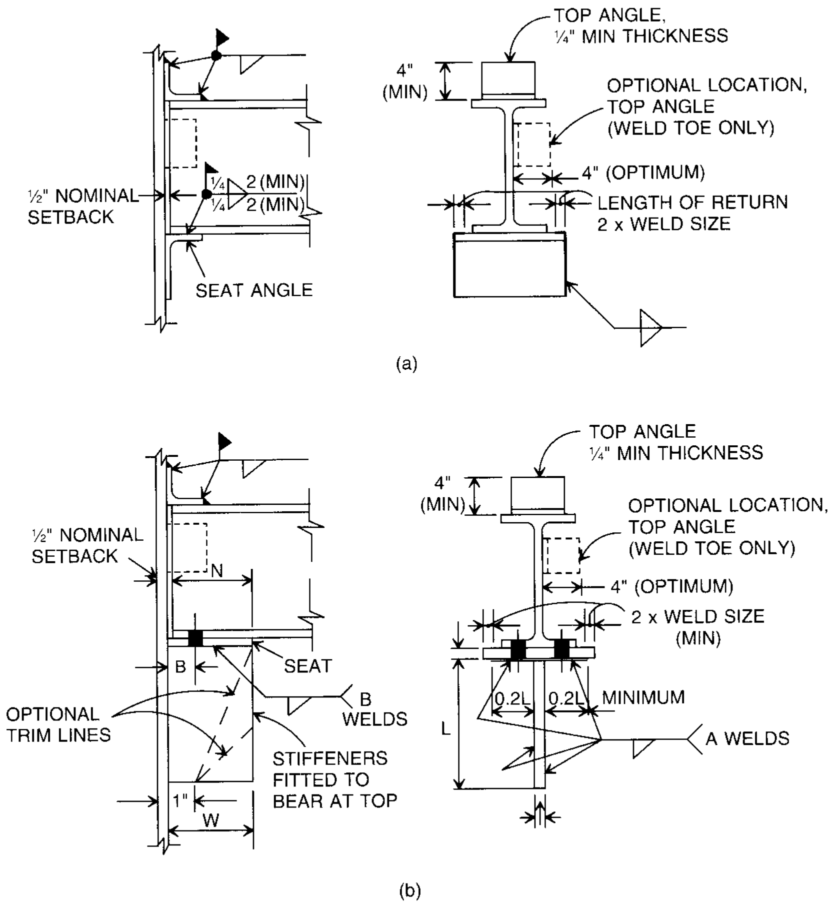


FIGURE 5.53 Standardized welded-seat connections. (*a*) Unstiffened seat. (*b*) Stiffened seat.

problem. Its use also avoids the erection safety problem associated with framed connections where the same bolts support beams on both sides of the column web.

When a seat is attached to one side of the column web, the column web is subjected to a local bending pattern because the load from the beam is applied to the seat at some distance e_f from the face of the web. The stiffened seat design table (Table 9.9) in the AISC LRFD manual includes this effect. For unstiffened seats, column web bending also occurs, but its effects are less critical because the loads and eccentricities for unstiffened seats are generally much smaller than for stiffened seats. Fig. 5.54*a* presents a yield line pattern which can be used to assess the strength of the column web. The nominal capacity of the column web is

$$R_w = \frac{2m_pL}{e_f}\left(2\sqrt{\frac{T}{b}} + \frac{T}{L} + \frac{L}{2b}\right) \tag{5.22}$$

where the terms are defined in Fig. 5.54*a* and

$$m_p = \frac{1}{4}t_w{}^2F_y \tag{5.23}$$

$$b = \frac{T - c}{2} \tag{5.24}$$

Since this is a yield limit state, $\phi = 0.90$.

Design of a seated connection generally is based on the assumption that the seat carries the full beam reaction. The top, or cap, angle only provides lateral support. Even for large beams, this angle can be small and can be attached with only two bolts in each leg (Fig. 5.52) or a toe weld along each leg (Fig. 5.53).

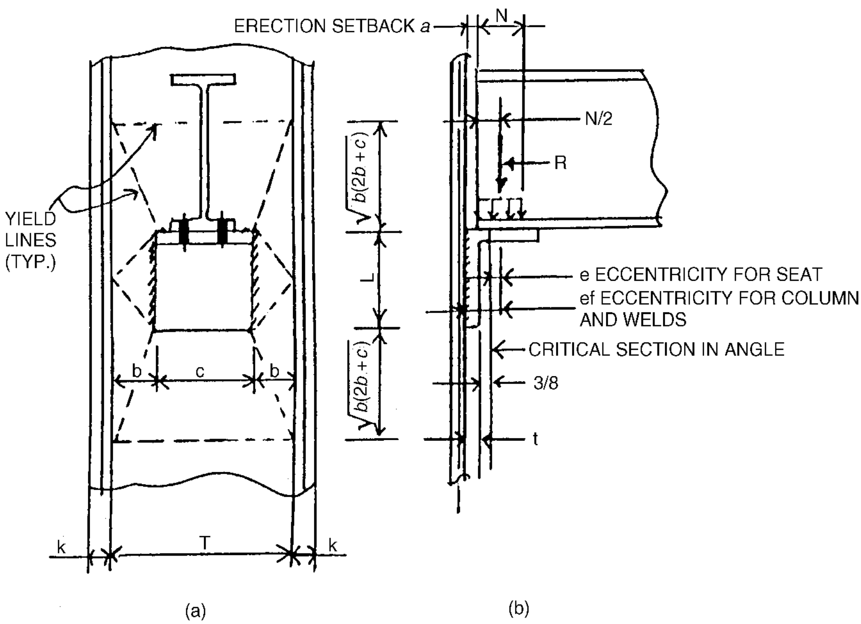


FIGURE 5.54 Unstiffened seated connection. (*a*) Yield lines for analysis of column web. (*b*) Design parameters. (*From* W. A. Thornton and T. Kane, "Connections," Chapter 7, *Steel Design Handbook—LRFD Method,* A. R. Tamboli, Ed., 1997, *McGraw-Hill,* 1997, *with permission.*)

With the nominal tolerance of ½ in between beam end and face of support, the length of support provided a beam end by a seat angle equals the width of the outstanding leg less ½ in. Thus a typical 4-in-wide angle leg provides 3½ in of bearing. Because of the short bearing, the capacity of a seated connection may be controlled by the thickness of the beam web, for resisting web yielding and crippling.

Tables in the AISC manuals list beam reactions, R for ASD and ϕR for LRFD, for 4-in wide outstanding angle legs that are based on a nominal setback a of ½ in and a beam underrun of ¼ in. Thus, calculations are based on the beam end being ¾ in from the face of the column. The reaction is assumed centered on the bearing length N. Both manuals list additional parameters, R_1 through R_4 for ASD and ϕR_1 through ϕR_6 for LRFD, that can be used to determine the web crippling and local web yielding limitations for other bearing lengths.

For unstiffened seats, the bearing length is assumed to extend from the beam end toward midspan. For stiffened seats, the bearing length is assumed to extend from the end of the seat toward the beam end. In design of the seat, however, an eccentricity from the face of the support of 80% of the beam-seat width is used if it is larger than the eccentricity based on the reaction position at the center of N.

Unstiffened Seats. The capacity of the outstanding leg of an unstiffened seat is determined by its resistance to bending. The critical section for bending is assumed to be located at the toe of the fillet of the outstanding leg. When reactions are so large that more than a nominal 3½ in of bearing is required, stiffened seats usually are used.

In addition to the capacity of the outstanding leg, the capacity of an unstiffened seat depends also on the bolts or welds used to connect to the column. The small eccentricity of the beam reaction generally is neglected in determining bolt capacities, but is included when calculating weld capacities.

Example—Unstiffened Seat—AISC LRFD. A W14 × 22 beam of Grade 50 steel is to be supported on an unstiffened seat to a W14 × 90 Grade 50 column. The factored reaction (required strength) is 33 kips. Design the unstiffened seat.

The nominal erection set back a = ½ in. For calculations, to account for underrun, use a = ¾. Try a seat 6 in long (c = 6). From AISC LRFD Manual Table 9.7, with beam web of approximately ¼ in, a ⅝ in angle gives a capacity 37.7 kips > 33 kips, OK Choosing an A36 angle 6 × 4 × ⅝, Table 9.7 indicates that a 5⁄16 in fillet weld of the seat vertical leg (the 6 in leg) to the column web is satisfactory (41.0 kips). Consider this to be a preliminary design that must be checked.

The first step in checking is to determine the required bearing length N. Note that N is not the horizontal angle leg length minus a, but rather cannot exceed this value. The bearing length for an unstiffened seat starts at the end of the beam and spreads from this point, because the toe of the angle leg tends to deflect away from the bottom flange of the beam. The bearing length cannot be less that the beam k distance and can be written in a general way as

$$N = \max\left\{\frac{R - \phi R_1}{\phi R_2}, \frac{R - \phi R_3}{\phi R_4} \text{ or } \frac{R - \phi R_5}{\phi R_6}, k\right\} \tag{5.25}$$

where R_1 through R_6 are defined in the AISC LRFD Manual and are tabulated in the Factored Uniform Load Tables. For the $W14 \times 22$, $\phi R_1 = 25.2$, $\phi R_2 = 11.5$, $\phi R_3 = 23.0$, $\phi R_4 = 2.86$, $\phi R_5 = 20.4$, and $\phi R_6 = 3.81$. Thus

$$N = \max\left\{\frac{33 - 25.2}{11.5}, \frac{33 - 23.0}{2.86} \textit{ or } \frac{33 - 20.4}{3.81}, 0.875\right\} = \max\{.67, 3.50 \textit{ or } 3.31, 0.875\}$$

Therefore N is either 3.50 or 3.31, depending on whether $N/d \leq .2$ or $N/d > .2$, respectively.

With $d = 13.74$, $N/d = 3.50/13.74 = 0.255$ or $N/d = 3.31/13.74 = 0.241$. Since $N/d > .2$, $N = 3.31$ in.

It was stated earlier that $(N + a)$ cannot exceed the horizontal angle leg. Using $a = 3/4$ in, $N + a = 3.31 + 0.75 = 4.06$ which is close enough to 4 in to be OK.

The design strength of the seat angle critical section is

$$\phi R_b = \frac{1}{4} \frac{ct^2}{e} \phi F_y \tag{5.26}$$

where the terms are defined in Fig. 5.54. From Fig. 5.54, $e_f = N/2 + a = 3.31/2 + 0.75 = 2.41$ and $e = e_f - t - 0.375 = 2.41 - 0.625 - 0.375 = 1.41$. Substituting values,

$$\phi R_b = \frac{6 \times 0.625^2 \times 0.9 \times 36}{4 \times 1.41} = 13.5 \text{ kips}$$

Since 13.5 kips < 33 kips, the seat is unsatisfactory. The required thickness can be determined from

$$t_{req'd} = \sqrt{\frac{4 \text{ Re}}{\phi F_y c}} = \sqrt{\frac{4 \times 33 \times 1.41}{0.9 \times 36 \times 6}} = 0.98 \text{ in} \tag{5.27}$$

Therefore, an angle 1 in thick can be used, although a thinner angle between 5/8 and 1 may check since e depends on t. There is no L 6 × 4 × 1 available, so use L 6 × 6 × 1. The extra length of the horizontal leg is irrelevant.

