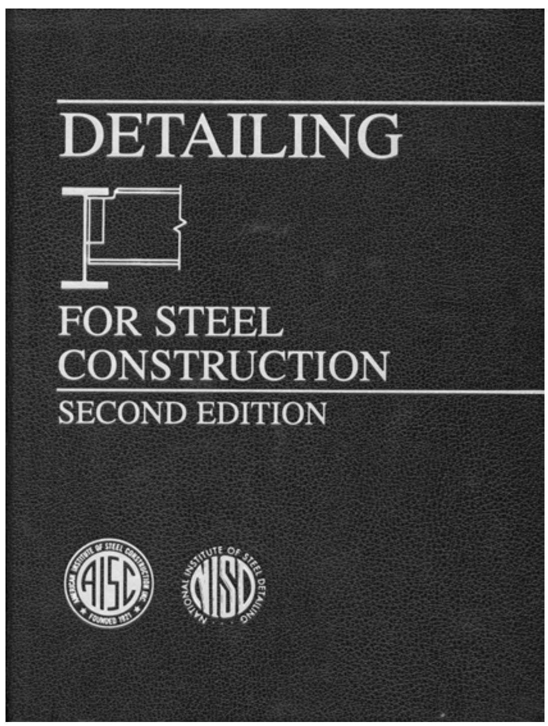
Revision and Errata List, September 1, 2003

AISC/NISD Detailing for Steel Construction, Second Edition

The following editorial corrections have been made in the First Printing, 2002. To facilitate the incorporation of these corrections, this booklet has been constructed using copies of the revised pages, with corrections noted. The user may find it convenient in some cases to hand-write a correction; in others, a cut-and-paste approach may be more efficient.



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The committee also gratefully acknowledges the following people for their contributions to this book: Michel Cloutier, Harry A. Cole, Timothy Egan, Areti Gertos, Louis Geschwindner, Keith Grubb, John L. Harris, Chris Harms, Cynthia Lanz, Keith Mueller, Janet S. Tuegel, Jerry Loberger, Thomas J. Schlafly, William Segui, Mark A. Snyder, Ramulu S. Vinnakota, and John Wong.

must know what size and weight limits the erector can handle at the job site. The steel detailer obtains erection information from the fabricator or erector. The customary practice for obtaining answers to questions about design information is for the fabricator to send inquiries to the owner's designated representative for construction (usually the general contractor), who then submits them to the owner's designated representative for design (normally the structural engineer of record, through the architect). Sometimes a direct communication avenue is permitted between the steel detailer and the structural engineer of record and the fabricator, general contractor and architect are kept aware of the questions and answers. As time is generally critical for the fabricator, this system speeds the process whereby the steel detailer can have design information clarified. Also, it allows the structural engineer of record and the steel detailer to communicate in terms familiar to each other, resolve confusion regarding a question and avoid a back-and-forth string of misunderstandings and unclear or partial answers. A sense of teamwork by and cooperation amongst the parties mentioned above is an essential ingredient to the successful completion of a project.

RAW MATERIAL

The fabrication shop, where structural steel is cut, punched, drilled, bolted and welded into shipping pieces for subsequent field erection, does not produce the steel material. The steel is produced at a rolling mill, normally from recycled steel, and shipped to the fabrication shop. At this stage the steel is referred to as raw material. The great bulk of raw material can be classified into the following basic groups:

- Wide-Flange Shapes (W) used as beams, columns, bracing and truss members.
- Miscellaneous Shapes (M), which are lightweight shapes similar in cross-sectional profile to W shapes.
- American Standard Beams (S).
- Bearing Pile Shapes (HP) are similar in cross-sectional profile to W shapes, have essentially parallel flange surfaces and have equal web and flange thickness. The width of flange approximates the depth of the section.
- American Standard Channels (C).
- Miscellaneous Channels (MC), which are special purpose channel shapes other than the standard C shapes.
- Angles (L), consist of two legs of equal or unequal widths. The legs are set at right angles to each other.
- Structural Tees (WT, MT and ST) made by splitting W, M and S shapes, usually along the mid-depth of their webs. The Tee shapes are furnished by the producers or cut from the parent shapes by the fabricator.
- Hollow Structural Sections (HSS) are available in round, square and rectangular shapes.
- Steel Pipe is available in standard, extra strong and double-extra strong sizes.
- Plates, Bars or Flats (PL, Bar, FL) are rectangular pieces used as connection material. While some fabricators make connection pieces using automated equipment to cut plates to the necessary size, other fabricators use Bars or Flats with predetermined widths. The detailer should check with the fabricator to determine their shop practices and list the proper material on the drawings. Bars are limited to maximum widths of 6 or 8 in., depending on thickness; plates are available in widths over 8 in., subject to thickness and length limitations. Rev. 9/1/03

A clear understanding of the various forms and shapes in which structural steel is available is essential before the steel detailer can prepare shop and erection drawings. The *AISC Manual of Steel Construction Load and Resistance Factor Design (LRFD) 3rd Edition* (referred to hereafter as the Manual) Part 1 lists all shapes commonly used in construction, including sizes, weights per foot, dimensions and properties, as well as their availability from the rolling mill producers. Figure 1-1 (in this chapter) shows typical cross - sections of raw material. Note that S, C and MC shapes are characterized by tapered inner flange surfaces and W shapes have parallel inner and outer flange surfaces. M shapes may have either parallel or tapered inner surfaces of the flanges, depending on the particular section and the producer. For details of this nature refer to the Manual or producers' catalogs.

Plates are defined by the rolling procedure. Sheared plates are rolled between rolls and trimmed (sheared or gas gut) on all edges. Stripped plates are furnished to required widths by shearing or gas cutting from wider sheared plates.

Hollow Structural Sections are rectangular, square and round hollow sections manufactured by the electric-resistance welding (ERW) or submerged-arc welding (SAW) methods. These sections allow designers and builders to produce aesthetically interesting structures and efficient compression members. They are used as columns, beams, bracing, truss components (chords and/or web members) and curtain wall framing. In addition to the Manual, the steel detailer should refer to the *AISC Hollow Structural Sections Connections Manual*, which provides guidance in developing connections for HSS.

Figure A1-2 (Appendix A) has been prepared to show the customary methods of designating and billing individual pieces of structural shapes and plates on shop drawings, the conventional way of picturing these shapes and the correct names of their component parts. This system is generally accepted and used by steel detailers, although some minor deviations may occur when trade name or proprietary designations are substituted for certain "Group Symbols" listed in the billing material. Figure A1-2 should be studied carefully.

- Prepare system of assembling and shipping piece marks
- Enter and check base grid system
- Enter and check columns with base plate data
- Enter and check beams and other structural members
- Prepare advance bills for ordering material
- Produce and check Anchor Rod Setting Plan
- Enter and check connections
- Generate clash check
- Produce and check column and beam details etc.
- Submit for approval
- Revise details per approval comments
- Submit to fabricator for production
- Generate field bolt list

The operating data sheet shown in Figure 2-2 indicates that the information required by the steel detailing group is presumed to be shown on the design drawings, Drawings S-1 thru S-14. This information and the supplementary data described in the job specifications should be complete and final. However, to verify this assumption the drafting project leader assigned to the contract must study the design drawings and specifications carefully. This will reduce time lost later in obtaining missing information, which could seriously delay the progress of the work.

In this project the selling basis is lump-sum and, unless otherwise advised by the sales department, the drafting project leader can assume that all of the required framing is covered on the design drawings. Later, the owner's designated representative for design may issue revised and supplementary design drawings amplifying and clarifying information shown on the original-issue design drawings. Any change in the scope of work may require an adjustment of the contract price. In such a case the detailing group must obtain instructions from the sales department or project manager before proceeding with the work.

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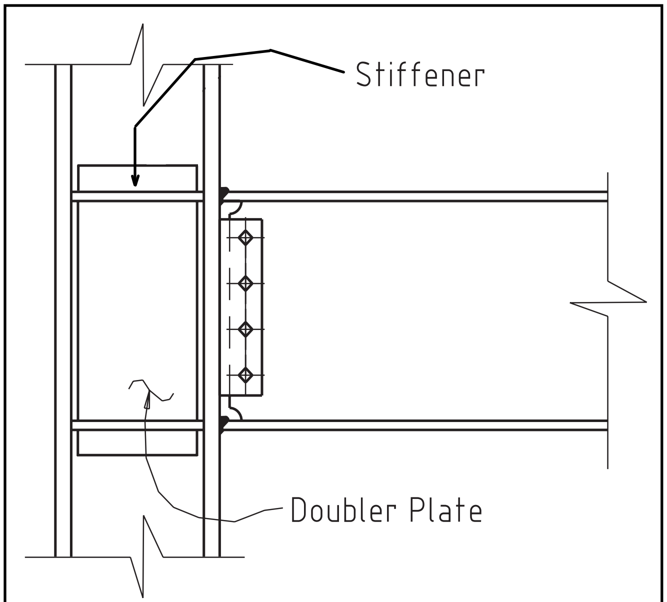

Figure 2-5. Typical moment connection.

CONTRACT DOCUMENT ERRORS

As indicated above the detailing manager must study the design drawings, subsequent revisions and pertinent specifications as soon as they are received by the steel detailing group for use in preparing shop drawings and all the relative documents for the fabricator. The steel detailing group must become familiar with the details of the project.

The accuracy of the contract documents is the responsibility of the owner's designated representative for design. Section 3.3 of the *AISC Code of Standard Practice* requires that design discrepancies be reported when discovered, but does not obligate the fabricator or the steel detailer to find the discrepancies.

One of the more common problems found on drawings produced by computer programs is the connection of a deep beam to a much shallower supporting beam. For instance a W24 may be shown connecting to the web of a W16 with the tops of both beams at the same elevation ("flush top"). This may result in an expensive connection for the W24 to the W16, involving possible reinforcement of the web of the W24 and/or the W16. Such a situation should be brought to the attention of the owner's designated representative for design to determine if a deeper, more suitable beam could be substituted for the W16.

Sometimes, the sum of a string of dimensions on drawings does not agree with the given overall (total) dimension. At other times dimensions are omitted. Another error commonly found on drawings is incorrectly described material sizes.

On some projects the specifications issued are similar to those used on a previous project by the designer. Thus, some references to products and regulations that were job-specific on the previous project may not be applicable to the present project. Another problem occurs in specifications when they differ from information on the design drawings. The Code stipulates that design drawings govern over the specifications. Again, when these discrepancies are found, they must be referred to the design team for resolution.

When beam-to-column flange moment connections are required on a project, often column webs must be reinforced with transverse stiffeners and/or web doubler plates, which can be expensive. The designer may show only a sketch of a typical moment connection (see Figure 2-5, for example), illustrating such stiffening in the web of the column. The steel detailer should note that the *AISC Code of Standard Practice*, Section 3.1 requires that doubler plates and stiffeners "shall be shown in sufficient detail in the structural

Edge Distances

One of several factors to be considered in determining the bearing strength at bolt holes is the minimum distance (measured in the direction of a transmitted force) from the center of a standard hole to the free edge of a connected part. (See AISC Specification Section J3.4.) In the Specification, Table J3.4 lists applicable values of minimum edge distances for the commonly used bolt sizes. The edge distance is increased by an increment listed in Table J3.6 when oversized holes and short- and long-slotted holes comprise the connection. Maximum bearing strengths at bolt holes are established by AISC Specification Section J3.10. Edge distances may be increased to provide for a required bearing strength.

AISC Specification Section J3.5 establishes the maximum distances from the center of bolt holes to the nearest edge of parts in contact. The limits specified are intended to provide for the exclusion of moisture, thus preventing corrosion between the parts.

Snug-Tightened and Pretensioned Bearing Connections

In bearing joints the bolt shear load is resisted by the bolt bearing against the sides of the holes in the connected material. The clamping force of the high-strength bolts contributes to the connection rigidity, but the shear load is not considered to be resisted by the friction between the connected parts. High-strength bolted bearing connections are used when the slip between joint surfaces, necessary to bring the bolts into bearing in holes in the connected material, can be tolerated.

Faying surfaces must be free of loose mill scale, dirt and other foreign material. However, the presence of paint, oil, lacquer or galvanizing is not prohibited. Burrs that would prevent solid seating of the connected parts in the snug tight condition must be removed. Snug-tight is defined as the tightness that exists when the plies of the joint are in firm contact. Usually, this may be attained by a few impacts of an impact wrench or the full effort of a worker using an ordinary spud wrench.

Two shear strength levels are provided for each type of high-strength bolt in a bearing connection. AISC Specification Table J3.2 lists nominal shear strengths (F_v) values for A325 and A490 bolts in bearing connections. Full advantage of bearing connections can only be realized when bolt threads are not allowed to cross a shear plane (X-type). When bolt threads cross a shear plane, the lower F_v values must be used (N-type).