It can be seen from the above result that the AISC LRFD Manual Table 9-7 should not be relied upon for final design. The seats and capacities given in Table 9-7 are correct for the seats themselves, but the beam may not check. AISC Manual Tables 9-6 and 9-7 were originally derived based on the bearing length required for web yielding (i.e., ϕR_1 and ϕR_2). The new requirements of web crippling (ϕR_3 and ϕR_4, or ϕR_5 and ϕR_6) must be considered in addition to the capacities given in the AISC tables. If web yielding is more critical than web crippling, the tables will give satisfactory capacities. A future revision of the tables is anticipated.

Next, the weld of the seat vertical leg to the column web is checked. AISC LRFD Manual Table 9-7 indicated a 5/16-in fillet weld was required. This can be checked using AISC LRFD Manual Table 8-38 with the following: $e_x = e_f = 2.41$, $l = 6$, $a = 2.41/6 = 0.40$, and thus $C = 2.00$. Consequently, $\phi R_{\text{weld}} = 2.00 \times 5 \times 6 = 60.0$ kips > 33 kips OK. The weld sizes given in AISC Tables 9-6 and 9-7 will always be found to be conservative because they are based on using the full horizontal angle leg minus a as the bearing length N. Finally, check the column web using the yield line pattern of Fig. 5.45a and Eq. (5.22).

$$m_p = 0.25 \times (0.440)^2 \times 50 = 2.42 \text{ in-kips per in}$$

$$T = 11.25 \text{ in}$$

$$c = 6 \text{ in}$$

$$L = 6 \text{ in}$$

$$b = \frac{11.25 - 6}{2} = 2.625 \text{ in}$$

$$\phi R_{web} = \frac{0.9 \times 2 \times 2.42 \times 6}{2.41} \left(2 \sqrt{\frac{11.25}{2.625}} + \frac{11.25}{6} + \frac{6}{2 \times 2.625} \right)$$

$$= 77.6 \text{ kips} > 33 \text{ kips OK.}$$

This completes the calculations for this example. The final design is shown in Fig. 5.55.

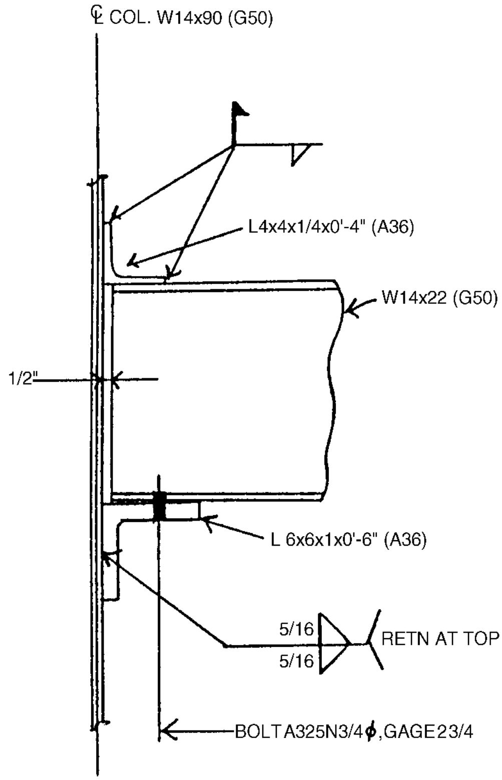


FIGURE 5.55 Example of unstiffened seated connection. (*From* W. A. Thornton and T. Kane, "Connections," Chapter 7, *Steel Design Handbook—LRFD Method,* A. R. Tamboli, Ed., 1997, *McGraw-Hill,* 1997 *with permission.*)

Stiffened Seats. These require that stiffeners be fitted to bear against the underside of the seat. The stiffeners must be sized to provide adequate length of bearing for the beam, to prevent web yielding and crippling. Area of stiffeners must be adequate to carry the beam reaction at the allowable bearing stress.

When bolts are used, the seat and stiffeners usually are angles (Fig. 5.52). A filler with the same thickness as the seat angle is inserted below the seat angle, between the stiffeners and the face of support. For light loads, a single stiffener angle may be used (type B, Fig. 5.52). For heavier loads, two stiffener angles may be required (type A, Fig. 5.52). Outstanding legs of these angles need not be stitched together. To accommodate the gage of fasteners in the supporting member, paired stiffeners may be separated. But the separation must be at least 1 in wide and not more than twice $k - t_s$, where k is the distance from outer surface of beam flange to web toe of fillet (in), and t_s is the stiffener thickness (in).

For standardized stiffened-seat connections, ⅜-in-thick seat angles are specified. The outstanding leg is made wide enough to extend beyond the outstanding leg of the stiffener angle. The width of the vertical leg of the seat angle is determined by the type of connection.

In determination of the bearing capacity of a stiffener, the effective width of the outstanding leg of the stiffener generally is taken as ½ in less than the actual width.

When stiffened seats are to be welded, they can be fabricated by welding two plates to form a tee (Fig. 5.53*b*) or by cutting a T shape from a wide-flange or I beam. When two plates are used, the stiffener should be fitted to bear against the underside of the seat. Thickness of the seat plate usually equals that of the stiffener but should not be less than ⅜ in.

The stiffener usually is attached to the face of the support with two fillet welds over the full length L of the stiffener. The welds should be returned a distance of at least $0.2L$ along the underside of the seat on each side of the stiffener. The welds are subjected to both shear and tension because of the eccentricity of the loading on the seat. Design is much the same as for the bracket in Fig. 5.49 (Art. 5.31).

Size and length of welds between a seat plate and stiffener should be equal to or greater than the corresponding dimensions of the horizontal returns.

Stiffener and seat should be made as narrow as possible while providing required bearing. This will minimize the eccentricity of the load on the welds. For a channel, however, the seat plate, but not the stiffener, usually is made 6 in wide, to provide space for two erection bolts. In this case the seat projects beyond the stiffener, and length of welds between seat and stiffener cannot exceed the stiffener width.

Determination of stiffener thickness may be influenced by the thickness of the web of the beam to be supported and by the size of weld between seat and support. One recommendation is that stiffener thickness be at least the product of beam- web thickness and the ratio of yield strength of web to yield strength of seat material. A second rule is based on Table 5.13. To prevent an A36 plate from being over- stressed in shear by the pair of vertical fillet welds (E70XX electrodes), the plate thickness should be at least

$$t = \frac{2 \times 0.707D \times 21}{14.4} \simeq 2D \tag{5.28}$$

where D is weld size (in). Although expressed in ASD format, the relationship also applies for LRFD.

Similarly, overstressing in shear should be avoided in the web of a supporting member of A36 steel where stiffened seats are placed on opposite sides of the web. If the vertical welds are also on opposite sides of a part of the web, the maximum weld size is half the web thickness.

Example—Welded Stiffened Seat—AISC ASD. For A36 steel, design a welded stiffened seat of the type shown in Fig. 5.53*b* to carry the 85-kip reaction of a W27 × 94 beam on a column web.

The tables for allowable uniform loads in the AISC ASD manual give for a W27 × 94, $R_1 = 41.8$, $R_2 = 11.6$, $R_3 = 60.4$, and $R_4 = 3.59$. Also, $k = 1\frac{7}{16}$ in. Thus the required bearing length (in) is 6.85 in, the largest value of k,

$$N = (85 - 41.8)/11.6 = 3.72 \qquad \text{and} \qquad N = (85 - 60.4)/3.59 = 6.85$$

The required width of seat is 6.85 + 0.5 = 7.35 in. Use an 8-in-wide seat. From the AISC ASD manual, Table VIII, the required length L of stiffener plate (Fig. 5.53*b*) is 16 in when ⁵⁄₁₆-in fillet welds (welds A) are used between the stiffener plate and the column web. According to Table VIII, this connection is good for 94.4 kips > 85 kips—OK.

The thickness of the stiffener plate is taken as twice the weld size when A36 plate and E70 electrodes are used. Therefore, the stiffener plate should be ⅝ in thick (dimension t of Fig. 5.53*b*). The seat-plate thickness is usually taken to be the same as the stiffener plate but should not be less than ⅜ in.

The weld between the stiffener and the seat plate (welds B of Fig. 5.53*b*) is usually taken to be the same size as welds A, but an AISC minimum weld can be used as long as it develops the strength of the $0.2L$ returns of welds A to the seat plate. In the calculation of the capacity of the fillet weld, it is convenient to use the capacity per inch per sixteenths of an inch of weld size, which is equal to $21.0 \times 0.707/16 = 0.928$ kips for a 21.0-ksi allowable stress. Thus the design load for welds B is $0.2 \times 16 \times 5 \times 0.928 \times 2 = 29.7$ kips. The length of welds B is 8 in. The required weld size in number of sixteenths of an inch is

$$D = 29.7/(8 \times 2 \times 0.928) = 2.0$$

Thus a 3⁄16-in fillet weld will check, but the AISC minimum weld for the 5⁄8-in stiffener plate is 1⁄4 in. Use a 1⁄4-in fillet weld for welds B.

5.33 MOMENT CONNECTIONS

The most commonly used moment connection is the field welded connection shown in Fig. 5.56. This connection has been in common use throughout the U.S. for many years. In current seismic design covered by the AISC ''Seismic Provisions for Structural Steel Buildings,'' it is permitted for use in ordinary moment-resisting frames (Art. 9.7) without requirements for physical testing. It is also permitted for use in special moment-resisting frames, when the member sizes used for the specific project have been tested to demonstrate that the required ductility level can be achieved. Furthermore, it is widely used in areas of low seismicity where the AISC seismic provisions do not apply, and in frames designed primarily for wind and gravity forces, such as in the following example.

Example—Three Way Moment Connection—AISC LRFD. The moment connection of Fig. 5.56*a* is a three-way moment connection. Additional views are shown in Figs. 5.56*b* and 5.56*c*. If the strong axis connection requires stiffeners, there will be an interaction between the flange forces of the strong and weak axis beams. If the primary function of these moment connections is to resist lateral maximum load from wind or seismic sources, the interaction can generally be ignored because the maximum lateral loads will act in only one direction at any one time. If the moment connections are primarily used to carry gravity loads, such as would be the case when stiff floors with small deflections and high natural frequencies are desired, there will be interaction between the weak and strong beam flange forces. The calculations here will be for a wind or a seismic condition in a region of low to moderate seismicity, but interaction will be included to demonstrate the method.

Following common practice based on tests, the load path assumed here is that the moment is carried entirely by the flanges, and the shear entirely by the web. Proceeding to the connection design, the strong axis beam, beam No. 1, will be designed first.

Design of Beam No. 1. Beam No. 1 is a composite W21 × 62 section of A36 steel. The flange connection is a full penetration weld so no connection design is required. The column must be checked for stiffeners and doublers.

Design of Column Stiffeners. The connection is to be designed for the full moment capacity of the beam, ϕM_p. Thus, the flange force F_f is

$$F_f = \frac{\phi M_p}{d - t_f} = \frac{389 \times 12}{(20.99 - 0.615)} = 229 \text{ kips}$$

where d is the beam depth and t_f is the flange thickness. From the column load tables of the AISC LRFD Manual, Vol. I, find the following parameters.

Web yielding: $P_{wy} = P_{wo} + t_f\, P_{wi} = 174 + 0.615 \times 24.3 = 189$ kips < 229 kips, thus stiffeners are required at both flanges.

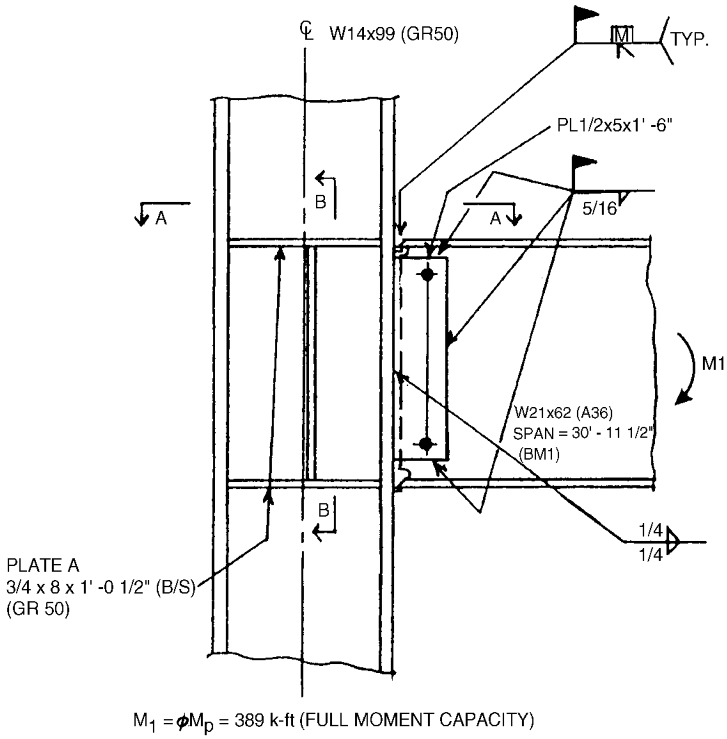


FIGURE 5.56*a* Example of field-welded moment connection. For section A–A, see Fig. 5.56*c*. For section B–B, see fig. 5.56*b*. (*From* W. A. Thornton and T. Kane, ''Connections,'' Chapter 7, *Steel Design Handbook—LRFD Method,* A. R. Tamboli, Ed., 1997, *McGraw-Hill,* 1997 *with permission*.)

Web buckling: $P_{wb} = 261$ kips > 229 kips—no stiffener required at compression flange.

Flange bending: $P_{fb} = 171$ kips < 229 kips—stiffener required at tension flange.

From the above three checks (limit states), a stiffener is required at both flanges. For the tension flange, the total stiffener force is $229 - 171 = 58$ kips and for the compression flange, the stiffener force is $229 - 189 = 40$ kips. Because the loads may reverse, use the larger value of 58 kips as the stiffener force for both flanges. Then, the force in each stiffener is $58/2 = 29$ kips, both top and bottom.

The minimum stiffener width w_s depends on the flange width of the beam, b_{fb}, and the web thickness of the column, t_{wc}:

$$w_s = \frac{b_{fb}}{3} - \frac{t_{wc}}{2} = \frac{8.24}{3} - \frac{0.485}{2} = 2.5 \text{ in}$$

Use a stiffener 6½ in wide to match column.

The minimum stiffener thickness t_s is

$$t_s = \frac{t_{fb}}{2} = \frac{0.615}{2} = 0.31 \text{ in}$$

Use a stiffener at least ⅜ in thick.

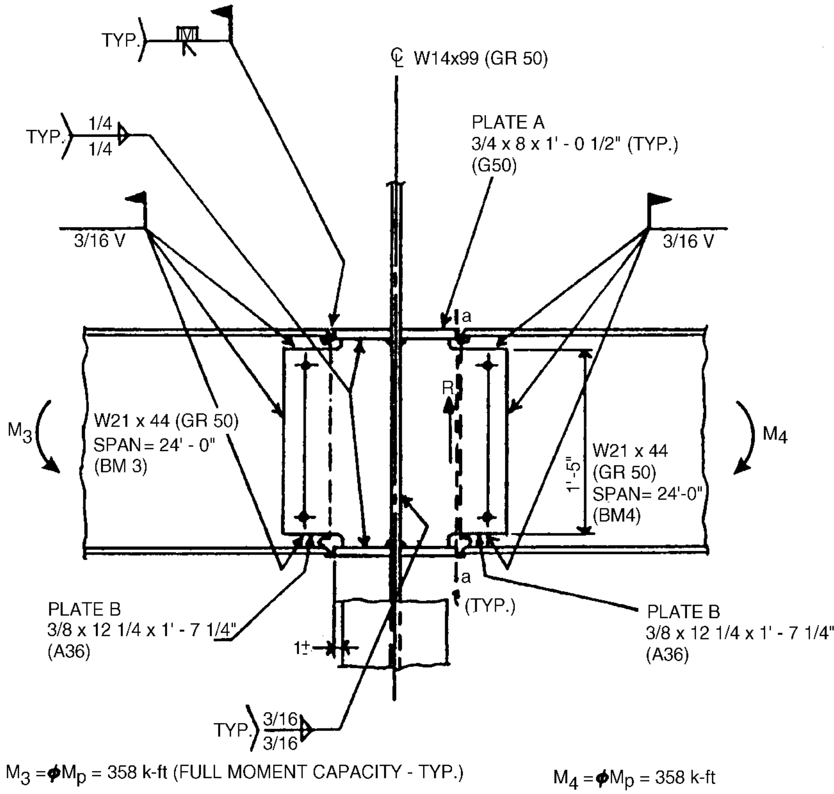


FIGURE 5.56*b* Example of field-welded moment connection; section B–B of Fig. 5.56*a*. (*From* W. A. Thornton and T. Kane, "Connections," Chapter 7, *Steel Design Handbook—LRFD Method,* A. R. Tamboli, Ed., 1997, *McGraw-Hill,* 1997 *with permission.*)

The minimum stiffener length l_s depends on the column depth, d_c, and the flange thickness of the column, t_{fc}:

$$\frac{d_c}{2} - t_{fc} = \frac{14.16}{2} - 0.78 = 6.3 \text{ in}$$

The minimum length is for a "half depth" stiffener, which is not possible in this example because of the weak axis connections. Therefore, use a full depth stiffener, 12½ in long.

A final stiffener size consideration is a plate buckling check which requires that

$$t_s \geq w_s \frac{\sqrt{F_y}}{95} = 6.5 \frac{\sqrt{36}}{95} = 0.41 \text{ in}$$

Therefore, the minimum stiffener thickness is ½ in and the final stiffener size for the strong axis beam is ½ × 6½ × 12½ in. The contact area of this stiffener against the inside of the column flange is 6.5 − 0.75 = 5.75 in due to the snip to clear the column web-to-flange fillet. The stiffener design strength is thus 0.9 × 36 × 5.75 × 0.5 = 93.2 kips > 29 kips, OK.

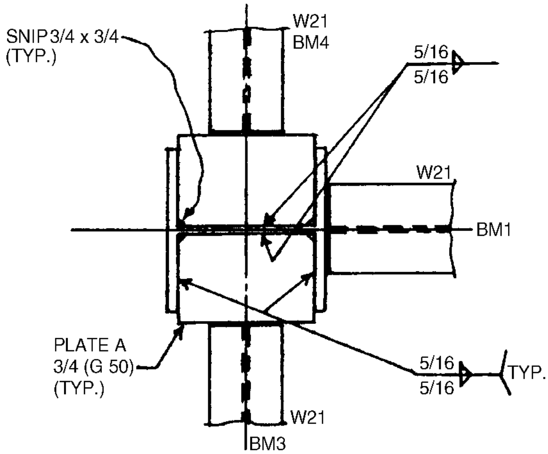


FIGURE 5.56*c* Example of field-welded moment connection; section A–A of Fig. 5.56*a*. (*From* W. A. Thornton and T. Kane, "Connections," Chapter 7, *Steel Design Handbook—LRFD Method,* A. R. Tamboli, Ed., 1997, *McGraw-Hill,* 1997 *with permission*.)

Weds of Stiffeners to Column Flange and Web. Putting aside for the moment that the weak axis moment connections still need to be considered and will affect both the strong axis connection stiffeners and welds, the welds for the ½ × 6½ × 12½ strong axis stiffener are designed as follows. For the weld to the inside of the flange, the connected portion of the stiffener must be developed. Thus, the 5¾ in contact, which is the connected portion, is designed for 93.2 kips rather than 29 kips, which is the load the stiffener actually carries. The size of the weld to the flange in number of sixteenths of an inch is thus

$$D_f = \frac{93.2}{2 \times 5.75 \times 1.392 \times 1.5} = 3.9$$

A ¼ in fillet weld is indicated, or use the AISC minimum if larger. The factor 1.5 in the denominator above comes from the AISC LRFD Specification Appendix J, Section J2.4 for transversely loaded fillets. The weld to the web has a length 12.5 − 0.75 − 0.75 = 11.0 in, and is designed to transfer the unbalanced force in the stiffener to the web. The unbalanced force in the stiffener is 29 kips in this case. Thus,

$$D_w = \frac{29}{2 \times 11.0 \times 1.392} = 0.95$$

Because only a 1⁄16 in weld is indicated, the AISC minimum fillet weld size governs.

Doubler Plate Design The beam flange force (required strength) delivered to the column is $F_f = 229$ kips. The design shear strength of the column $\phi V_v = 0.9 \times 0.6 \times 50 \times 0.485 \times 14.16 = 185$ kips < 229 kips, so a doubler appears to be required. However, if the moment that is causing doublers is $\phi M_p = 389$ ft-kips, then from Fig. 5.57, the column story shear is

$$V_s = \frac{\phi M_p}{H}$$

where H is the story height. If $H = 13$ ft

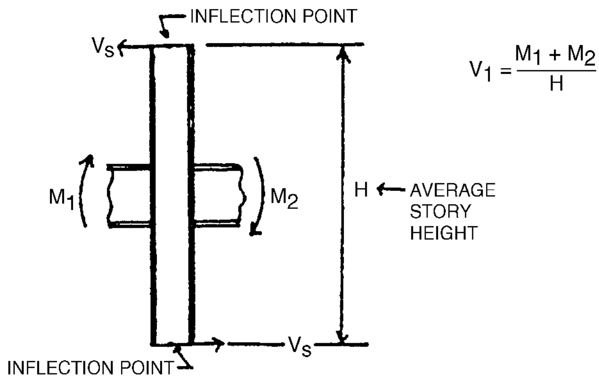


FIGURE 5.57 Relationship between column story shear and beam end moments. (*From* W. A. Thornton and T. Kane, "Connections," Chapter 7, *Steel Design Handbook—LRFD Method,* A. R. Tamboli, Ed., 1997, *McGraw-Hill,* 1997 *with permission.*)

$$V_s = \frac{389}{13} = 30 \text{ kips}$$

and the shear delivered to the column web is $F_f - V_s = 229 - 30 = 199$ kips. Since 199 kips $>$ 185 kips, a doubler plate (or plates) is still indicated. However, if some panel zone deformation is acceptable, and is considered in the analysis, the panel zone strength may be increased. AISC LRFD Specification Eq. K1-11 or Eq. K1-12, contain the following extra term which acts as a multiplier on the basic strength ϕV_v:

$$\frac{3b_{fc}t_{fc}^{2}}{d_b d_c t_{wc}} = \frac{3 \times 14.565 \times 0.780^2}{20.66 \times 14.16 \times 0.485} = 0.187$$

If the column load is less than $0.75\ P_y = 0.75 \times A_c F_{yc} = 0.75 \times 29.1 \times 50 = 1091$ kips, which is the usual case, Eq. K1-11 applies and

$$\phi V_v = 185(1 + 0.181) = 220 \text{ kips}$$

Since 220 kips $>$ 199 kips, no doubler is required. In a high-rise building where the moment connections are used for drift control, the extra term can still be used, but an analysis which includes inelastic joint shear deformation should be considered.