Thread lengths on high-strength bolts are made shorter than those on other types of bolts for the express purpose of assuring a minimum encroachment of threads into the grip (total thickness of all plies of connected material). Lengths of thread for high-strength bolts may be found in Figure C1 of the Commentary on the RCSC Specification. Note, however, when A325T bolt lengths equal to or shorter than four times the nominal diameter are used, the bolt is threaded for the full length of the shank per ASTM Specification A325 Supplement S1. Rev. 9/1/03

For the convenience of the steel detailer, fabricators usually provide tabulations of bolt lengths for various diameters and grips. These tabulations take into account washer and nut thicknesses and allow for full thread engagement of the nut. The tabulated bolt lengths are satisfactory for bolts in slip-critical connections. However, if the higher F_v values permissible in bearing connections are to be used, required bolt lengths must be such that the amount of thread within the grip is less than the thickness of the thinner outside joint component.

The steel detailer is referred to the Manual, Part 7, Figure 7-1, which depicts bolt installation to exclude threads from the shear plane. If a bolt must be inserted so that the thickness (t) of the ply closest to the nut is less than that tabulated, the steel detailer must furnish a longer bolt and provide an additional hardened washer(s) under the nut. The extra washer(s) will permit full tightening of the nut. Where threads must be excluded from the shear plane, the steel detailer must give explicit instructions on the erection drawings to control the field installation of bolts.

Bolts, in connections that are neither within the slip-critical category nor required to be pretensioned bearing connections, shall be installed in properly aligned holes and need only be tightened to the snug tight condition. If the Owner's Designated Representative for Design designates in the contract documents that certain shear/bearing connections are to be tightened to pretension, such bolts shall be installed and tightened as if they were slip-critical. (See RCSC Specification Section 8(c)).

Slip-Critical Connections

Joints in which, in the judgment of the owner's designated representative for design, slip would be detrimental to the behavior of the joint are defined as slip-critical. In slip-critical joints the bolt shear is resisted by friction between the contact surfaces of the connected parts, but must also be designed to resist shear and bearing on the bolts, since limited slip may occur in the service life of the structure. These joints include, but are not necessarily limited to, joints subject to fatigue or significant load reversal, joints with bolts in oversize holes or in slotted holes with the applied force approximately in the direction of the long dimension of the slots and joints in which welds and bolts share in transmitting shear loads at a common faying surface. The owner's designated representative for design must designate in the contract documents, which joints are to be slip-critical as described in RCSC Specification Section 4.

mandatory, it is the basis of many fabricators' standard system of connection angles. Advantages lie in simplicity of detail and fabrication. Refer to the Manual, Part 10 for examples of selecting double angle connections using the published tables.

Shear End-Plate Connections

The primary use of the end-plate is to resist gravity load. To assure the rotational flexibility required in the design of these connections, plates in the ¼-in. to ⅜-in. thickness range are used. An end-plate has good resistance to axial compression in the beam. Usually, however, in the thickness range noted above it is unsuitable in resisting axial tension. Plates that are to resist axial tension should be designed by the owner's designated representative for design. Depending on the length of the plate, it offers fair resistance to torsion loads. Normally, the plate is fillet-welded to the beam end (see Figure 3-7). It can be either field bolted or field welded to the supporting member. If field welding is chosen, erection bolt holes should be provided.

The main objection of some fabricators to this connection is that the beam must be cut square on both ends and to accurate length. Other fabricators, however, are equipped to square-cut beams accurately and favor using end plates. This connection does not handle beam camber well unless the connection is a very shallow end-plate. Sometimes, the beams are purposely detailed and fabricated short for erection purposes and must be shimmed, when required, to maintain the desired building dimensions.

End-plate connections are used mainly on filler beams, but can be used to connect beams to columns (see Figure 3-8) and can be adapted easily for use with skewed members. The plate can be punched with either standard holes or horizontal short slots.

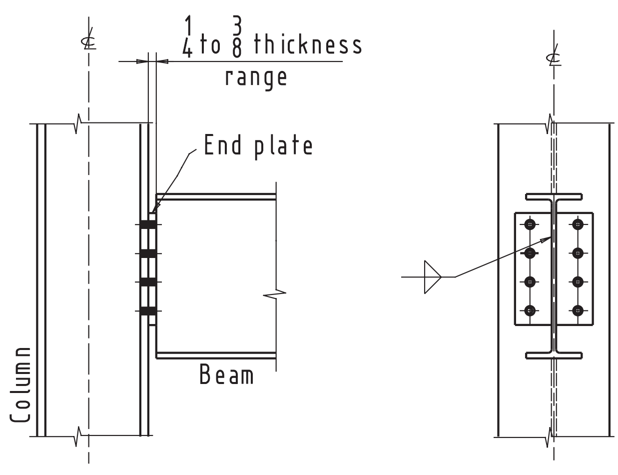

Figure 3-8. Typical end-plate connection to a column flange.

The steel detailer is referred to the Manual, Part 10 for further information, an example and Table $\boxed{10\text{-}4}$ which are to be used in selecting an end-plate connection. Rev. 9/1/03

Seated Beam Connections

Seated beam connections consist of two basic types: (1) unstiffened seated connections and (2) stiffened seated connections.

Seated connections are suited particularly for connecting beams to column webs, primarily because of the ease of beam erection and their inherent safety features. They may be used, also, to connect beams to supporting girder beams when the girder beams are deep enough to accommodate the seat. Although seldom applied, seated connections may be used to connect beams to column flanges, provided they do not create interference with fireproofing or other architectural finishes. Tables 10-5 through 10-8 in the Manual, Part 10 provide design data for these types of connections.

Seated beam connections have the following advantages over double angle connections:

- They permit fabrication of plain punched beams.
- They afford the erector a means of landing the beam while aligning the field holes and inserting erection or permanent bolts.
- They result in better erection clearances when a beam connects to a column web (for example see Figure 3-9). As the overall length of an unframed punched beam is less than the back-to-back distance of connection angles for a framed beam, the beam can be lowered more easily into the trough formed by the flanges of the supporting column. This feature was discussed under "Clearance for Field work" in Chapter 1.
- In the case of larger size beams, seated connections reduce the number of field bolts to be installed, resulting in an overall economy.

In the design of a seated connection the end reaction from the beam is assumed to be delivered to the seat angle or seat plate. The top (or "cap") angle is added to provide lateral support at the top flange of the beam. Alternatively, if attaching the angle to the top flange is unsuitable, it can be connected to the beam web as close to the top flange as possible. This angle is not required to resist any calculated shear or moment at the end of the beam. It can be relatively small and need have no more than two bolts in each of its legs, even in the case of large, heavy beams. However, this angle is required for stability and satisfactory performance of a seated connection. Customary practice is to ship top angles loose (Not permanently attached to the beams or columns) for subsequent field installation after plumbing the structure.

Examples of all-bolted unstiffened seated connections are provided in the Manual, Part 10.

Referring to step (1), the choice of seated beam Connection Type (A, B, etc.) is, to some extent, determined by the supporting structural member. A deep connection similar to Connection Type B may not be practical for beam-to-girder framing because of the limited depth of the girder. In the case of a seated connection to the web of W8 and W10 columns, a seat length of 6 in. is generally used, and for webs of W12 and larger columns the 8 in. seat length commonly is used.

Referring to step (2) above, as noted previously in "Shear in High-Strength Bolted Connections", the bearing strength of the connected material with respect to the fasteners should be investigated in all shear connections. In many cases calculating the bearing strength may not be necessary, as the bearing strength may be obviously greater than the shear strength. Generally, this is the case when a connection is only on one side of the supporting member. However, the bearing strength always should be considered, even if it is not calculated.

Stiffened Seated Connections

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When a beam reaction exceeds the outstanding leg strength of an unstiffened seated beam connection as listed in Table 10-6, a stiffened seated beam connection may be used. In such conditions stiffeners are fitted to bear on the underside of a seat plate (see Figure 3-11).

Table 10-7 in the Manual, Part 10 gives design strengths of the outstanding legs of two stiffener angles having 3½-in., 4-in. and 5-in. outstanding legs with a thickness range of 5/16-in. to ¾-in. inclusive. Two tables are shown, one for steel with F_y = 36 ksi and the other for steel with F_y = 50 ksi.

To allow for clearing the welds connecting the seat plate to the supporting member, the effective width of the stiffener angles used in determining the bearing strengths in Table 10-8 is assumed to be ¾-in. less than the actual width of the outstanding leg. For example the design bearing strength of a 3½-in. wide and 3/8-in. thick stiffener angle of A36 steel is computed as follows (see AISC Specification Section J8).

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One stiffener:

$$\phi R_n = \phi \times 1.8 \times F_y \times A_{pb}$$

$$\phi R_n = 0.75 \times 1.8 \times 36 \times \tfrac{3}{8}(3\tfrac{1}{2} - \tfrac{3}{4}) = 50.1 \text{ kips}$$

Two stiffeners:

$$\phi R_n = 50.1 \times 2 = 100.2 \text{ kips}$$

100 kips is the strength listed in Table 10-7.

The minimum thickness of seat plates used with these stiffeners is 3/8-in. The plate is made at least wide enough to

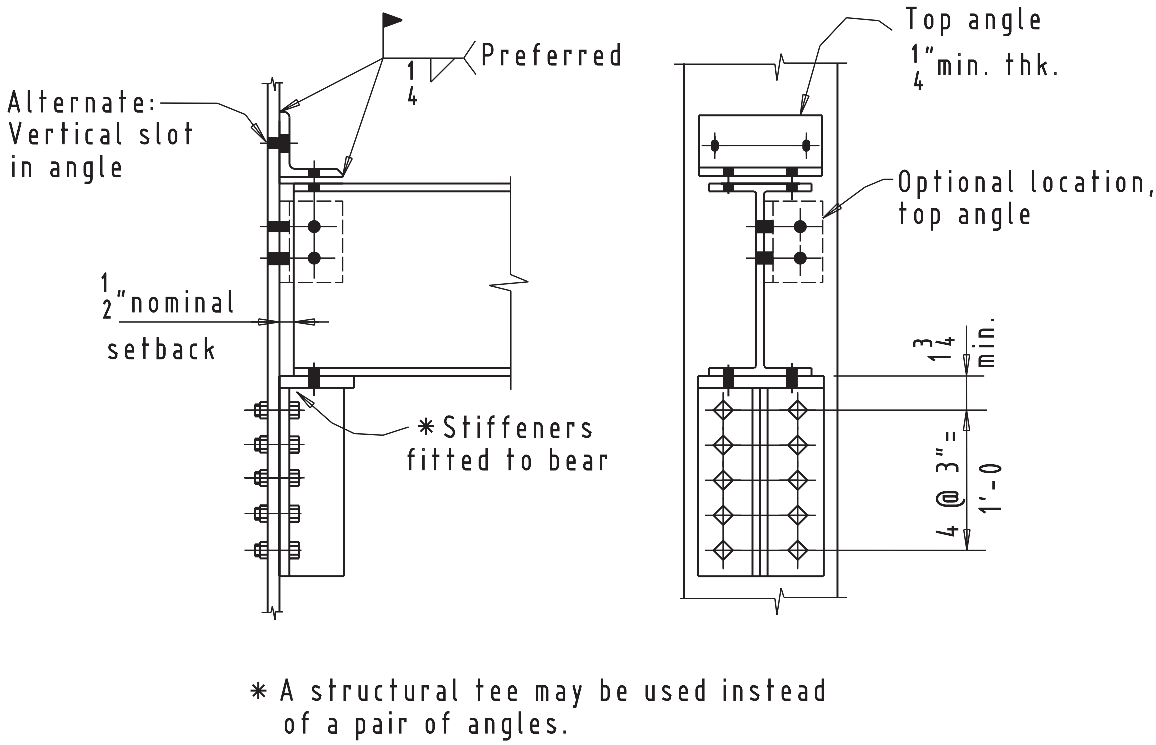

Figure 3-11. Stiffened seated connection.

5. Determine size of top angle.

The seat plate should be field attached to the supported beam with high-strength bolts.

Single-Plate Connections

A single-plate connection is one with a plate shop welded to a supporting member and field bolted to the supported member. The supporting member almost always will either be a beam (Figure 3-12) or a column (Figure 3-13). This type of connection is not suitable for connecting to the web of a column because of difficulty in entering the bolt, reaming (if required) and pretensioning the bolt. Because the web bolts are in single shear, the connection requires more bolts than a double angle connection. However, single-plates are economical to fabricate and are safe to erect in practically all configurations. When combined with reasonable end-reaction requirements, they can be used extensively to simplify construction.