Doubler Plate Placement. If a doubler plate or plates had been required in this example, the most economical arrangement would be to place the doubler plate against the column web between the stiffeners (the panel zone) and to attach to weak axis shear connection plates, plates *B*, to the face of the doubler. This is permissible provided that the doubler is capable of carrying the entire weak axis shear load $R = 107$ kips (specified shear load for W21×44 G50 beam) on one vertical cross section of the doubler plate. To see this, consider Fig. 5.58. The portion of the shear force induced in the doubler plate by the moment connection flange force F_f is H. For the doubler to be in equilibrium under the forces H, vertical shear forces $V = Hd/w$ exist. The welds of the doubler at its four edges develop the shear strength of the doubler. Let the shear force R from the weak axis connection be applied to the face of the doubler at or near its horizontal center as shown in Fig. 5.58. If it is required that all of the shear R can be carried by one vertical section a-a of Fig. 5.58, (0.9 × 0.6 × $F_y d \geq R$, where t_d is the doubler thickness and F_y is the yield stress of the doubler and the

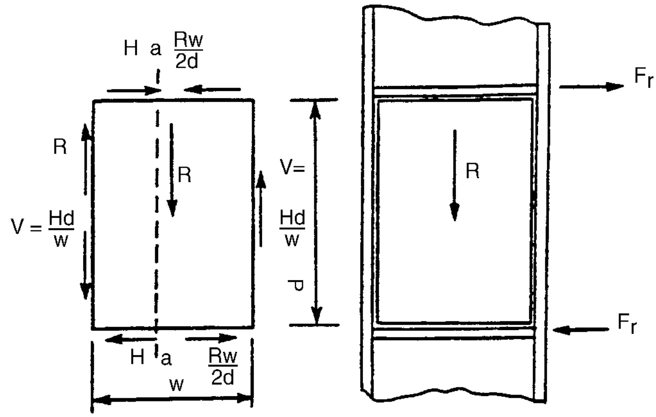


FIGURE 5.58 Force equilibrium diagram for doubler plate with weak axis shear load. (*From* W. A. Thornton and T. Kane, "Connections," Chapter 7, *Steel Design Handbook—LRFD Method,* A. R. Tamboli, Ed., 1997, *McGraw-Hill,* 1997 *with permission.*)

column), then the free body diagram of Fig. 5.58 is possible. In this figure, all of the shear force R is delivered to the side of the doubler where it is opposite in direction to the shear delivered by the moment connection, thereby avoiding over-stressing the other side where the two shears would add. Since the doubler and its welds are capable of carrying V or R alone, they are capable of carrying their difference. The same argument applies to the top and bottom edges of the doubler. Also, the same argument holds if the moment and/or weak axis shear reverse(s).

Associated Shear Connections—Beam 1. The specified shear for the web connection is $R = 163$ kips, which is the shear capacity of the W21 × 62 A36 beam. The connection is a shear plate with two erection holes for erection bolts. The shear plate is shop welded to the column flange and field welded to the beam web. The limit states are plate gross shear, weld strength, and beam web strength.

For plate gross shear try a plate ½ × 18 in. the gross shear strength is

$$\phi R_{gv} = 0.5 \times 18 \times 0.9 \times 0.6 \times 36 = 175 \text{ kips} > 163 \text{ kips OK.}$$

Plate net shear need not be checked because it is not a valid limit state.

The weld to the column flange is subjected to shear only. Thus, the weld size in number of sixteenths of an inch is

$$D = \frac{163}{2 \times 18 \times 1.392} = 3.25$$

Use a ¼-in fillet weld.

The weld to the beam web is subjected to the shear plus a small couple. Using AISC LRFD Manual Table 8-42, $l = 18$ in, $kl = 4.25$ in, $k = 0.24$, $x = 0.04$, $xl = 0.72$ in $al = 4.28$ in, $a = 0.24$, $c = 2.04$, and thus,

$$D = \frac{163}{2.04 \times 18} = 4.44$$

A 5⁄16-in fillet weld is satisfactory.

Next consider the beam web thickness. To support a 5⁄16 fillet weld on both sides of a plate, AISC LRFD Manual Table 9-3, shows that a 0.72-in thick web is required. For a 5⁄16-in fillet on one side, a 0.36 in thick web is required. Since the W21 × 62 web is 0.400 in thick, it is OK.

Design of Beams Nos. 3 and 4. These beams are W21 × 44 sections of Grade 50 steel and are composite. The flange connection is a full penetration weld so again, no design is required for the connection. Section A-A of Fig. 5.56*a* shows the arrangement in plan. See also Fig. 5.56*c*. The connection plates A are made ¼ in thicker than the W21 × 44 beam flange to accommodate under and over rolling and other minor misfits. Also, the plates are extended beyond the toes of the column flanges by ¾ to 1 in to improve ductility. Plates A should also be welded to the column web, even if not required to carry load, to provide improved ductility.

The flange force for the W21 × 44 is based on the full moment capacity as required in this example, so ϕM_p = 358 ft-kip and

$$F_f = \frac{358 \times 12}{(20.66 - 0.45)} = 213 \text{ kips}$$

Figure 5.59 shows the distribution of forces on Plates A, including the forces from the strong axis connection. The weak axis force of 213 kips is distributed ¼ to each flange and ½ to the web. This is done to cover the case when the beams may not be reacting against each other. In this case, all of the 213 kips must be passed to the flanges. To see this, imagine that beam 4 is removed and Plate A for beam 4 remains as a back-up stiffener. One-half of the 213 kips from beam 3 passes into the beam 3 near side column flanges, while the other half is passed through the column web to the back-up stiffener, and thence into the far side flanges, so that all of the load is passed to the flanges. This is the load path usually assumed, although others are possible.

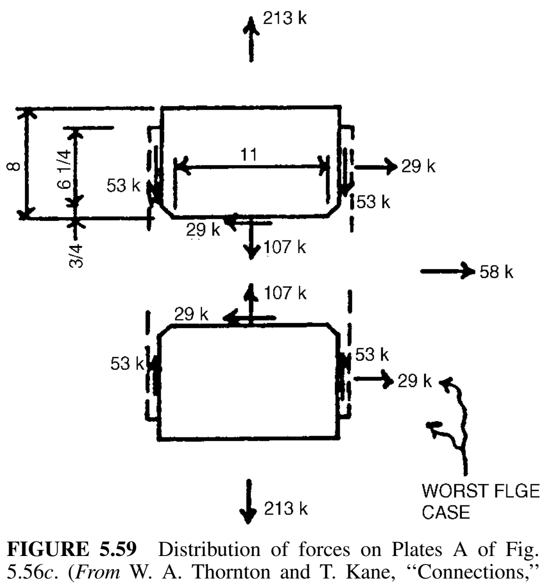


FIGURE 5.59 Distribution of forces on Plates A of Fig. 5.56*c*. (*From* W. A. Thornton and T. Kane, "Connections," Chapter 7, *Steel Design Handbook—LRFD Method,* A. R. Tamboli, Ed., 1997, *McGraw-Hill,* 1997 *with permission.*)

Merging of Stiffeners from Strong and Weak Axis Beams. The strong axis beam, beam 1, required stiffeners ½ × 6½ × 12½ in. Weak axis beams 3 and 4 require Plates A which are ¾ × 8 × 12½ in. These plates occupy the same space because the beams are all of the same depth. Therefore, the larger of the two plates is used, as shown in Fig. 5.56*a*.

Since the stiffeners are merged, the welds that were earlier determined for the strong axis beam must be revisited. For the weld to the flanges, from Fig. 5.59, the worst case combined flange loads are 53 kips shear and 29 kips axial. The length of weld is 6¼ in. Thus, the weld size is

$$D_f = \frac{\sqrt{29^2 + 53^2}}{2 \times 6.25 \times 1.392} = 3.5$$

which indicates a ¼-in fillet weld. This is also the AISC minimum size. However, remember that for axial load, the contact strength of the stiffener must be developed. The contact strength in this case is $0.9 \times 36 \times 6.25 \times 0.75 = 152$ kips, because the stiffener has increased in size to accommodate the weak axis beams. But the delivered load to this stiffener cannot be more than that which can be supplied by the beam, which is $229/2 = 114.5$ kips. Thus

$$D_f = \frac{114.5}{2 \times 6.25 \times 1.392 \times 1.5} = 4.39$$

which indicates that a 5⁄16 in fillet weld is required. This is the fillet weld that should be used as shown in Fig. 5.56*c*.

For the weld to the web, from Fig. 5.59, calculate the size as

$$D_w = \frac{\sqrt{29^2 + 107^2}}{2 \times 11.0 \times 1.392} = 3.62$$

Use a ¼-in fillet weld.

Stresses in Stiffeners (Plate A). The weak axis beams are 50 ksi yield point steel and are butt welded to Plates A. Therefore, Plates A should be the same strength steel. Previous calculations involving this plate assumed it was A36, but changing to a higher strength will not change the final results in this case because the stiffener contact force is limited by the delivered force from beam 1 rather than the stiffener strength.

The stiffener stresses for the flange welds are, from Fig. 5.59

$$f_v = \frac{53}{0.75 \times 6.25} = 11.3 \text{ ksi} < 0.9 \times .6 \times 50 = 27 \text{ ksi OK.}$$

$$f_a = \frac{29}{0.75 \times 6.25} = 6.19 \text{ ksi} < 0.9 \times 50 = 45 \text{ ksi OK.}$$

For the web welds,

$$f_v = \frac{29}{.75 \times 11} = 3.5 \text{ ksi} < 27 \text{ ksi OK.}$$

$$f_a = \frac{107}{.75 \times 11} = 13.0 \text{ ksi} < 45 \text{ ksi OK.}$$

Associated Shear Connections—Beams 3 and 4. The specified shear for these beams is $R = 107$ kips. First consider the weld to the beam web. As with the strong axis beam web connection, this is a field welded connection with bolts used for erection only. The design load (required strength) is $R = 107$ kips. The beam web shear R is essentially constant in the area of the connection and is assumed to act at the edge of Plates A (Section A-A of Fig. 5.56*b*). This being the case, there will be a small eccentricity on the C shaped field weld. Following AISC LRFD Manual Table 8.42, $l = 17$ in, $kl = 4$ in, $k = 0.24$, $x = 0.04$, $xl = 0.68$ in, $al = 4.25 - .68 = 3.57$ in. From Table 8.42 by interpolation, $c = 2.10$. The weld size required is

$$D = \frac{107}{2.10 \times 17} = 2.99$$

which indicates that a $\frac{3}{16}$ fillet weld should be used.

Plate B is a shear plate and will be sized for gross shear. Try a $\frac{3}{8}$-in thick plate of A36 steel. Then

$$\phi R_V = 0.9 \times 0.6 \times 36 \times 0.375 \times 17 = 124 \text{ kips} > 107 \text{ kips OK.}$$

Consider the weld of plate B to the column web. This weld caries all of the beam shear, $R = 107$ kips. The length of this weld is 17.75 in. Thus, the required weld size is

$$D = \frac{107}{2 \times 17.75 \times 1.392} = 2.17$$

A $\frac{3}{16}$-in fillet weld is indicated. Because this weld occurs on both sides of the column web, the column web thickness should satisfy the relationship $0.9 \times 0.6 \times t_w \geq 1.392 \times D \times 2$ or $t_w \geq 0.103 \times 2.17 = 0.22$ in. Since the column web thickness is 0.485 in, the web can support the $\frac{3}{16}$-in fillets. The same result can be achieved using AISC LRFD Manual Table 9.3.

Now consider the weld of plate B to plates A. There is a shear flow $q = VQ/I$ acting on this interface, where $V = R = 107$ kips, Q is the statical moment of each Plate A with respect to the neutral axis of the I section formed by Plates A as flanges and Plate B as web, and I is the moment of inertia of Plates A and Plate B. Thus,

$$I = \frac{1}{12} \times 0.375 \times (19.25)^3 + 0.75 \times 12.5 \times \left(\frac{19.25 + .75}{2}\right)^2 \times 2 = 2100 \text{ in}^4$$

$$Q = 0.75 \times 12.5 \times 10 = 93.8 \text{ in}^3$$

and

$$q = \frac{107 \times 93.8}{2100} = 4.78 \text{ kips per in}$$

The required weld size is

$$D = \frac{4.78}{2 \times 1.392} = 1.72$$

Since plates A are $\frac{3}{4}$ in thick, the AISC minimum fillet weld of $\frac{1}{4}$ in prevails.

A reconsideration of welds for Plates A is now required. In the course of this example, these welds have already been considered twice. The first time was as stiffener welds for the strong axis beam. The second time was for the combination of forces from the weak and strong axis beam flange connections. Now, additional forces are added to plates A from the weak axis beam web connections. The additional force is $4.78 \times 6.25 = 30$ kips. Figure 5.60 shows this force and its distribution to the plate edges. Rechecking welds for Plates A for the flange weld,

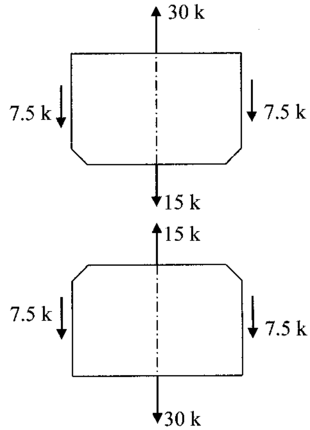


FIGURE 5.60 Additional forces acting on Plates A of Fig. 5.56*c*. (*From* W. A. Thornton and T. Kane, "Connections," Chapter 7, *Steel Design Handbook—LRFD Method,* A. R. Tamboli, Ed., 1997, *McGraw-Hill,* 1997 *with permission.*)

$$D_f = \frac{\sqrt{29^2 + (53 + 7.5)^2}}{2 \times 6.25 \times 1.392} = 3.86$$

which indicates that a ¼-in fillet is required. A 5⁄16-in fillet has already been determined. For the web weld,

$$D_w = \frac{\sqrt{(107 + 15)^2 + 29^2}}{2 \times 11.0 \times 1.392} = 4.08$$

This weld, which was previously determined to be a ¼-in fillet weld, must now be increased to a 5⁄16-in fillet weld.

This completes the calculations required to design the moment connections. Load paths of sufficient strength to carry all loads from the beams into the column have been provided. However, note that it sometimes happens in the design of this type of connection that the beam is much stronger in bending than the column. In the example just completed, this is not the case. For the strong axis W21 × 62 beam, $\phi M_p = 389$ ft-kip while for the column, $\phi M_p = 647$ ft-kip. If the ϕM_p of the column were less than half the ϕM_p of the beam, then the connection should be designed for $2(\phi M_p)$ of the column because this is the maximum moment that can be developed between the beam and column (maximum moment the system can deliver). Similar conclusions can be arrived at for other arrangements.

5.34 BEAMS SEATED ATOP SUPPORTS

There are many cases where beams are supported only at the bottom flange. In one-story buildings, for example, beams often are seated atop the columns. In bridges, beams generally are supported on bearings under the bottom flange. In all these cases, precautions should be taken to ensure lateral stability of the beams (see also Art. 5.29).

Regardless of the location of support, the compression flange of the beams should be given adequate lateral support. Such support may be provided by a deck or by bracing. In addition, beams supported at the lower flange should be braced against forces normal to the plane of the web and against eccentric vertical loading. The eccentricity may be caused by a column not being perfectly straight or perfectly plumb. Or the eccentricity may be produced by rolling imperfections in the beam; for example, the web may not be perfectly perpendicular to the flanges.

To resist such loadings, AASHTO and AREMA require that deck spans be provided with cross frames or diaphragms at each support. AREMA specifies that cross frames be used for members deeper than 3 ft 6 in and spaced more than 4 ft on centers. Members not braced with cross frames should have I-shaped diaphragms as deep as depth of beams permit. AASHTO prefers diaphragms at least half the depth of the members. Cross frames or diaphragms should be placed in all bays. This bracing should be proportioned to transmit all lateral forces, including centrifugal and seismic forces and cross winds on vehicles, to the bearings.

In through bridge spans, girders should be stiffened against lateral deformation by gusset plates or knee braces with solid webs. These stiffeners should be connected to girder stiffeners and to floorbeams.

In buildings, lateral support may be provided a girder seated atop a column by fastening cross members to it and stiffening the web at the support (Fig. 5.61*a*). Or a knee brace may be inserted at the support between the bottom flange of the girder and the bottom flange of a cross member (Fig. 5.61*b*). Where an open-web joist frames into a girder seated atop a column, lateral support can be provided by connecting the top chord of the joist to the top flange of the girder and attaching the bottom chord as shown in Fig. 5.61*c*.

5.35 TRUSS CONNECTIONS

Truss members generally are designed to carry only axial forces. At panel points, where members intersect, it is desirable that the forces be concurrent, to avoid bending moments. Hence the gravity axes of the members should be made to intersect at a point if practicable. When this cannot be done, the connections should be designed for eccentricity present. Also, groups of fasteners or welds at the ends of each member should have their centers of gravity on or near the gravity axis of the member. Otherwise, the member must be designed for eccentric loading. In building trusses not subject to repeated variation in stress, however,

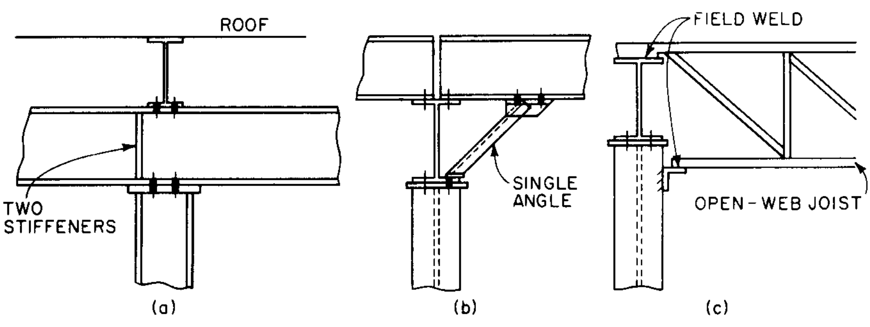


FIGURE 5.61 Bracing for girders. (*a*) Stiffeners brace a girder that is continuous over the top of a column. (*b*) Knee brace restrains a girder seated atop a column. (*c*) Open-web joist provides lateral support for a girder atop a column.

eccentricity of welds and fasteners may be neglected at the ends of single-angle, double-angle, and similar members.

At panel points, several types of connections may be used. Members may be pin-connected to each other (Art. 5.6), or they may be connected through gusset plates, or they may be welded directly to each other. In bridges, fasteners should be symmetrical with the axis of each member as far as practicable. If possible, the design should fully develop the elements of the member.

Design of connections at gusset plates is similar to that for gusset-plate connections illustrated in Art. 5.25. In all cases, gusset plates should be trimmed to minimize material required and for good appearance. When cuts are made, minimum edge distances for fasteners and seats for welds should be maintained. Reentrant cuts should be avoided.

At columns, if the reactions of trusses on opposite sides of a support differ by only a small amount (up to 20%), the gravity axes of end diagonal and chord may be allowed to intersect at or near the column face (Fig. 5.62). The connections need be designed only for the truss reactions. If there is a large difference in reactions of trusses on opposite sides of the support, or if only one truss frames into a column, the gravity axes of diagonal and chord should be made to intersect on the column gravity axis. Shear and moment in the column flange should be considered in design of the end connection.

If the connection of bottom chord to column is rigid, deflection of the truss will induce bending in the column. Unless the bottom chord serves as part of a bracing system, the connection to the column should be flexible. When the connection must be rigid, bending in the column can be reduced by attaching the bottom chord after dead-load deflection has occurred.

Splices in truss chords should be located as near panel points as practicable and preferably on the side where the smaller stress occurs. Compression chords should have ends in close contact at bolted splices. When such members are fabricated and erected with close inspection and detailed with milled ends in full-contact bearing, the splices may be held in place with high-strength bolts proportioned for at least 50% of the lower allowable stress of the sections spliced.

In other cases, compression and tension chords should be designed for not less than the average of the calculated stress at the point of splice and the strength of the member at that point but not less than 75% of the strength of the member (Arts. 5.26 and 5.27). Tension and compression chords may be spliced with complete-penetration groove welds without splice plates.

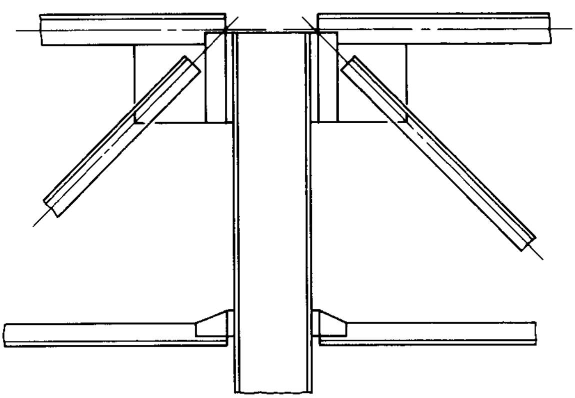


FIGURE 5.62 Truss connection to a column. The gravity axes of the top chords and the diagonals meet near the faces of the support.

Gusset Plates. These should be sized to resist all loads imposed. The design of gussets and the connection of members to the gusset is based on statics and yield strength, which are the basic ingredients of the lower-bound theorem of limit analysis. Basically, this theorem states that if equilibrium is satisfied in the structure (or connection) and yield strength is nowhere exceeded, the applied load will, at most, be equal to the load required to fail the connection. In other words, the connection will be safe. In addition to yield strength, gussets and their associated connections also must be checked to ensure that they do not fail by fracture (lack of ductility) and by instability (buckling).

AASHTO has a stability criterion for the free edges of gusset plates that requires that an edge be stiffened when its length exceeds plate thickness times $347/\sqrt{F_y}$, where F_y is the specified yield stress (ksi). This criterion is intended to prevent a gusset from flexing when the structure deforms and the angles between the members change. Because of repeated loading, this flexing can cause fatigue cracks in gussets.

For statically loaded structures, which include structures subjected to wind and seismic loads, this flexing is not detrimental to structural safety and may even be desirable in seismic design because it allows more energy absorption in the members of the structure and it reduces premature fracture in the connections. For buildings, the AISC seismic specification has detailing requirements to allow gussets to flex in certain situations. AISC ASD and LRFD manuals include requirements regarding gusset stability. It is important that gusset buckling be controlled to prevent changes in structure geometry that could render the structure unserviceable or cause catastrophic collapse (see also Art. 5.36).