Primarily, the single-plate connection is used to support gravity shear load. It is capable of resisting both axial compression and tension loads in the beam. The single-plate may not be suitable for resisting torsion loads and should be used with caution if the beam lacks lateral support of its compression flange. It has moderate rigidity depending to some extent on the depth of the connection, i.e., the number of bolts and their size. Either standard holes or horizontal short slots can be punched in the plates.

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Other advantages to this type of connection are that it can be adapted for skews and for sloping connections. The steel detailer is referred to the Manual, Part 10 for information, examples and Table 10-9 to be used in selecting single-plate connections.

Single-plates used in simple shear connections of beams to column webs where the column web is stiffened (the result of a beam-to-column flange moment connection) require special consideration. In this case the field connection must be made clear of the edges of the column flanges to provide for access and the ability to erect the beam. The extension of the connection beyond normal gage lines results in an eccentric moment. As the resistance of the column to weak-axis bending is considerably less than that in the strong axis, the eccentric moment in this type of connection must be considered. Similarly, eccentricities larger than normal gages also may be a concern in connections to girder webs. The design of these types of connections is treated in the Manual, Part 10.

Single-Angle Connections

Single-angle connections are used in applications similar to the single-plate connection and provide distinct erection advantages over a double angle connection. Its primary function is to resist gravity shear loads. The single-angle is one of the most flexible of all the "simple" connections. Like the single-plate, it is economical to fabricate and safe to erect in virtually all configurations. When used with reasonable end-reaction requirements, the single-angle can be used extensively to simplify construction. It has poor torsion resistance and should not be used in cases where it

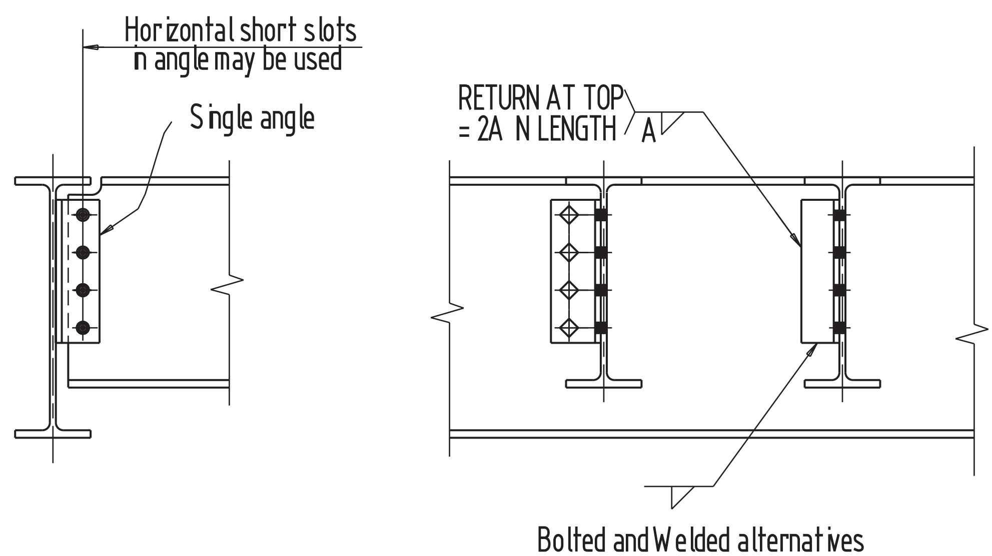

Figure 3-14. Single angle connection.

COMMON WELDED SHEAR CONNECTIONS

Simple welded framed and seated beam connections are of the same types as those used in bolted fabrication discussed earlier in this chapter. They may be shop bolted and field welded, shop welded and field bolted or shop and field welded. Figure 3-22 illustrates these three combinations in a framed connection. When permanent field connections are made with welds, open holes are provided for erection bolts. A minimum of two erection bolts is required, but more may be necessary depending on the beam size and the specific framing conditions. Shop assembling and shop welding of fitting material is normally performed without using temporary bolts. In the shop, framing angles and other fittings are held temporarily in place with clamps or fixtures, and then are tack welded in final position prior to completing the specified shop welds.

Welded connections are especially suitable for use with HSS. The design of HSS connections is addressed in the AISC Hollow Structural Sections Connections Manual.

Double Angle Connections

Three basic welded connections can be designed for framed beams:

Case I: Connection angles are welded to the beam web and are bolted to the supporting member (Figure 3-23a). Usually, welds are made in the shop.

Case II: Connection angles are welded to the supporting member and are bolted to the beam web (Figure 3-23b). Note in Figure 3-23b the bottom flange of the beam is shown cut (coped) to permit erection of the beam. In the case of beam-to-beam framing at least one of the welds, B, must be made in the field to permit swinging the beam into final position.

Case III: Connection angles are welded both to the beam web and the supporting member (Figure 3-23c). (For ease and safety in erection Case III connections usually are provided with erection bolt holes.)

Tables 10-2 and 10-3 in the Manual, Part 10 list the weld design strengths in kips for a wide range of flexible welded framed connections.

Table 10-2 refers to Cases I and II. The connections have been developed specifically to give weld design strengths for several lengths of connection angles and weld sizes suitable for combination with Table 10-1, the Manual, Part 10. Table 10-3 has been developed for Case III, in which the connections are welded completely in the shop and field. In all of these tables the design weld strength in kips is given for weld A and weld B for several lengths *L* of framing angle and for several sizes of weld.

When fasteners are used in combination with the welds listed in Table 10-2 (Cases I and II), the design strength of the number of fasteners selected can be investigated by referring to Table 10-1 or by direct calculation. The design strength of the fastener group must be equal to or greater than the factored load to be carried.

The design strengths of weld A and weld B, selected from Tables 10-2 or 10-3, must be compared and the controlling value used.

To provide flexibility in the welded framed beam connections listed, the framing angles are limited in these tables. This also provides an acceptable degree of flexibility for Case I and Case II connections.

In Table 10-2 the angle thickness must be (1) equal to the weld size plus 1/16-in. or (2) equal to the thickness taken from the applicable Table 10-1, whichever is greater.

In general 2L4×3½ will accommodate usual gages with the 4-in. leg attached to the supporting member. Design strengths are based on angles of F_y = 50 ksi steel.

For all-welded double-angle connections (Table 10-3) the minimum angle thickness equals the weld size plus 1/16-in. The length to be used is as tabulated. Use 2L4×3 for lengths equal to or greater than 18 in.; use 2L3×3 for others. Design strengths for these connections are based on use of F_y = 50 ksi steel.

In both Case I and III connections weld A, connecting the angles to the beam web, is loaded eccentrically. The design strength of the weld group is determined using the "instantaneous center of rotation" method described in the Manual, Part 7. For a discussion of this method and of development of Tables 10-2 and 10-3, the steel detailer should read the Manual, Parts 7, 8 and 10. The steel detailer should use Table 10-2 for welds in Case I and II connections and Table 10-3 for Case III connections.

In Table $\boxed{10\text{-}2}$ covering combinations of welded or bolted connections, the design strength of weld A (Case I) is based on 3-in. long welds at the top and bottom of the angles and welds of the same size along the entire length of the angles. The 3½-in. wide angle web leg permits the use of a standard 2¼-in. gage if fasteners are used. However, to provide a more economical connection the width of the web leg can be reduced from 3½ in. to 3 in. when welds are used and the value determined from Table 10-3. Rev. 9/1/03

In Table 10-3 the angle leg against the beam web has been limited to 3 in. Allowing for a ½-in. setback of the beam end, the design strength of this weld is based on 2½-in. long welds at the top and bottom of the angles and welds of the same size along the entire length of the angles.

The distribution of forces in weld B, which connects the angles to the supporting beam, results in tension at the top of the weld. As a safeguard against the initiation of a crack, the vertical welds at the top of the angles are returned horizontally for a length of twice the weld size. This return is not included in the effective length of the weld in computing the strengths listed in Tables 10-2 and 10-3.

Rev. 9/1/03 The design strengths of weld B in Table $\boxed{10\text{-}3}$ are based on 4-in. wide outstanding legs. This width will accommodate the standard 5½-in. gage center-to-center of open holes customarily used for bolts in Case I. When Case II is used, the outstanding legs may be reduced to 3 in. for lengths up to, but excluding, 18 in. and the value for B welds interpolated from Table 10-3. Conservative results will be provided in this range.

In Tables 10-2 and 10-3 the minimum thickness of beam web required to develop the design strengths of weld A are given below the respective F_y values for each steel. If the actual beam web thickness is less than those listed, the design strength of weld A must be reduced by multiplying it by the ratio of actual thickness to tabulated minimum thickness. Thus, from Table 10-3 if ¼-in. weld A, with a design strength of 134 kips and an 8-in. long angle, is considered for a W14×43 beam (web thickness = 0.305 in.) of F_y = 36 ksi steel, the design strength must be multiplied by 0.305/0.57, giving 72 kips.

The tabulated minimum thickness for weld A is calculated by equating the shear strengths per in. of the weld and base materials as follows:

$$2 \times 1.392 \times D = 0.75 \times (0.60 \times F_u) \times t_{min}$$

$$t_w = \frac{2 \times 1.392 \times D}{0.90 \times 0.60 \times F_y} = \frac{5.16 \times D}{F_y}$$

where

t_w = minimum web thickness, in. and

D = weld leg size, in.

The multiplier "2" represents the number of weld lines.

A similar limitation applies to weld B. When welds line up on opposite sides of a support, the minimum support thickness is the sum of the thicknesses required for each weld. In either case when less than the minimum material is available, the tabulated weld design strength must be reduced by the ratio of the thickness provided to the minimum thickness required.

Other restrictions on weld size concern the minimum and maximum size of fillet weld permitted on various thicknesses of material as stipulated in Specification Section J2.2b.

The design strengths of welds listed in Tables 10-2 and 10-3 are based on the use of E70 electrodes. If E60 electrodes are used, multiply the tabular values by 60/70 or 0.86.

Designs of Double Angle Connections

Following are step-by-step procedures for the determination of the three basic kinds of welded construction for framed beams.

Cases I and II:

Determine required factored beam end reaction.

1. From Table 10-1 of the Manual, Part 10, determine the number of fasteners and the length and thickness of connection angles.
2. From Table 10-2 using the length of angle determined above, select the weld size required.
3. Check if the angle thickness obtained from step 1 above can accommodate the weld size; increase the thickness, if necessary.
4. For weld A note the minimum web thickness required and reduce the tabulated design strength of the weld if the thickness of beam web is less than the minimum. For weld B investigate the design strength of the supporting material to receive the weld force.

Size of Connection Angles:

1. In general for Cases I and II use 4 × 3½ angles with 3½ in. web legs and 4 in. outstanding legs.
2. The width of web leg in Case I may be reduced optionally from 3½ in. to 3 in.

Case III:

1. Enter Table 10-3 under Welds A and B; select the design strength which will be adequate for the beam end reaction. The angle length must be compatible with the depth of the beam. The maximum length is the *T*-distance minus a distance that will provide sufficient room for welding along the top and bottom edges of the angles. (See discussion of this in Chapter 1 "Clearance for Welding".) The recommended minimum length is half the *T*-dimension. If several selections can be made from Table $\boxed{10\text{-}3}$ which satisfy the requirements for weld design strength and angle length, the following criteria should be considered: Rev. 9/1/03
When possible, avoid using welds larger than $^5/_{16}$-in., as they require more than one pass by the welding operator.
A weld size that requires a minimum web thickness greater than that of the beam web should be avoided where possible. This requires the weld value to be reduced, as explained previously.
After the above two criteria are satisfied, using the shorter length of connection angles and lesser length of weld generally is advisable, although, if the same

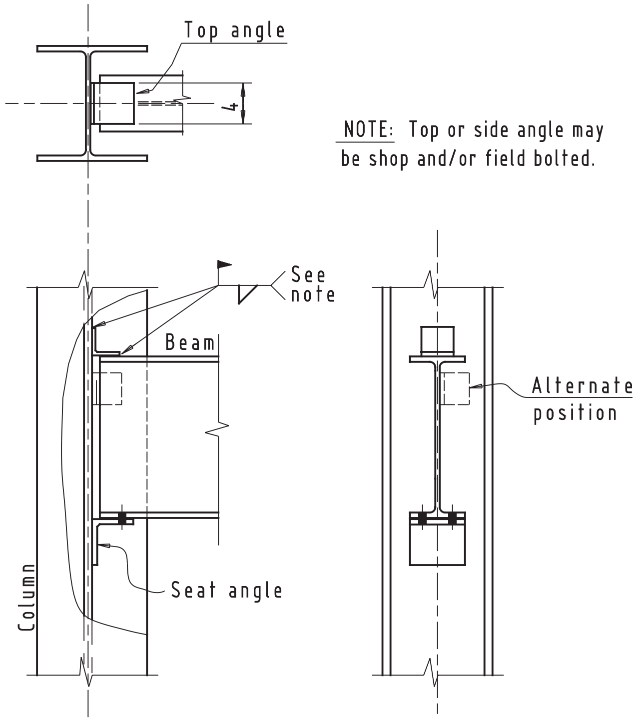

Figure 3-24. Unstiffened seated connection.

connection will be duplicated extensively, making a cost study and evaluating all factors is suggested.