Fracture in gusset-plate connections also must be prevented not only when connections are made with fillet welds, which have limited ductility in their transverse direction, but also with bolts, because of the possibility of tearout fracture and nonuniform distribution of tension and shear to the bolts.

5.36 CONNECTIONS FOR BRACING

The lateral force-resisting system of large buildings is sometimes provided by a vertical truss with connections such as that in Fig. 5.63. The design of this connection is demonstrated in the following example by a method adopted by the AISC, the **uniform-force method,** the force distributions of which are indicated in Fig. 5.64. The method requires that only uniform forces, no moments, exist along the edges of the gusset plate used for the connection.

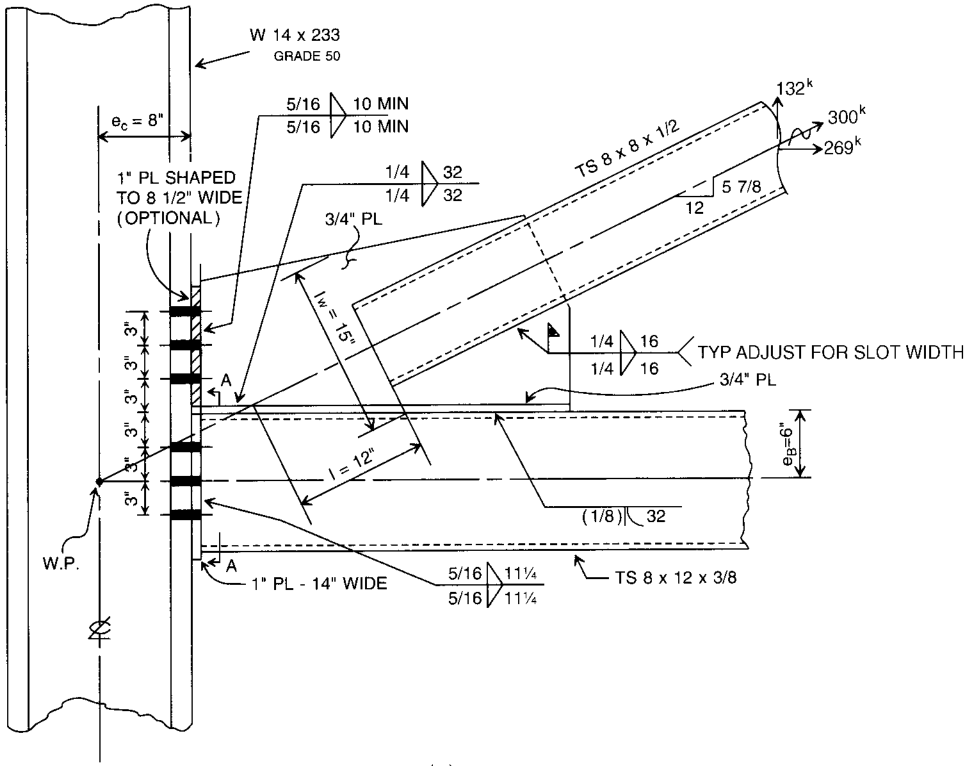
Example—AISC LRFD. Design the bracing connection of Fig. 5.63 for the factored loads shown. The column is to be made of grade 50 steel and other steel components of A36 steel. This connection comprises four connection interfaces: diagonal brace to gusset plate, gusset plate to column, gusset plate to beam, and beam to column. Use the AISC LRFD specification.

Brace to Gusset. The diagonal brace is a tube, slotted to accept the gusset plate and is field welded to the gusset with four ¼-in fillet welds. For a design stress of $0.75 \times 0.60 \times 70 = 31.5$ ksi, a ¹⁄₁₆-in fillet weld has a capacity of 1.392 kips per in of length. Hence the weld length required for the 300-kip load is $300/(1.397 \times 4 \times 4) = 13.9$ in. Use 16 in of ¼-in fillet weld. The ½-in tube wall is more than sufficient to support this weld.

Tearout. Next, the gusset is checked for tearout. The shear tearout area is $= 2 \times 16t$, where t is the gusset thickness. The nominal shear fracture strength is

$$0.6F_uA_v = 0.6 \times 58 \times 32t = 1114t$$

and the nominal tension fracture strength is



(a)

FIGURE 5.63 Details of the connection of a tubular brace to a tubular beam at a column. (*a*) A gusset plate is welded to the brace, to a plate welded to the top of the beam, and to end plates, which are field bolted to the column. (*b*) Section *A–A* through the tubular beam.

$$F_u A_t = 58 \times 8t = 464t$$

Because $1114t > 464t$, the controlling limit state is shear fracture and tension yielding. The design strength for tearout is thus

$$\phi R_{to} = 0.75(1114t + 8t \times 36) = 1052t \geq 300$$

Solution of this inequality gives $t \geq 0.285$ in. Try a gusset thickness of $\frac{5}{16}$ in.

Buckling. For a check for gusset buckling, the critical or "Whitmore section" has a width $l_w = 15$ in, and the gusset column length $l = 12$ in (Fig. 5.63*a*). The slenderness ratio of the gusset column is

$$kl/r = 0.5 \times 12\sqrt{12}/0.3125 = 66.4$$

For the yield stress $F_y = 36$ ksi, the design compressive stress on the Whitmore section (from the AISC LRFD manual, Table 3.36) is $\phi F_{cr} = 24.1$. The actual stress is

$$f_a = 300/(15 \times 0.3125) = 64.0 \text{ ksi} > 24.1 \text{ ksi—NG}$$

Recalculation indicates that a $\frac{3}{4}$-in plate has sufficient strength. For $\frac{3}{4}$-in plate, the slenderness ratio and design compression strength are

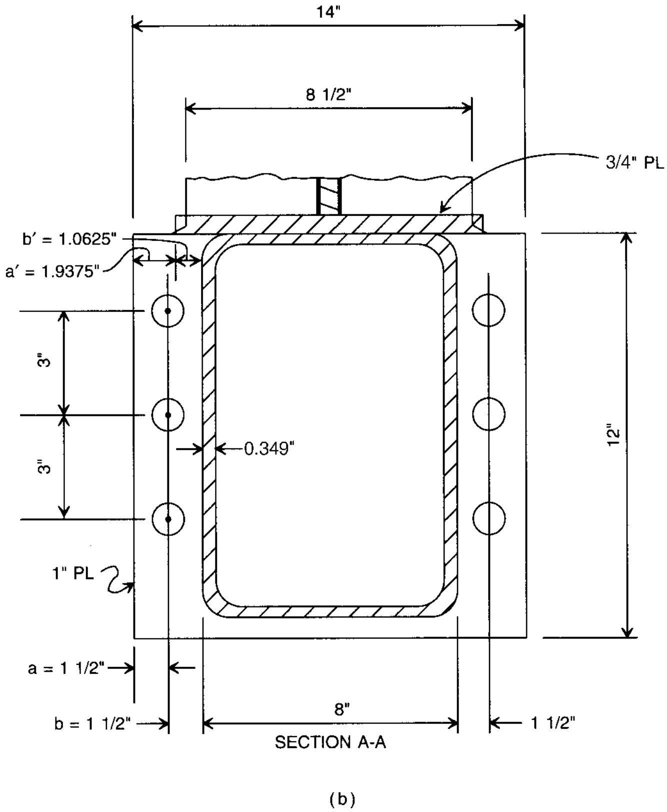


FIGURE 5.63 *Continued.*

$$kl/r = 0.5 \times 12\sqrt{12}/0.75 = 27.7$$

$\phi F_{cr} = 29.4$ ksi and the actual stress is

$$f_a = 300/(15 \times 0.75) = 26.7 \text{ ksi} < 29.4 \text{ ksi—OK}$$

Stress Components. The forces at the gusset-to-column and gusset-to-beam interfaces are determined from the geometry of the connection. As shown in Figs. 5.63*a* and 5.64, for the beam, $e_b = 6$ in; for the column, $e_c = 8$ in, and $\tan\theta = 12/5.875 = 2.0426$. The parameters α and β determine the location of the centroids of the horizontal and vertical edge connections of the gusset plate:

$$\alpha - \beta \tan\theta = e_B \tan\theta - e_c \tag{5.30}$$

This constraint must be satisfied for no moments to exist along the edges of the gusset, only uniform forces. Try $\alpha = 16$ in, estimated, based on the 32-in length of the horizontal connection plate B (Fig. 5.65*a*), which is dictated by geometry. By Eq. (5.30),

$$\beta = \frac{16 - 6 \times 2.0426 + 8}{2.0426} = 5.75 \simeq 6$$

For $\beta = 6$, three rows of bolts can be used for the connection of the gusset to the column.

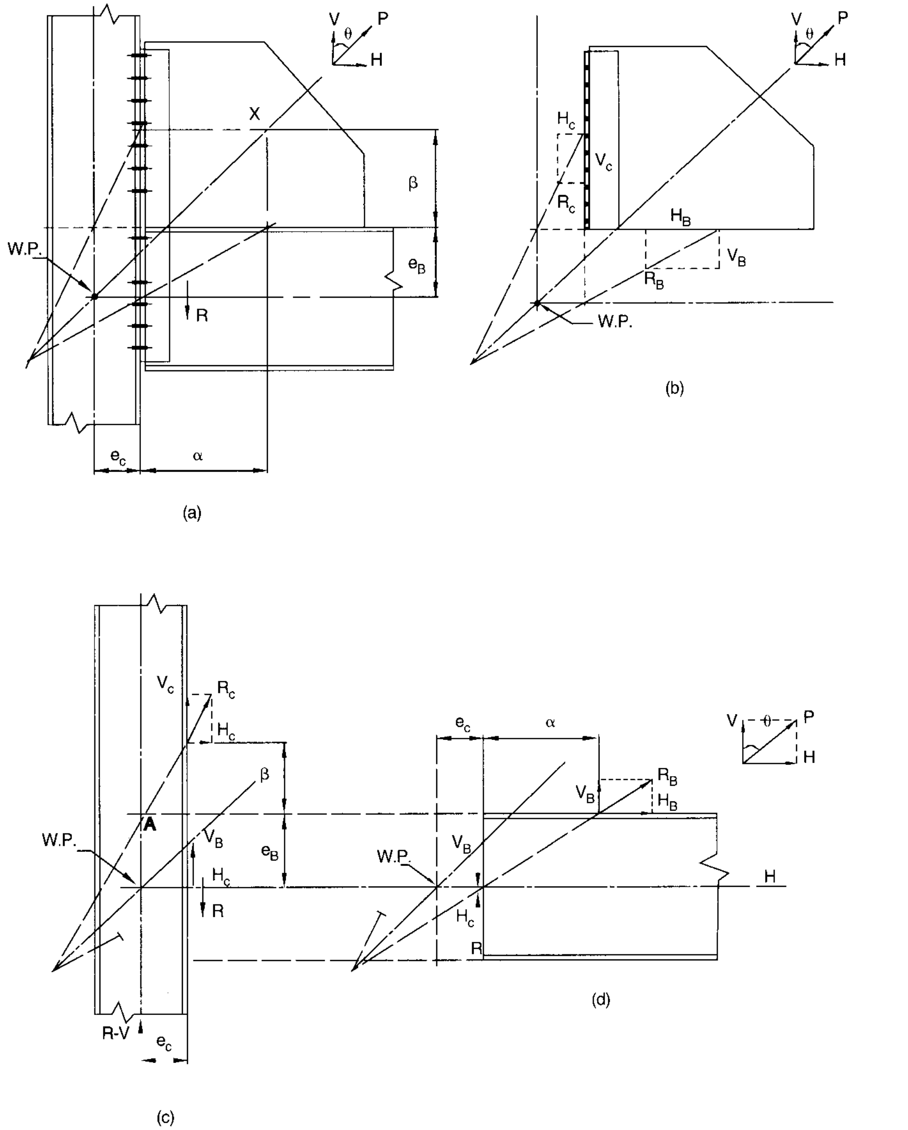
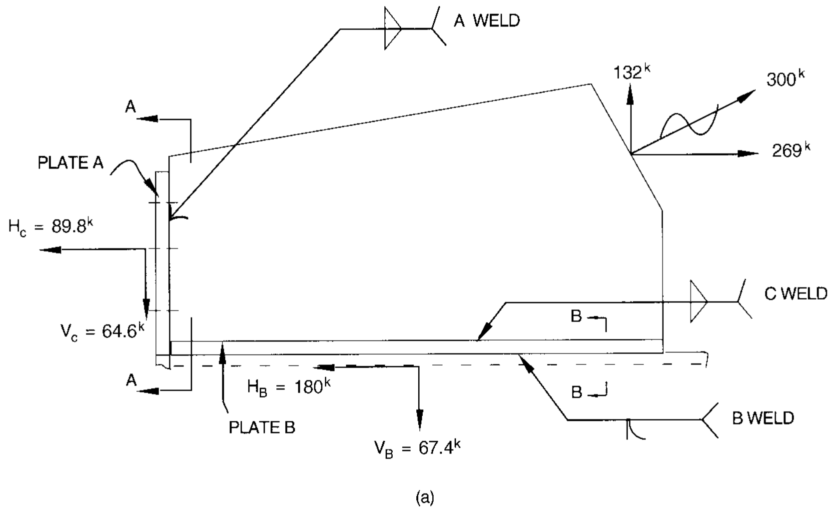


FIGURE 5.64 Determination of forces in a connection of a brace to a beam at a column through a gusset plate, by the uniform force method. (*a*) Lines of action of forces when no moments exist along the edges of the gusset plate. (*b*) Horizontal and vertical forces act at the top of the beam and along the face of the column. (*c*) Location of connection interfaces are related as indicated by Eq. (5.30). (*d*) For an axial force P acting on the brace, the force components in (*b*) are given by: $H_B = \alpha P/r$, $V_B = e_B P/r$, $H_C = e_C P/r$, and $V_C = \beta P/r$, where $r = \sqrt{(\alpha + e_C)^2 + (\beta + e_B)^2}$.



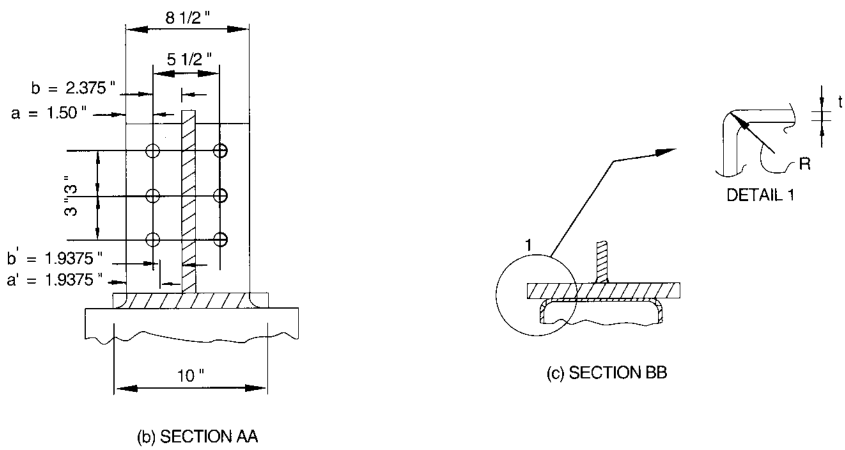


FIGURE 5.65 Gusset plate in Fig. 5.63*a*. (*a*) Force distribution on the gusset. (*b*) Bolt arrangement in plate *A*. (*c*) Attachment of gusset plate *B*.

The distance from the working point *WP*, the intersection of the axes of the brace, beam, and column, to *X* (Fig. 5.64*a*) is

$$r = \sqrt{(16 + 8)^2 + (5.75 + 6)^2} = 26.72 \text{ in}$$

The force components are

$$H_B = 300 \times 16/26.72 = 180 \text{ kips}$$

$$V_B = 300 \times 6/26.72 = 67.4 \text{ kips}$$

$$V_C = 300 \times 5.75/26.72 = 64.6 \text{ kips}$$

$$H_C = 300 \times 8/26.72 = 89.8 \text{ kips}$$

These forces are shown in Fig. 5.65.

Gusset to Column. Try six A325N ⅞-in-diameter bolts. For a nominal strength of 48 ksi, the shear capacity per bolt is

$$\phi r_v = 0.75 \times 48 \times 0.6013 = 21.6 \text{ kips}$$

These bolts are subjected to a shear $V_C = 64.6$ kips and a tension (or compression if the brace force reverses) $H_C = 89.8$ kips. The required shear capacity per bolt is $64.6/6 = 10.8$ kips < 21.6 kips—OK.

The allowable bolt tension when combined with shear is

$$\phi B = 0.75 \times 0.6013(117 - 2.5 \times 10.8/0.6013)$$
$$= 32.5 \text{ kips} \leq 0.75 \times 90 \times 0.6013 = 40.6 \text{ kips}$$

Therefore $\phi B = 32.5$ kips. The applied tension per bolt is $89.8/6 = 14.9$ kips < 32.5 kips—OK.

Prying Action on Plate A. Check the end plate (Plate A in Fig. 5.65) for an assumed thickness of 1 in. Equations (5.2) and (5.3) are used to compute the allowable stress in the plate. As shown in section *A–A* (Fig. 5.65*b*), the bolts are positioned on 5½-in cross centers. Since the gusset is ¾ in thick the distance from the center of a bolt to the face of the gusset is $b = (5.5 - 0.75)/2 = 2.375$ in and $b' = b - d/2 = 2.375 - 0.875/2 = 1.9375$ in. The distance from the center of a bolt to the edge of the plate is $a = 1.50$ in $< (1.25b = 2.9969)$. For $a = 1.50$, $a' = a + d/2 = 1.50 + 0.875/2 = 1.9375$ in; $\rho = b'/a' = 1.9375/1.9375 = 1.0$; $p = 3$ in; and $\delta = 1 - d'/p = 1 - 0.9375/3 = 0.6875$.

For use in Eqs. (5.2) and (5.3), and converting to LRFD format,

$$t_c = \sqrt{\frac{4.44\phi Bb'}{pF_y}} = \sqrt{\frac{4.44 \times 32.5 \times 1.9375}{3 \times 36}} = 1.609 \text{ in}$$

Substitution in Eq. (5.2) yields

$$\alpha' = \frac{1}{\delta(1 + \rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] = \frac{1}{0.6875 \times 2.00}\left[\left(\frac{1.609}{1}\right)^2 - 1\right] = 1.156$$

Since $\alpha' > 1$, use $\alpha' = 1$. Then, from Eq. (5.3), the design strength is

$$\phi T_a = \phi B \left(\frac{t}{t_c}\right)^2 (1 + \delta\alpha') = 32.5 \left(\frac{1}{1.609}\right)^2 (1 + 0.6875)$$
$$= 21.2 \text{ kips} > 14.9 \text{ kips—OK}$$

The column flange can be checked for bending (prying) in the same way, but since the flange thickness $t_f = 1.720$ in $\gg 1$, the flange is satisfactory.

Weld A. This weld (Fig. 5.65*a*) has a length of about 10 in and the load it must carry is

$$P_A = \sqrt{H_C^2 + V_C^2} = \sqrt{89.9^2 + 64.6^2} = 111 \text{ kips}$$

The weld size required in sixteenths of an inch is

$$D = \frac{111}{2 \times 10 \times 1.392} = 3.99$$

A ¼-in fillet weld is sufficient, but the minimum fillet weld permitted by the AISC specifications for structural steel buildings for 1-in-thick steel is 5⁄16 in (unless low-hydrogen electrodes are used). Use a 5⁄16-in weld.

Gusset to Beam. Weld *B* (Fig. 5.65*a*) is 32 in long and carries a load

$$P_B = \sqrt{H_B^2 + V_B^2} = \sqrt{180^2 + 67.4^2} = 192 \text{ kips}$$

This weld is a flare bevel-groove weld that attaches plate *B* to the top corners of the beam (Fig. 5.66). Satisfying ASTM tube specification A500, this shape has a radius $R = 3t$, where t = tube thickness = ⅜ in nominal (Fig. 5.65*c*). The AISC specifications assume that $R = 2t$. Use of this radius to compute weld throat t_e (Fig. 5.66) is conservative for the following reason: The design strength of the flare groove weld, when it is made with E70 electrodes, is given by the product of t_e and the weld length and the design stress, 31.5 ksi. The AWS defines t_e as 5⁄16R when the groove is filled flush (Fig. 5.66).

With $R = 2t$, $t_e = 2 \times 0.375 \times 5/16 = 0.234$ in. The throat required is

$$T_{\text{req}} = \frac{192}{2 \times 32 \times 31.5} = 0.095 \text{ in} < 0.234 \text{ in—OK}$$

Thus use a flush flare bevel-groove weld.

Experience shows that tube corner radii vary significantly from producer to producer, so verify in the shop with a radius gage that the tube radius is $2 \times 0.375 = 0.75$ in. If $R < 0.75$, verify that 5⁄16$R \geq$ (0.095 in ≃ ⅛ in). If 5⁄16$R <$ ⅛ in, supplement the flare groove weld with a fillet weld (Fig. 5.67) to obtain an effective throat of ⅛ in.

Plate B. This plate is used to transfer the vertical load $V_B = 67.4$ kips from the "soft" center of the tube top wall to the stiff side walls of the tube. For determination of thickness t_B, plate *B* can be treated as a simply supported beam subjected to a midspan concentrated load from the gusset plate (Fig. 5.68). The bending moment in the beam is 67.4 × 8⁄4 =

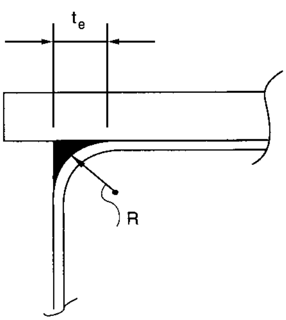


FIGURE 5.66 Effective throat of a flare bevel-groove weld.

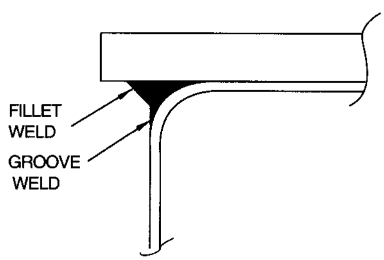


FIGURE 5.67 Fillet weld reinforces a flare bevel-groove weld.

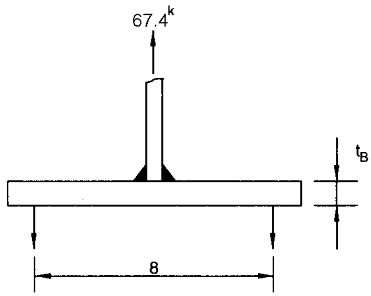


FIGURE 5.68 Plate *B* of Fig. 5.65*a* acts as a simple beam under load from the gusset plate.