2. Weld A:
   Compare design strength with factored reaction.
   Note the minimum thickness of beam web required and reduce the design strength of the weld as previously explained if the thickness of the beam web is less than the minimum. If the weld design strength is inadequate, select a longer length angle and recheck the required design strength of the longer weld.
3. Weld B:
   Compare design strength with factored reaction.
   Investigate the design strength of the supporting material to receive the weld force. If it is deficient, reduce the weld size as explained previously. If this results in inadequate weld design strength, select a longer length angle and recheck the design strength of the longer weld.
   Use 3 × 3 angles for lengths up to, but excluding, 18 in. and 4 × 3 angles (4-in. leg o.s.) for lengths 18 in. and longer.
   For examples of bolted/welded double-angle connections and all-welded double-angle connections the steel detailer should refer to the Manual, Part 10.

SEATED BEAM CONNECTIONS

Unstiffened Seated Connections

One of the most frequently used flexible type welded beam connections is the unstiffened seat, an example of which is shown in Figure 3-24. For factored loads up to the limits of the design strengths, these require a minimum of shop and field welding. Design strengths for various thicknesses of unstiffened seated angles are given in Table 10-6 of the Manual, Part 10 for beams with steel of $F_y = \boxed{50\text{ ksi.}}$ The Table is based on the use of steel with F_y = 36 ksi for the seat angle and on the same factors discussed for bolted connections for Table 10-5. However, the outstanding leg of the seat angles may be made either 3½ in. or 4 in. wide. Rev. 9/1/03

The thickness of seat angle required to support a particular beam and its given end reaction is determined from Table 10-6. The weld size and length of vertical leg of the seat angle connected to a supporting member also are determined from the Table. Design strengths are based on the use of *E70XX* electrodes.

The welds are placed along the ends of the vertical leg of the seat angle and extend the full length of the vertical leg. To ensure the weld size is maintained over the length of the weld, the welds are returned a distance equal to twice the weld size along the heel of the angle (Specification Section J2.2b). The eccentricity of the loading tends to pull the seat angle away from its support and creates a maximum force in the welds. Horizontal welds used across the top and bottom of the seat angle are permitted and would offer better resistance to the eccentric load. However, care should be taken that the top weld will not interfere with the end of the seated beam if the beam overruns its specified length.

The vertical welds attaching the seat to the supporting member are separated by the length of the seat and forces set up in the supporting member are resisted along four shear planes (refer to Figure 10-14 in the Manual, Part 10). Thus, reduction of the tabulated weld design strengths normally is not necessary when unstiffened seats line up on opposite sides of a supporting web. The forces on each side of the web due to eccentricity (from the beam end reactions with respect to the weld lines) react against each other and have no affect on the web. Therefore, for an 8-in. long 7 × 4 × ¾ seat angle supporting a beam of F_y = 36 ksi steel and a web thickness of 9/16-in. (ϕR_n = 71.6 kips), the minimum support thickness would be:

$$t = \frac{\phi R_n}{\phi \times F_w \times l \times 4(planes)}$$

$$t = \frac{71.6}{0.9 \times 0.60 \times 36 ksi \times 7 in. \times 4} = 0.132 in.$$

As in the case of all-bolted stiffened seats, the supported beam should be field bolted to the seat plate with high-strength bolts. The seat plate supporting a channel generally is at least 6 in. wide to accommodate two bolts. The width of stiffener need be no more than that required to provide adequate bearing for the given beam end reaction. In the case of a channel the seat plate may project beyond the stiffener.

The design strengths given in Table 10-8 of the Manual are based on welds made with E70XX electrodes.

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The table provides tabulated values that are valid for stiffeners with minimum thickness of:

$$t_{min} = \left(\frac{F_{y\,beam}}{F_{y\,stiffener}} \right) \times t_w$$

But not less than $2w$ for stiffeners with F_y = 36ksi nor $1.5w$ for stiffeners with F_y = 50ksi. In the above, tw is the thickness of the unstiffened supported beam web and w is the nominal weld size. So, when the supported beam with unstiffened web is of steel with F_y = 50 ksi and the seat is of steel with F_y = 36 ksi, the stiffener plate thickness must not be less than the ratio of yield stresses of the two steels, 50/36 = 1.39 ≈ 1.4 times the beam web thickness. Additionally, the minimum stiffener thickness (t) should be at least $2w$ for stiffener material with F_y = 36, where w is the weld size for 70 ksi electrodes.

The steel detailer is directed to the discussion on “Stiffened Seated Connections” in the Manual, Part 10 for the procedure and examples applicable to designing these connections in conjunction with Table 10-8.

In Figure 3-27 optional trim lines are shown for the stiffener plate. Where duplication of stiffener plates occurs, use

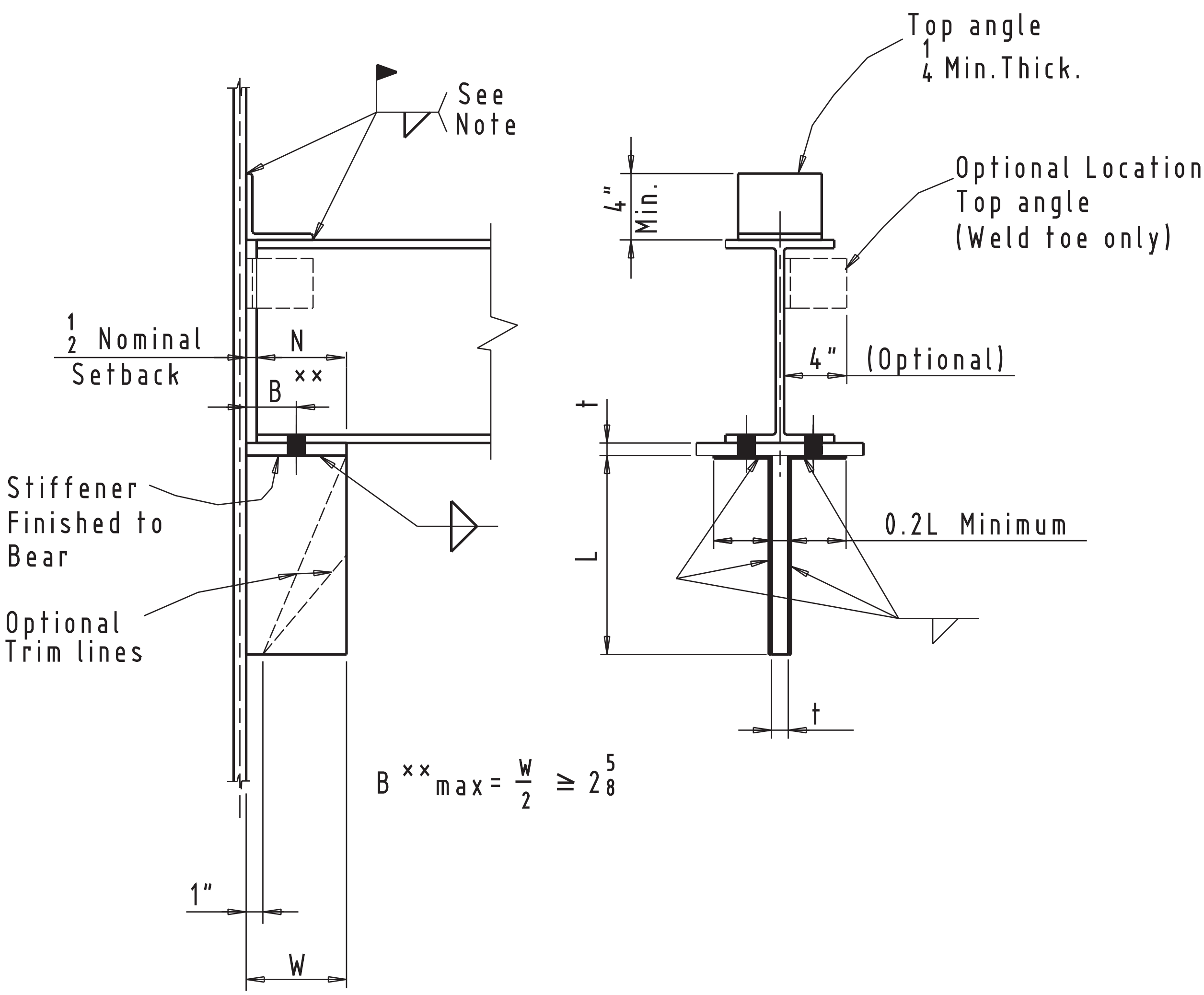

Figure 3-27. Stiffened seated connection.

of these cutting lines will save material provided that the plates are nested or multipled (see Chapter 1).

End-plate Connections

End-plate connections are discussed under bolted connections earlier in this chapter, where the steel detailer is referred to the Manual, Part 10 for information and details. Regarding welding criteria for end-plates, the steel detailer should note that the shop weld attaching the end-plate to the supported beam is not returned around the end of the web, which is in accordance with AISC Specification Section J2.2b (see Figure 3-26). The tabulated design strengths include an allowance for starting and stopping the weld on each side of the beam web.

Single-plate Connections

Earlier in this chapter the steel detailer is referred to the Manual, Part 10 for information and examples to be used in selecting a single-plate connection. The steel detailer should note that the size of the fillet weld connecting the plate to the supporting member (using F_y = 36 ksi plate material and E70XX electrodes) shall be equal to $\frac{3}{4} \times t_p$ where t_p is the thickness of the plate. This assures that the plate yields before the weld yields. Furthermore, the weld connecting the plate to its support should not be wrapped around the ends of the plate (see Figure 3-26).

For further information on Single-plate connections, readers should refer to the AISC Design Guide on the topic, available Spring, 2003.

Single-angle Connections

Table 10-11 in the Manual, Part 10 presents design strengths for single-angle connections where the angle is field bolted to the filler beam and shop welded to the supporting member. Note that the weld attaching the angle to the supporting member must be flexible. Thus, the weld is placed along the toe and across the bottom of the angle with a return at the top per Specification Section J2.2b. Weld placement across the entire top must be avoided. See Figure 3-14.

Tee Connections

The use of tees in connections was discussed earlier under bolted construction. Adequate flexibility must be provided when a tee is welded to a supporting member. This is provided by placing welds along the toes of the tee flange with returns at the top in accordance with Specification Sect J2.2b. Welds across the entire top of tee flanges must be avoided.

CONNECTIONS COMBINING BOLTS AND WELDS

When connections combine bolts and welds, refer to AISC Specification Section J1.9. Design drawings should indicate clearly where such connections occur.

SELECTING CONNECTIONS

Shear Connections

Although having the end reaction for each beam shown on the design drawings is desirable and would provide the most economical connections, usually the data given by the designer for the types of shear connections in building work is brief, often being confined to the general notes. Frequently, the notes simply refer to the AISC Manual, in which case the steel detailer would refer to Part 5 to determine the reactions and to Parts 9 and 10 to select the connection. Rev. 9/1/03
Some designers may establish minimum connections, but generally the fabricator is responsible for providing adequate shear strength to meet the criteria given on the contract documents, subject to the approval of the owner's designated representative for design. The design of these connections is covered earlier in this chapter. Some of the various types of shear connections used with columns are discussed in the following sections.

Framed and Seated Connections-Bolted

Figure 3-28 shows framed and seated connections for shop bolted-field bolted construction. The conventional two-angle framed connection with the beam web fasteners in double shear is shown in Figure 3-28a. Earlier in this chapter this type of connection was discussed. Figures 3-28b, 3-28c and 3-28f illustrate framed connections utilizing tees or single-angles with the beam web fasteners in single shear. By either arrangement using shop bolts for all fasteners through the column is possible and at the same time permits side erection of the beam. Because the web fasteners are in single shear, an additional row of web fasteners may be required, as shown in Figure 3-28c. Note that, when the beam in Figure 3-28f is connected to the inside of the outstanding leg of the connection angle, this type of connection is known as a wrapped connection.