135 in-kips. The plastic section modulus is $32t_B{}^2/4$. The design strength in bending for the plate is $0.9F_y = 0.9 \times 36 = 32.4$ ksi. Substitution in the flexure equation ($f = M/Z$) yields

$$32.4 = \frac{135 \times 4}{32t_B{}^2}$$

from which $t_B = 0.72$ in. This indicates that a ¾-in-thick plate will suffice. As a check of this 32-in-long plate for shear, the stress is computed:

$$f_v = \frac{180}{2 \times 32 \times 0.75} = 3.75 \text{ ksi} < 18.4 \text{ ksi—OK}$$

Weld C. The required weld size in number of sixteenths of an inch is

$$D = \frac{\sqrt{180^2 + 67.4^2}}{2 \times 32 \times 1.392} = 2.16$$

But use a ¼-in fillet weld, the AISC minimum.

Beam to Column. Plate *A*, 1 in thick, extends on the column flange outward from the gusset-to-column connection but is widened to 14 in to accommodate the bolts to the column flange. Figures 5.63*b* and 5.69 show the arrangement.

Weld D. This weld (Fig. 5.69) has a length of $2(12 - 2 \times 0.375) = 22.5$ in, clear of the corner radii, and carries a load

$$P_B = \sqrt{H_C{}^2 + V_B{}^2} = \sqrt{89.9^2 + 67.4^2} = 112 \text{ kips}$$

The fillet-weld size required in number of sixteenths of an inch is

$$D = \frac{112}{22.5 \times 1.392} = 3.58$$

Use the AISC minimum ⁵⁄₁₆-in fillet weld.

Bolts: Shear per bolt is $67.4/6 = 11.2$ kips < 21.6 kips—OK. The tensile load per bolt is $89.8/6 = 14.9$ kips. Design tension strength per bolt for combined shear and tension is

$$\phi B = 0.75 \times 0.6013(117 - 2.5 \times 11.2/0.6013) = 31.8 \text{ kips} > 14.9 \text{ kips—OK}$$

Equations (5.2) and (5.3) are used to check prying (bending) of Plate *A*. From Fig. 5.63*b*,

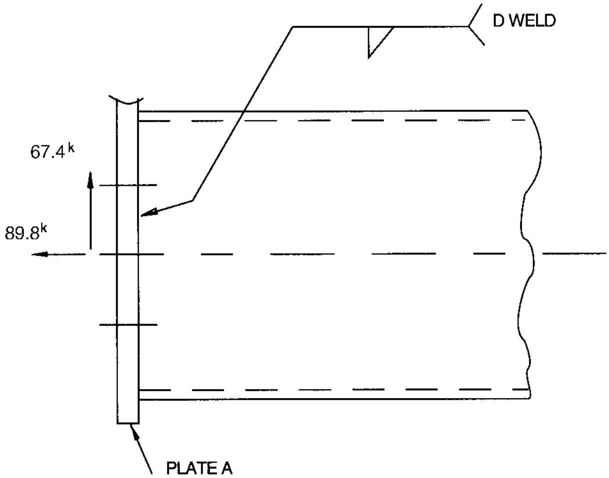


FIGURE 5.69 Plate *A* of Fig. 5.65*a* transmits loads from the beam to the column.

the distance from the center of a bolt to a wall of the tube is $b = 1.5$ in and $b' = b - d/2 = 1.5 - 0.875/2 = 1.0625$ in. The distance from the center of a bolt to an edge of the plate is

$$a = (14 - 11)/2 = 1.5 \text{ in} < 1.25b$$

Also for use with Eqs. (5.2) and (5.3),

$$a' = a + d/2 = 1.5 + 0.875/2 = 1.9375 \text{ in}$$

$$\rho = b'/a' = 1.0625/1.9375 = 0.5484$$

$$p = 3 \text{ in}$$

$$\delta = 1 - d'/p = 1 - 0.9375/3 = 0.6875$$

For use in Eqs. (5.2) and (5.3), and converting to LRFD frormat,

$$t_c = \sqrt{\frac{4.44 \times 31.8 \times 1.0625}{3 \times 36}} = 1.179$$

Substitution in Eq. (5.2) yields

$$\alpha' = \frac{1}{\delta(1 + \rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] = \frac{1}{0.6875 \times 1.5484}\left[\left(\frac{1.179}{1}\right)^2 - 1\right] = 0.3664$$

From Eq. (5.3), the allowable stress is

$$\phi T_a = \phi B \left(\frac{t}{t_c}\right)^2 (1 + \delta\alpha') = 31.8\left(\frac{1}{1.179}\right)^2 (1 + 0.3664 \times 0.6875)$$

$$= 28.6 \text{ kips} > 14.9 \text{ kips—OK}$$

Column Flange. This must be checked for bending caused by the axial load of 89.8 kips. Figure 5.70 shows a yield surface that can be used for this purpose. Based on this yield surface, the effective length of column flange tributary to each pair of bolts, for use in Eqs. (5.2) and (5.3) to determine the effect of prying, may be taken as

$$p_{\text{eff}} = \frac{p(n-1) + \pi b + 2c}{n} \tag{5.31}$$

where n = number of rows of bolts
$b = (8 + 3 - 1.070)/2 = 4.965$
$c = (15.890 - 8 - 3)/2 = 2.445$

$$p_{\text{eff}} = \frac{3(3-1) + 4.965\,\pi + 2 \times 2.445}{3} = 8.83 \text{ in}$$

Also for use with Eqs. (5.2) and (5.3).

$$b' = 4.965 - 0.875/2 = 4.528 \text{ in}$$

$$a = (14 - 11)/2 = 1.5 < 1.25b$$

$$a' = 1.5 + 0.875/ = 1.9375 \text{ in}$$

$$\rho = 4.528/1.9375 = 2.337$$

$$\delta = 1 - 0.9375/8.83 = 0.894 \text{ in}$$

$$t_c = \sqrt{4.44 \times 31.8 \times 4.528/(8.83 \times 50)} = 1.203 \text{ in}$$

$$t = \text{column flange thickness} = 1.720$$

Substitution in Eq. (5.2) gives

$$\alpha' = \frac{1}{0.894 \times 3.337}\left[\left(\frac{1.203}{1.720}\right)^2 - 1\right] = -0.1712$$

Since $\alpha' < 0$, use $\alpha' = 0$, and the design strength is

$$\phi T_a = \phi B = 31.8 \text{ kips} > 14.9 \text{ kips—OK}$$

This completes the design (see Fig. 5.63).

5.37 CRANE-GIRDER CONNECTIONS

Supports of crane girders must be capable of resisting static and dynamic horizontal and vertical forces and stress reversal. Consequently, heavily loaded crane girders (for cranes with about 75-ton capacity or more) usually are supported on columns carrying no other loads. Less heavily loaded crane girders may be supported on building columns, which usually extend above the girders, to carry the roof.

It is not advisable to connect a crane girder directly to a column. End rotations and contraction and expansion of the girder flanges as the crane moves along the girder induce severe stresses and deformations at the connections that could cause failure. Hence, vertical support should be provided by a seat, and horizontal support by flexible connections. These

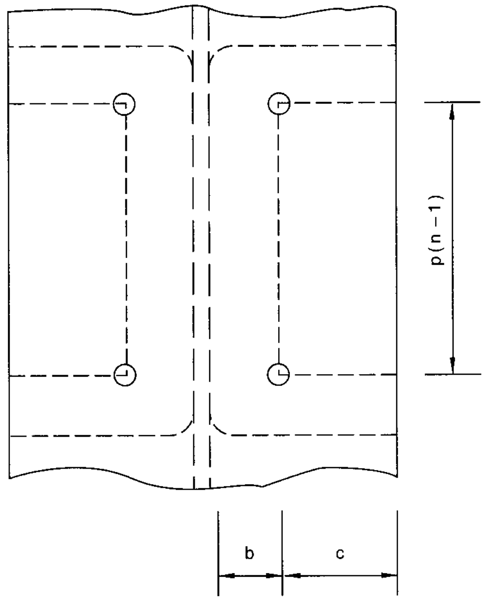


FIGURE 5.70 Yield lines for the flange of the column in Fig. 5.63*a*.

connections should offer little restraint to end rotation of the girders in the plane of the web but should prevent the girders from tipping over and should provide lateral support to the compression flange.

When supported on building columns, the crane girders may be seated on brackets (very light loads) or on a setback in those columns. (These arrangements are unsatisfactory for heavy loads, because of the eccentricity of the loading.) When a setback is used, the splice of the deep section and the shallow section extending upward must be made strong. Often, it is desirable to reinforce the flange that supports the girder.

Whether seated on building columns or separate columns, crane girders usually are braced laterally at the supports against the building columns. In addition, when separate columns are used, they too should be braced at frequent intervals against the building columns to prevent relative horizontal movement and buckling. The bracing should be flexible in the vertical direction, to avoid transfer of axial stress between the columns.

Figure 5.71 shows the connection of a crane girder supported on a separate column. At the top flange, horizontal thrust is transmitted from the channel through a clip angle and a tee to the building column. Holes for the fasteners in the vertical leg of the clip angle are horizontal slots, to permit sliding of the girders longitudinally on the seat. The seat under the bottom flange extends to and is attached to the building column. This plate serves as a flexible diaphragm. Stiffeners may be placed on the building column opposite the top and bottom connections, if needed. Stiffeners also may be required for the crane-girder web. If so, they should be located over the flanges of the crane-girder column.

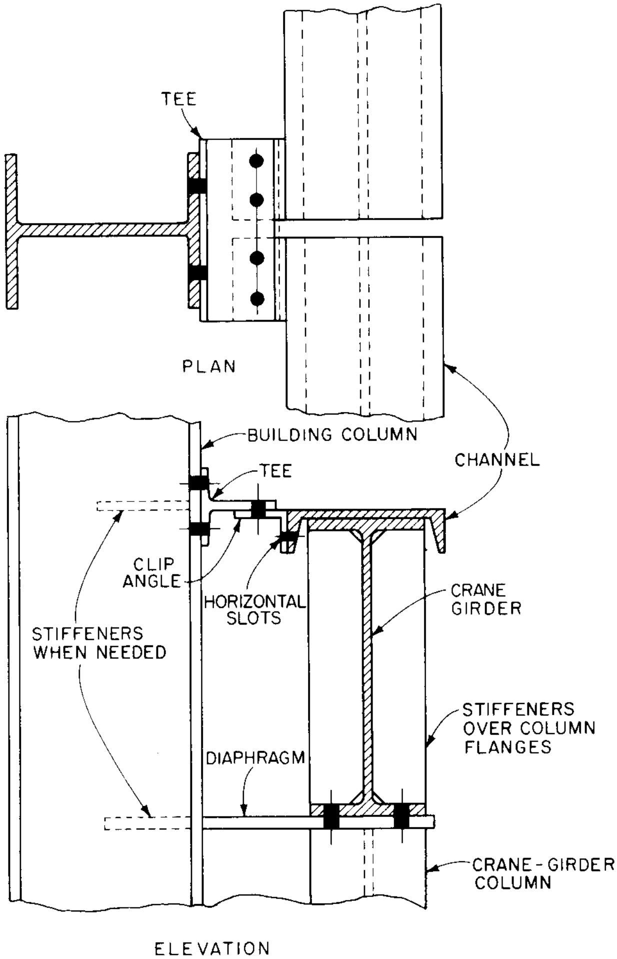


FIGURE 5.71 Crane girder connection at building column.