The framing connections shown in Figures 3-28a, 3-28b, 3-28c and 3-28f are not adapted to connecting beams to column webs. An impact wrench cannot be used to tighten the bolts through the beam web because of interference by the column flanges. Therefore, the seated connections shown in Figures 3-28d and 3-28e are generally employed. The unstiffened seat in Figure 3-28d is preferred if available strengths are great enough to support the beam reaction. The stiffened seat shown in Figure 3-28e can be made as strong as necessary. Quite often the strength of a seated connection is governed by the strength of the beam web. Either of these ficient weld is furnished to resist the maximum design strength these rods can carry.

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In Figure A3-45 note the groove weld joint numbers in the tails of the arrows. These are AWS D1.1 designations. These joint details and numbers are shown in the Manual, Part 8 under "Welded Joints". When the AWS joint number is shown in the tail of the arrow, giving the root opening and groove angle is unnecessary.

Truss Chord Splices-Welded

Usually, splices in tension chords of welded trusses are made with complete-joint-penetration groove welds. Joint preparation and welding are performed in accordance with AISC Specification Section J2.1. Where abutting members of different cross-sections occur in tension splices, a slope must be provided through the transition zone which does not exceed 1 in 2½ (4¾ in 12, approximately), although flatter slopes are preferred. This slope is accomplished by clipping external corners and sloping the weld faces (see Figure A3-47a). Where the difference in thickness is too great, the thicker part must be chamfered in addition to sloping the weld face (see Figure A3-47b).

Where tee or W shapes are spliced, complete-joint-penetration groove welds on the flanges require cutting the web to permit the use of backing bars, backing welds, far-side welds or full-length back-gouging of beveled welds (see Figure A3-47c). Extension bars and backing bar extensions are required to insure that the full cross-sectional area of the groove weld is effective for the entire width of the flange. The selection of the type of preparation for a grooved joint depends upon the thickness of the material and the fabricator's preference. Refer to Chapter 4 for discussion of extension bars and backing bars.

Where heavy rolled shapes (listed in ASTM A6 Groups 4 and 5) or shapes built-up by welding plates more than two inches thick together to form the CROSS-SECTION, and where these shapes or cross-sections are to be spliced by complete-joint-penetration welds and are subject to primary

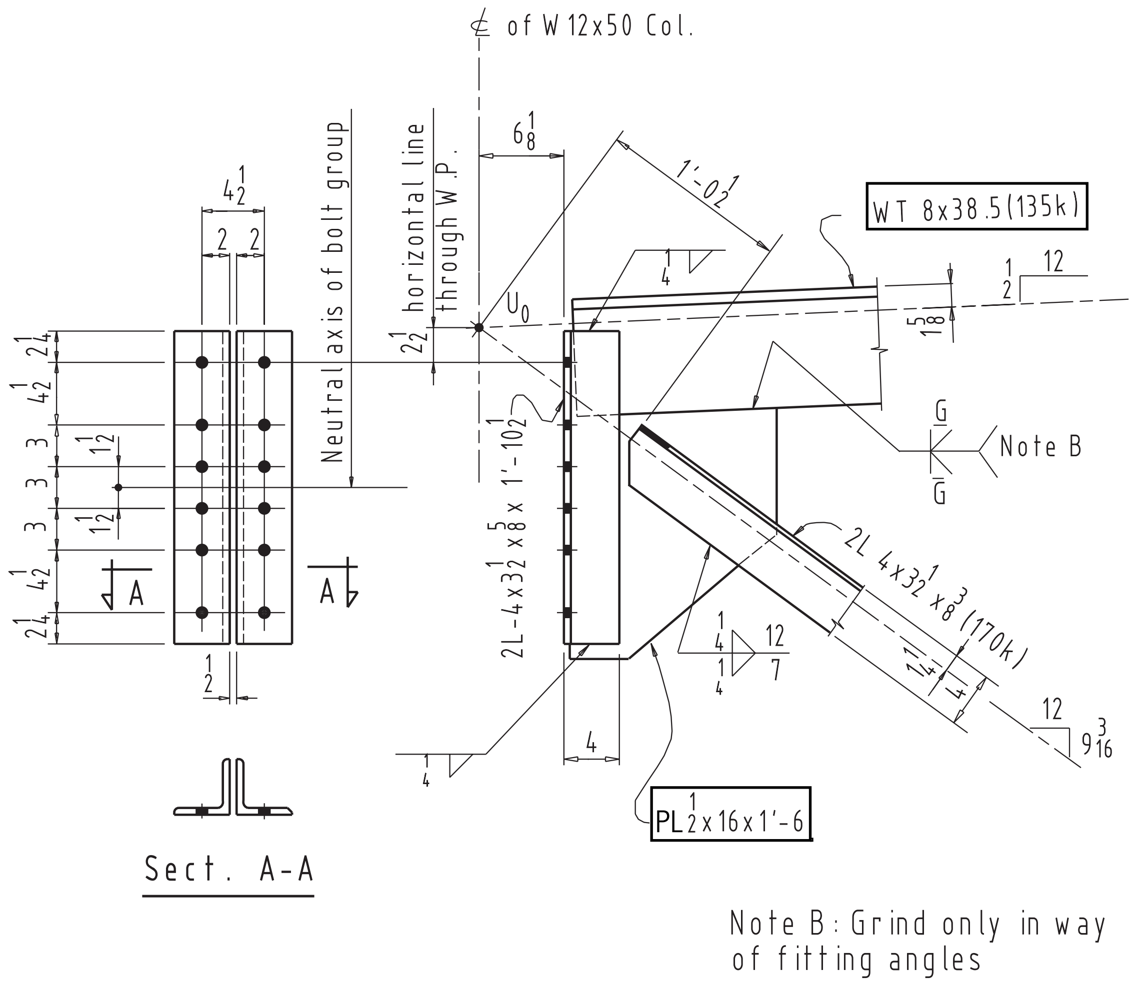

Figure 3-48. Top chord connection to column.

tensile stresses due to tension or flexure, the material to be spliced is subject to special mill order requirements. The material must be supplied with Charpy V-Notch testing in accordance with ASTM A6, Supplementary Requirement S5. The reason for the special testing is that the web center of heavy hot-rolled shapes as well as the interior portions of heavy plates may contain a coarser grain structure and/or lower toughness material than other areas of these products. Thus, when these materials are joined by complete-joint-penetration groove welds, which extend through the coarser and/or the lower notch toughness interior portions, tensile strains induced by weld shrinkage may result in cracking. The fabricator must observe special precautions when preparing the material for welding and in the welding process itself. Therefore, generally tensile splices in heavy sections are made using splice plates. Heavy sections subject to compression may be spliced using either partial-joint-penetration groove welds in combination with fillet welded splice plates, bolted splice plates or a combination of bolted/fillet-welded splice plates. The detailer is encouraged to read AISC Specifications Sections A3.1c and J1.5 and their Commentaries for further information on the matter of splicing heavy sections.

The AISC Specification Table J2.5 permits design strengths for complete-joint-penetration groove welds to be equal to the design strengths for the connected material, providing the proper grade of electrode is used. Thus, the strength of the welded splice is the same as that of the connected material of the same cross-sectional area. Tension splices should be checked for required net section. The access hole sometimes is left open and sometimes is filled in the completed joint, depending upon the owner's designated representative for design.

Chord splices are expensive to fabricate and should be avoided wherever possible. Joint U4, Figure A3-45, shows a groove weld for which some fabricators might choose to substitute a simple bead. If within the capacity of shop equipment, this is a satisfactory solution for most slightly pitched trusses. Rev. 9/1/03

In shop welded-field bolted construction the field splices in chords require material which may be bolted on both sides of the joint or shop welded on one side and field bolted on the other side. The design of these splices requires an engineering analysis and is beyond the scope of this text.

Top Chord Connection to Column

When the working lines of the top chord and diagonal intersect at the centerline of a column and the connection is located away from this intersection, as in Figure 3-48, the eccentricity results in a moment. Assuming the connection between the members is adequate, joint rotation is resisted by the combined flexural strength of the column, the truss

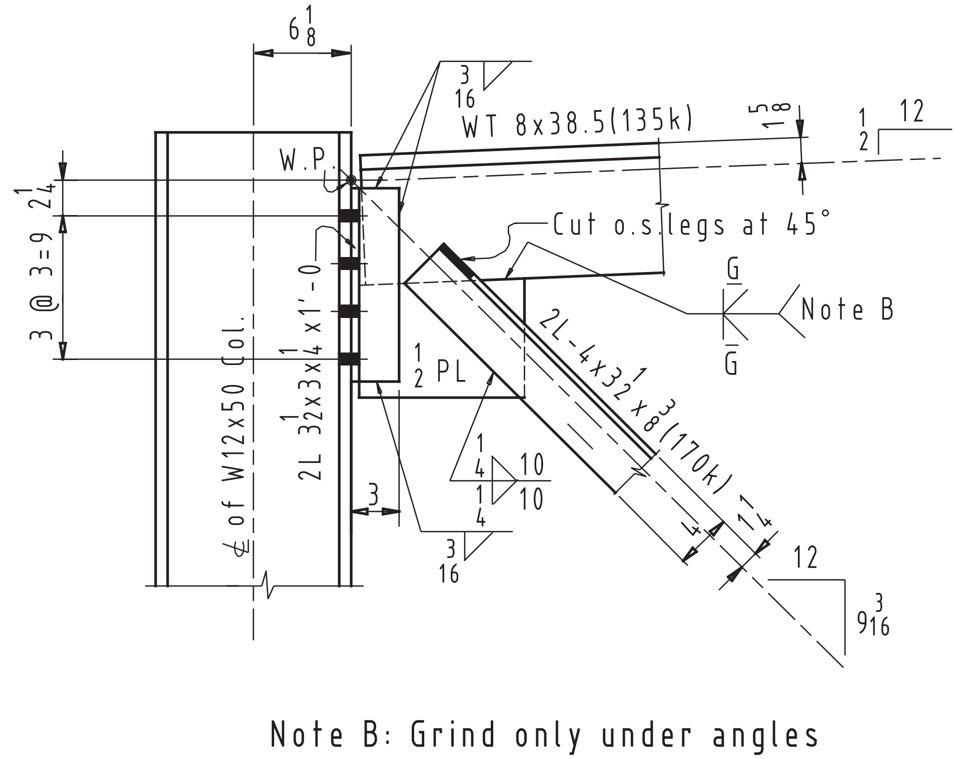

Figure 3-49. The common intersection of the three working lines is shifted from the column centerline to the face of the column.

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The value of $\boxed{\delta}$ will be the increase or decrease in panel length, depending on whether the load P induces tension or compression. The deformation of each panel on one side of the truss centerline is figured separately, then summed up to determine the total movement at the extreme ends of the truss.

SHIMS AND FILLERS

Shims are furnished to the erector for use in filling the spaces allowed for field clearance which might be present at connections such as simple shear connections, PR and FR moment connections, column base plates and column splices. These shims may be either strip shims, with round punched holes (see Figure 3-50a) or finger shims, with slots cut through to the edge (see Figure 3-50b).

Whereas the strip shim is less expensive to fabricate, the finger type has the advantage of lateral insertion without the need to remove erection bolts or pins already in place. If finger shims are inserted fully against the bolt shanks, they are acceptable for slip-critical connections and are not to be considered as an internal ply. However, they must have the surface preparation required by design for the slip-critical connections. In such cases the permissible bolt load may be taken as though the shims were not present. The reason is that less then 25% of the contact area is lost – not enough to affect the performance of the joint.

A filler is furnished to occupy a space that will be present because of dimensional separations between elements of a connection across which load transfer occurs. The steel detailer should become familiar with Specification Section J6, which describes the procedures for developing fillers in welded and bolted connections.

1. Although this expression does not apply to the throat areas of fillet welds in certain types of skewed work, or fillet welds with unequal legs, its use in most cases will provide conservative results. However, if a critically stressed fillet weld has one leg substantially longer or shorter than the other, the actual throat size should be determined and used in stress computations.
2. AWS D1.1 does not include these increased values for submerged arc fillet welds.
3. For further guidance on the design of skewed beam connections, reference an article by W. A. Thornton and L. Kloiber, titled "*Connections for Skewed Beams*" appearing in the May, 1999 issue of Modern Steel Construction magazine, and available at www.aisc.org.
4. Bending moment is a term that expresses the measure of the tendency of a structural member to bend. It is the product of a force expressed in units of weight times a distance expressed in units of length. The numerical value of bending moment is expressed as kip-ft, kip-in., lb-ft or lb-in.

CHAPTER 4
BASIC DETAILING CONVENTIONS

Definition of detailing conventions that are in universal use. Over the years, innovation, trial and error, common logic and a desire to improve shop and erection drawings have combined to evolve the standard practices and conventions we use today.

GOOD DETAILING PRACTICES

The steel detailer must become familiar with many requirements for the proper and correct preparation of shop and erection drawings. The following guidelines and good practices are independent of the type of structure being detailed. Some of the items listed may be considered “rules of thumb”. Discussions of these guidelines can be found in this and other chapters of this text. Chapter 6 lists guidelines for preparation of erection drawings.

General Drawing Presentation and Drafting Practices

1. Each shipping piece consists of main material alone or with detail material, which is a connection or other part attached to the main member.
2. On manually prepared shop drawings shipping pieces may be duplicated on one sketch when the differences are minor and the sketch does not become complicated by notes. Otherwise, a separate sketch should be made.
3. Lettering should be neat and legible. Notes should not run into the sketch or the associated dimensioning. Preferably, general notes should be placed near the title block. Small letters should be about $\frac{3}{32}$-in. high and numbers about $\frac{5}{32}$-in. high. The size and boldness of the letters should be in proportion to the importance of the information. Tiny lettering and obscure locations of notes on the drawing may cause the shop to overlook these notes. Notes should be terse, to the point and positive rather than negative. Remember, the worker in the shop must read the drawing under conditions of less light and cleanliness than that available to the steel detailer.
4. Lettering should be horizontal, vertical or parallel to a sloping member such that it can be read from the bottom or right side of the drawing.
5. Special notes should be provided to convey all detail instructions to the shop that are not shown readily by dimensions, sketches, detailing standards or billing. Information explained by adding one or more views or sections to a sketch will be more valuable to the shop than a sketch with many notes. Thus: “A picture is worth a thousand words.”
6. In all detailing use good line contrast – lighter lines for dimension lines and bolder lines for the object lines.
7. Dimension lines should be placed far enough from the sketch to allow sufficient room for dimensions. Generally, the first dimension line should be approximately $\frac{5}{8}$-in. from the sketch and $\boxed{\text{each succeeding}}$ line separated by about $\frac{3}{8}$-in. Rev. 9/1/03
8. All dimensions up to a foot are given in in., thus: $11\frac{15}{16}$. All dimensions of a foot or over are given in feet and in., thus: 1’-0 or 1’-2½.
9. The point of the arrowhead on a dimension line should touch the extension line and not go past it. Care must be taken in dimensioning so as to avoid possible misinterpretation.
10. Sections should be taken looking to the left and looking toward the bottom of the drawing. Avoid looking up and to the right.
11. Section views shall be oriented to retain the position indicated by the cutting plane, not rotated through 90°.
12. Never cross-hatch elements of sectional views.
13. Anchor rods, base plates and setting plates, grillages and embedded items should be the first pieces detailed.
14. Avoid one-hole structural connections, except when connecting rod bracing.
15. Re-entrant cuts, such as for beam copes and in bottom-chord gusset plates at columns, should be drawn with the radius clearly shown (See Figure 1-2).
16. On each shop drawing list the erection drawings where the members detailed on that particular shop drawing will be located.
17. Lengths of main members are not required to be drawn to scale.
18. Ends/edges to be finished must be marked Fin in accordance with the practice of the fabricator.
19. Angles and channel flanges should have gages detailed from the backs of their legs and webs, respectively.
20. Detail channels and angles looking at their backs.
21. If a shop intends to subcontract fabrication of concrete-filled HSS, detail them on shop drawings separate from other members so they can be sent to potential suppliers for pricing and subsequent fabrication.
22. Members to which wood will be attached are often supplied with attachment holes. Normally these holes are spaced randomly at about 2 ft to 3 ft on center and

are of the same diameter as any other holes required in the member. The steel detailer is not required to dimension holes for wood. A note such as “Wood holes @ 2’-6 will suffice.

23. Check with the fabricator if detailing different types of members (for example: beams and columns) on the same shop drawing is permissible.

Material Identification and Piece Marking

24. Each piece of detail material (sometimes referred to as “fittings”) must carry an assembly mark for identification and cross-referencing.
25. Any difference between detail material requires a different assembly mark.

Advance Bills of Material

26. Steel detailers must check all material against the advance material lists (see Chapter 5). If material has not been ordered or if it is ordered incorrectly, advise the fabricator.
27. When preparing advance material orders, ensure that the sizes are available. Extra long lengths may require splicing. Extra wide plates may not be available.
28. Avoid the use of Universal Mill (UM) plate material. Often, the edges of UM plate are rounded and may be wavy, which might pose a fit-up problem during fabrication.
29. Plate material that is to be bent should be ordered so as to assure that the bend line is perpendicular to the direction of rolling.

Shop bills of Material

30. Each assembly mark must be billed at least once in the shop bill, which is a pre-printed form on a drawing listing material required for fabrication of members in the shop. In some shops if the assembly mark occurs again on another shipping piece on the same drawing, the size need not be billed in the shop bill, but the mark and number of pieces must be given.

Beam and Column Details

31. For multiple-tier columns, out-to-out dimensions should be given at the bottom of columns and in-to-in dimensions between splice plates at the top of columns.
32. For column details, project sections off the web view looking towards the bottom of the column. (Figure 4-1).
33. At double-angle connections that are shop attached to column flanges, give the separation between the angles in sixteenths of an inch (Figure 3-33b).
34. If wrap-around connections are used, give the distance from the centerline of the column to the inside of the outstanding leg of the angle (Figure 4-2). Striping (spotting) is described in Chapter 6.
35. Extension figures are cumulative dimensions from a given point used to locate several connections (detail material or open holes) on a shipping piece. From that point fitters locate detail material and inspectors check the locations with the use of a tape. The figures must be given from the finished bottom of a column. Extension figures are given from a definite point at the left end of a beam (or girder). The dimension line for the extension figure locating the first connection from the bottom of a column or left end of a beam must always run unbroken to the point from which the extension is given (Figure 1-2). An alternative method for running dimensions is to provide a short dimension line pointing to the left at the point of origin for running dimensions, accompanied by RD for clarity (Figure A4-3).
36. Extension figures to web holes on beam and column details should never be placed on the same line as extension figures to the flanges.

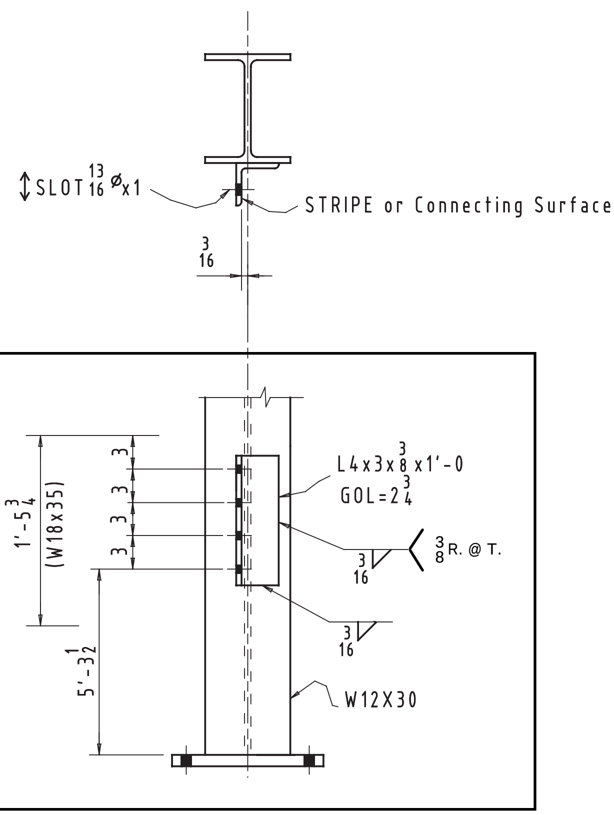

Figure 4-2. Wrap around connection use.

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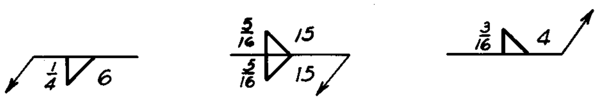
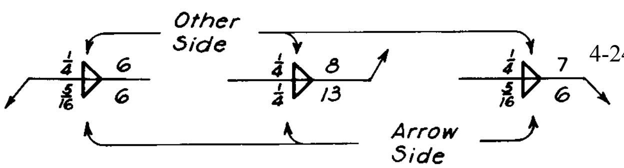
4-24a.

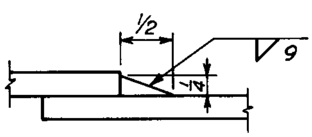
4-24b.



4-24c.



4-24d.

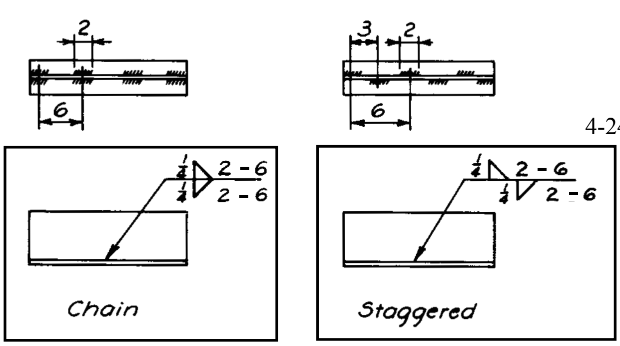


4-24e.

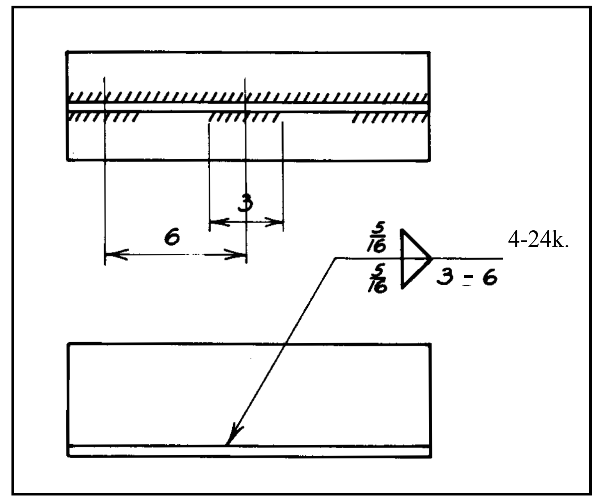
4-24f.

4-24g.

4-24h.



4-24j.



4-24k.

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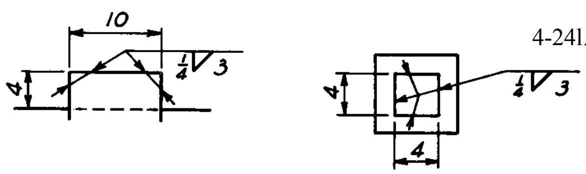
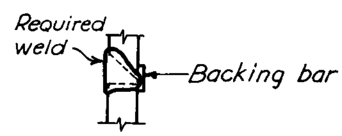
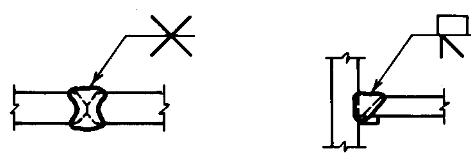


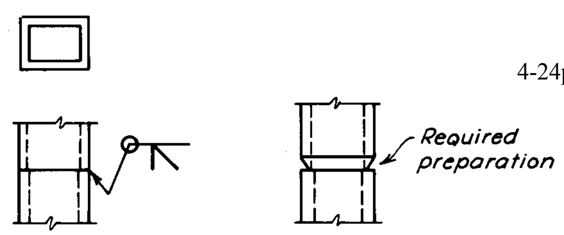
4-24l.



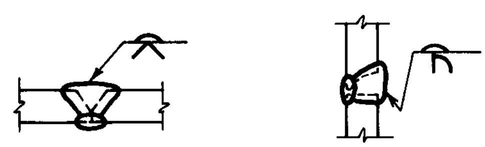
4-24m.



4-24n.



4-24p.



4-24o.

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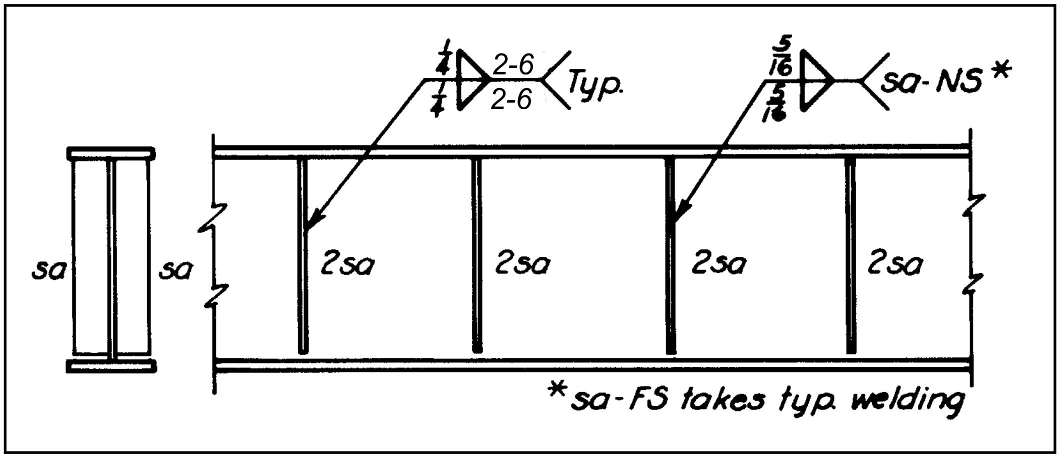


Figure 4-27. Typical welded joint.

skew. The Manual, Table 8-3 illustrates this condition. Case (A) shows a square-edged plate with a gap between the edge of the plate and the main piece. The size of the applicable fillet weld is the size required according to design plus the size of the gap. If the gap is too large to accommodate a fillet weld, a partial-joint-penetration groove weld may be suitable. Otherwise, consideration must be given to using a different joint configuration as illustrated in Table 8-36. The steel detailer should consult with the fabricator.

Shop Groove Welds

The construction and application of welding symbols for groove welds are similar to those for fillet welds. However, whereas fillet welds are represented by a single basic symbol (the triangle), groove welds involve seven basic symbols. These may be combined with each other or compounded with supplementary weld symbols to cover a wide variety of weld profiles and edge preparations. The shapes of the seven basic weld symbols for groove welds and their location significance are shown in Figure 4-28. Note that the vertical lines of bevel, J and flare-bevel groove weld symbols are always placed to the left when viewed facing the reference line.

In groove welds the weld metal is deposited substantially within the joint. The concept of the joint in groove welds is the same as for fillet welds and the arrow-side and other-side meanings of the symbol are the same. However, the direction of the arrow has an added significance for unsymmetrical joints such as bevel and J welds. For these welds the arrow not only designates the arrow-side of the joint, but also points toward the joint element which is to be grooved or otherwise prepared for welding (Figure 4-24m). The arrow direction is emphasized by an extra break whenever this special significance applies (Figure 4-20).

For symmetrical welds or where grooving the wrong edge for an unsymmetrical weld would be impossible, the arrow direction has no special significance beyond the usual arrow-side and other-side meanings (Figure 4-24n).

The weld-all-around symbol is used for groove welds in the same manner as for fillet welds (Figure 4-24o).

Back weld symbols are used with the weld symbols for single bevel, vee, U and J welds when completing the second or root side of these welds is necessary (Figure 4-24p).

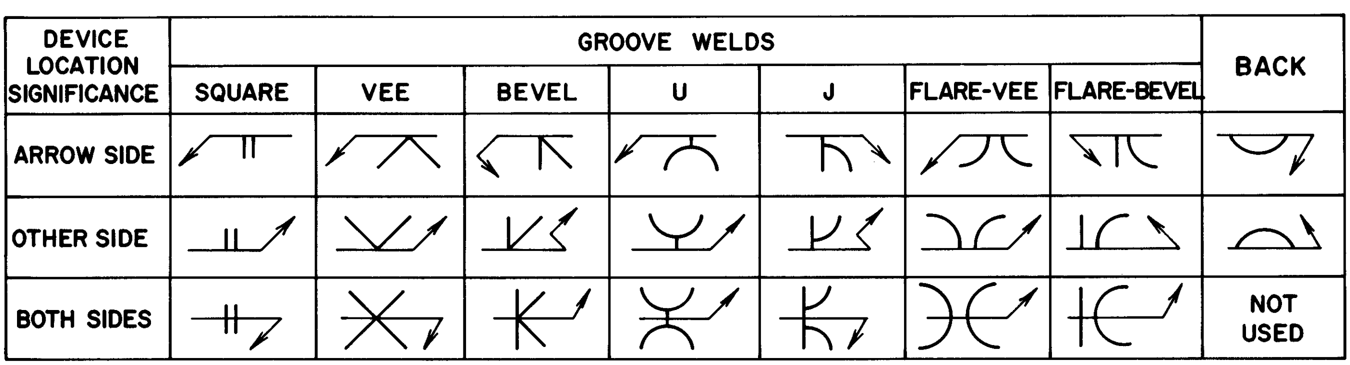


Figure 4-28. Groove weld symbols.

metal. The nominal angular value should be limited to 45°, as shown in Figure 4-29c.

When partial-joint-penetration groove welds are permitted, the contract documents should specify the effective weld length and the required effective throat. The shop drawings in turn should show the groove depth S and geometry that will provide for the specified effective throat E. Some fabricators indicate both the weld size and effective throat on the shop drawings to avoid confusion in the interpretation of these welds.

Partial-joint-penetration groove welds are used primarily for welded compression splices, the connection of elements of heavy box sections and pedestals and, in general, for joints where the stress to be transferred is substantially less than that which would require complete-joint-penetration groove welds. They are not recommended in joints subject

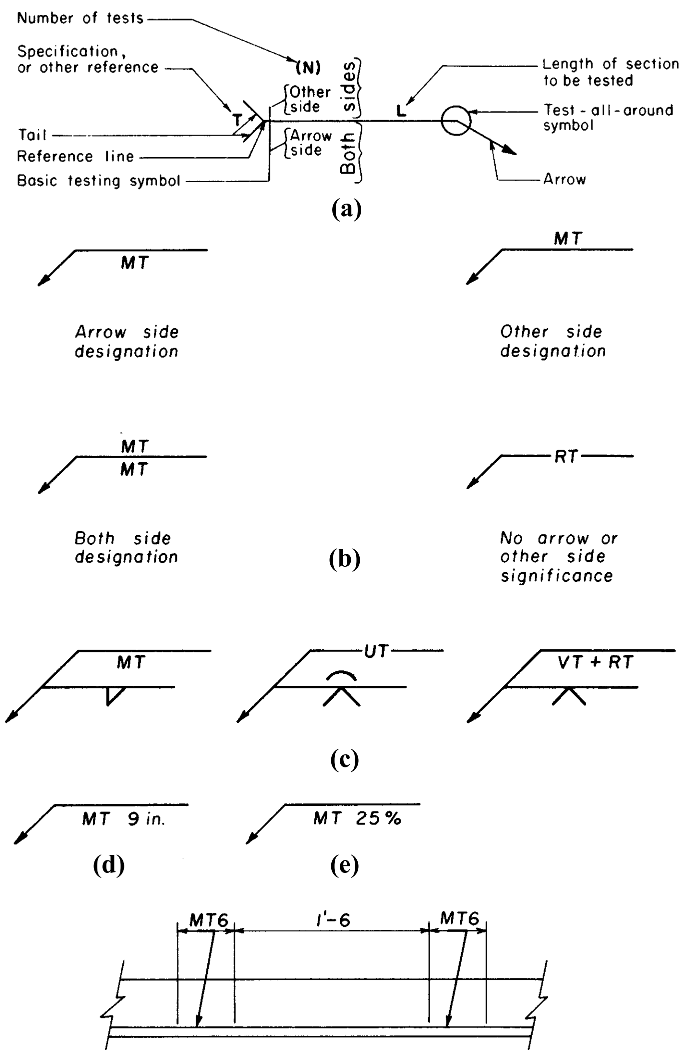


Figure 4-30. Non-destructive testing symbols combined with welding symbols.

to dynamic or cyclic loading, except as noted above for joining of components in built-up members.

Many fabricators require that partial-joint-penetration groove welds be detailed completely on their drawings in order to avoid possible misinterpretation of these welding requirements. The Manual, Part 8 contains details of the AWS prequalified joints.

Stud Welds

Welding symbols for stud welding by stud-welding guns are shown in Figure 4-24bb, which is designated as a circle containing an "X" symbol lying only on the arrow side. Stud size is located to the left of the circle, pitch to the right, and number of studs numerated in parenthesis below the circle.

On shop and erection drawings welded studs may be located by dimensions, as shown in Figure 4-24cc. The "X" symbol is used to designate the studs in plan to avoid confusing them with bolts or other fasteners for which holes must be provided. Note that studs are not permitted to be shop welded to the top flanges of beams and girders and to other surfaces on which erection workers would be walking prior to the installation of decking.

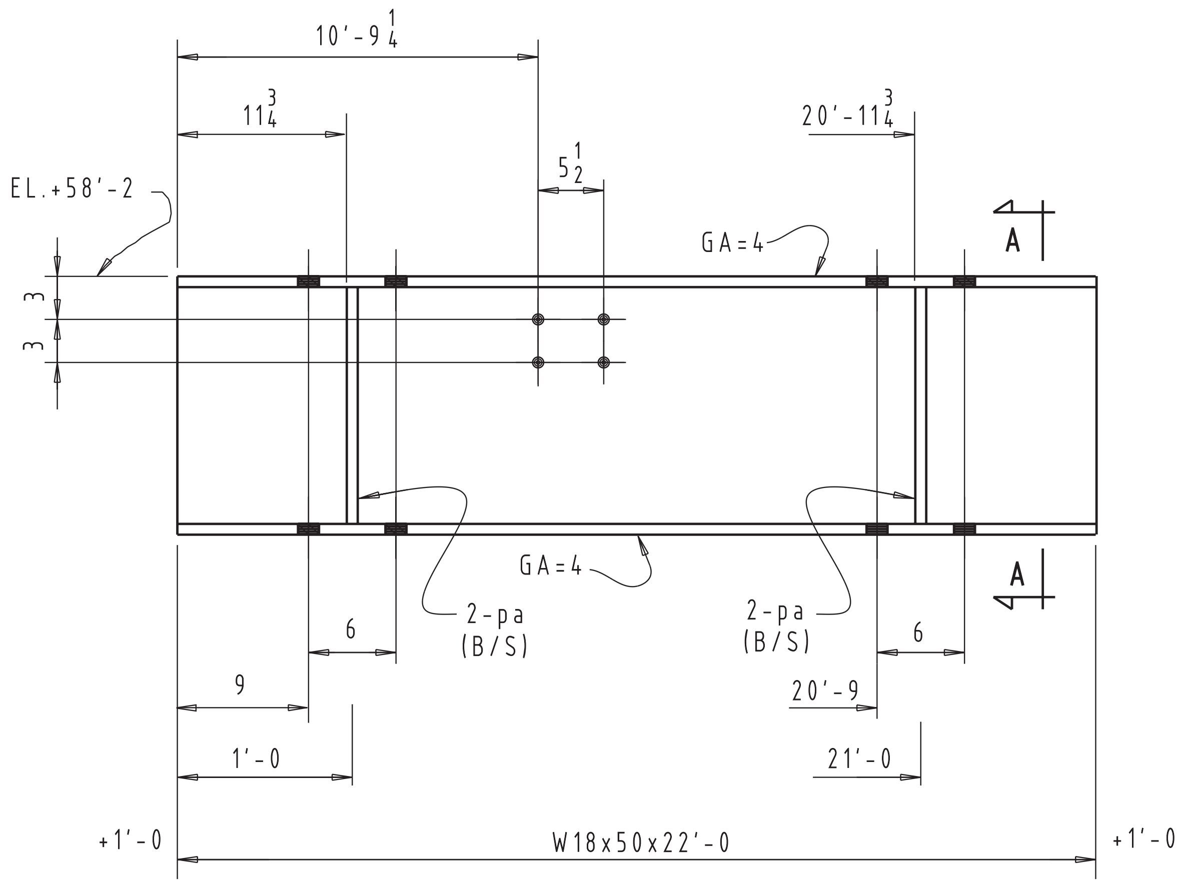
Shop Plug and Slot Welds

Plug and slot welding symbols employ a rectangular basic weld symbol (Figure 4-24dd). The arrow-side and other-side meanings for plug and slot welds indicate which of the two parts is to be welded (Figure 4-24ee).

The size of a plug weld is the diameter of the hole. The size of a slot weld includes the width and length of the slot. Hole diameters and widths of slots usually are made in odd sixteenths to permit use of standard punches in structural shops. Plug weld size is shown preceding the basic weld symbol Figure 4-24ff). Slot weld sizes are noted by detail references, shown preceding the basic weld symbol, which refer to dimensioned sketches elsewhere on the drawing (Figure 4-24gg). The position of both plug and slot welds is shown by a dimensioned location, toward which the arrow points (Figure 4-24hh). Long runs of multiple-spaced plug or slot welds may be located as shown in Figure 4-24jj.

Plug and slot welds are assumed to be filled completely with weld metal unless the depth of the filling, in inches, is shown inside the weld symbol. If the top of the weld must be flush with the surface of the plate, a supplementary weld symbol is added to the basic weld symbol, as for groove welds (Figure 4-24kk).

The steel detailer is cautioned not to apply plug or slot weld symbols to large openings which properly should be fillet welded around an interior joint boundary. An example of this type of fillet welding in an opening is shown in Figure 4-11. *AWS Symbols for Welding, Brazing and Nondestructive Examination, AWS A2.4*, provides for more



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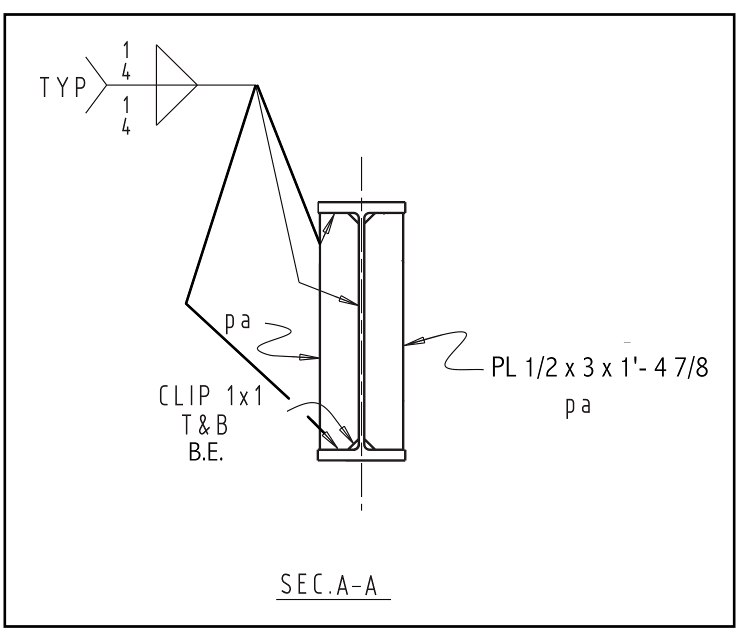

NOTE TO SHOP:

Seal weld all joints.
B.E. denotes Bearing End

MAT'L: ASTM A992
HOLES: 13/16 Ø
WELD: E70XX ELECTRODES
PAINT: GALV.

Figure 4-36. Web stiffeners welded to the flanges of a W shape or plate girder.

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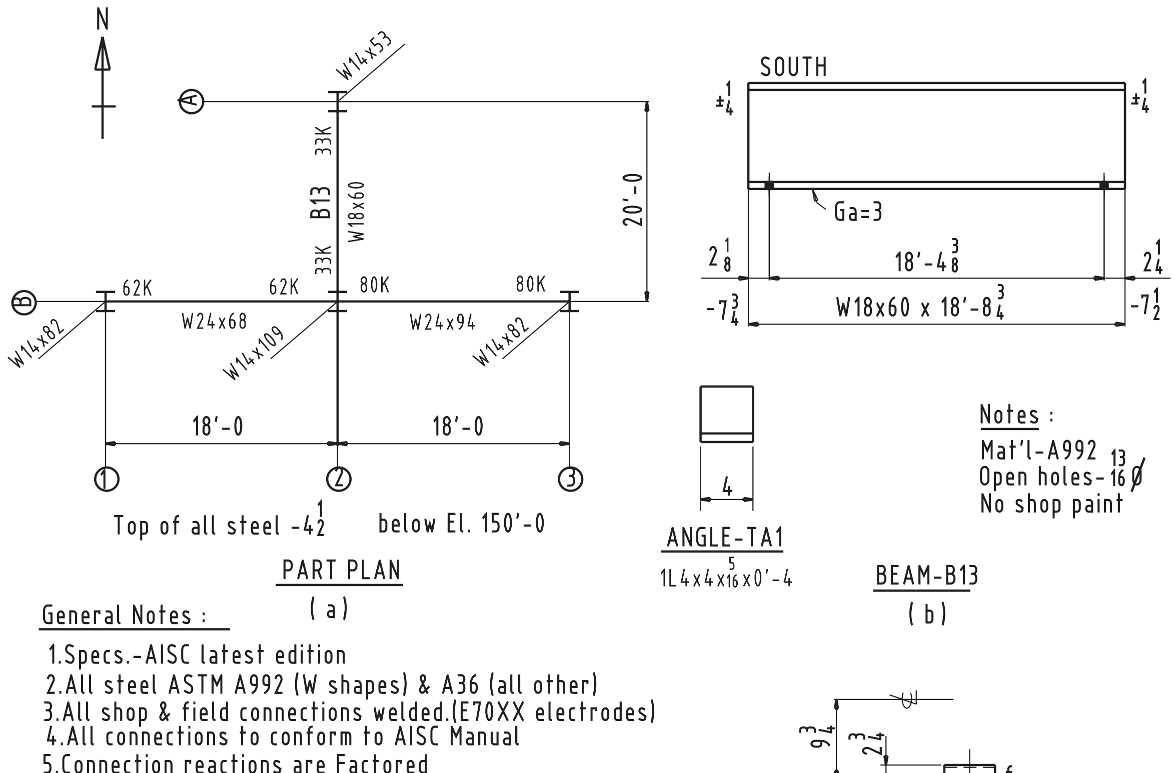

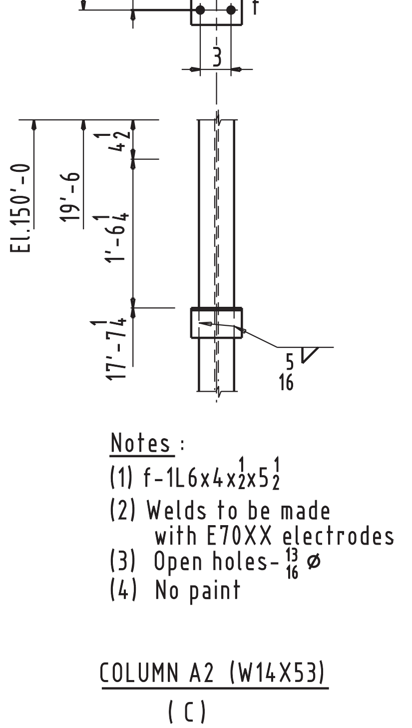

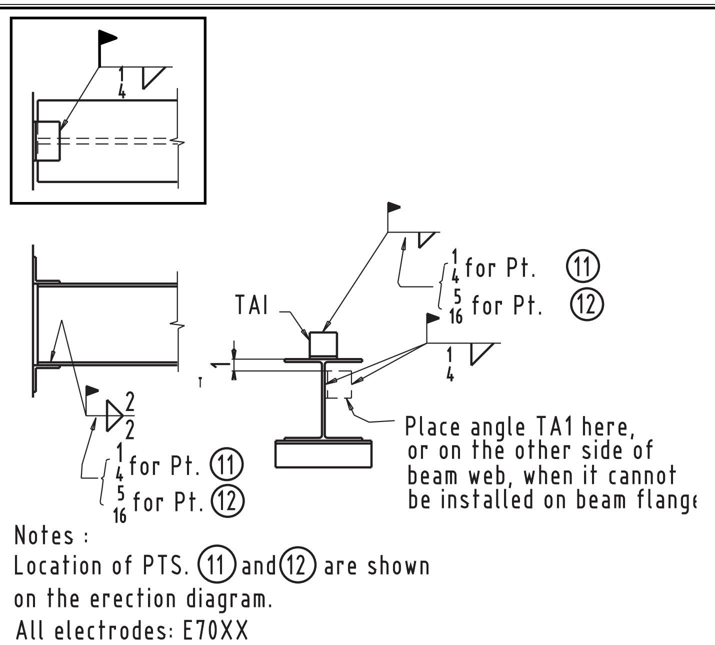

Figure 7-43. Seated connection.

Figure 7-42. Part plan and details of an all-welded project.

ment, a detail similar to the one shown in Figure 7-44 may be used.

The stability of a beam end supported on a bearing plate can be provided in one of several ways:

- The beam end can be built into solid concrete or masonry using anchorage devices.
- The beam top flange can be stabilized through interconnection with a floor or roof system, provided that system is itself anchored to prevent its translation relative to the beam bearing.
- A top-flange stability connection can be provided
- An end-plate or transverse stiffeners located over the bearing plate extending to near the top-flange k-distance can be provided. Such stiffeners must be welded to the top of the bottom flange and to the beam web, but need not extend to or be welded to the top flange.

In each case, the beam and bearing plate must also be anchored to the support. Additional information is provided in the Manual, page 2-13.

TRUSSES

Types of Construction

Most building trusses are shop welded and field bolted. Welded trusses provide a savings in main material because the members usually do not have any holes for fasteners, therefore, tension members may be designed on the basis of gross section. Also, detail material, such as gusset plates joining truss components, is eliminated in many cases, resulting in savings in weight and, usually, in fabrication costs.

The type of welded truss most commonly used consists of tee sections for the top and bottom chords and angles for the web members, as shown in Figures A7-45 and A7-46. Note that the angles extend over and are welded to the stem of the tee. Provided that the loads in the webs are small enough, no extension plates are required. Otherwise, extension plates must be welded to the WT stem using a full penetration detail. When the forces are too large for a tee section to be used for the chords, W shapes, with the web vertical, may be used instead. This requires the use of gusset plates, which are welded to the top flange of the bottom chord section and bottom flange of the top chord section. Web member angles are welded to the gusset plates.

Another type of construction for heavily loaded trusses consists of W shapes for both chord and web members, as shown in Figure 7-47. Connections in these trusses are made usually by groove welding flanges to flanges and fillet welding webs directly or indirectly by the use of gussets. Because the fitting-up of joints in this type of construction is affected seriously by dimensional variations in the rolled shapes (see "Standard Mill Practice, Rolling Tolerances", the Manual, Part 1), members made by welding plates into H shapes are preferred by some fabricators.

The steel detailer will note by referring to Figures A7-45 and A7-46 that the gravity axes (through the centers of gravity of the cross sections) of the web and chord sections serve as working lines. This is a basic difference from bolted construction, where gage lines are used for the working lines.

Members of bolted trusses, except in the case of very heavily loaded trusses, usually are made up of angles because of the ease with which they may be connected by means of a single gusset plate at each panel point. Generally, a pair of angles, with one angle placed on either side of the connecting gusset plate, is fabricated to act as a single composite unit. Often, angles are punched or drilled in pairs.

When equal leg angles are used, there can be no misunderstanding as to which legs should be placed back-to-back. When unequal legs are used in welded or bolted construction, however, the proper legs must be assembled together. In tension members the wider legs are placed, generally, against the stems of the tee chord sections or the gusset plates. In compression members the proper arrangement of unequal leg angles is very important. This fact will be

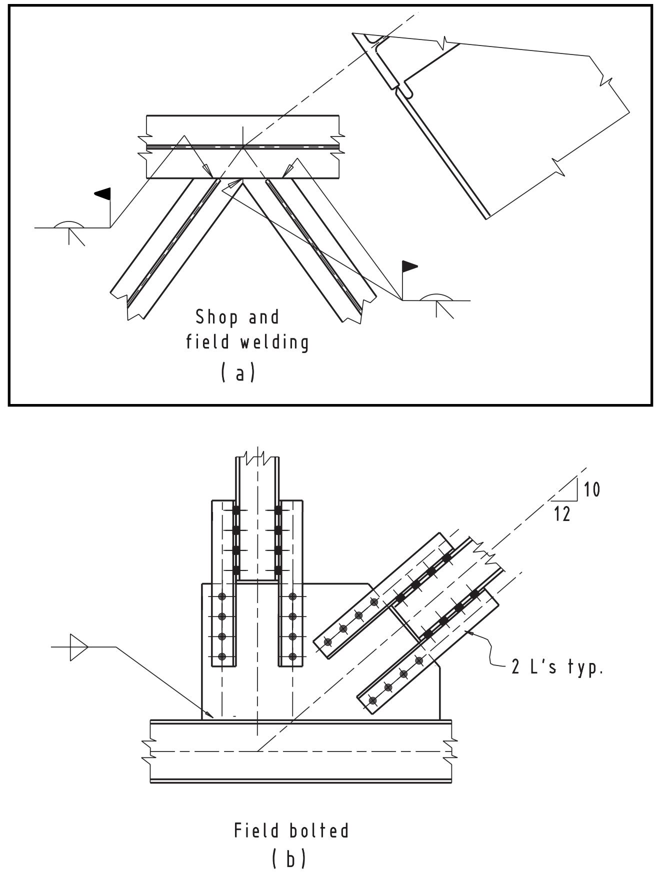

Figure 7-47. Construction of heavily loaded trusses consisting of W shapes for both chord and web members.

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Clearance for Field Work

Figure 7-86 shows a study made to check the clearance available for erecting seated filler beams where the side angle connections are to be shop-fastened to the supporting girder beams. Similar studies of erection clearances are necessary in the case of framed filler beams (Figures 7-87 and 7-88).

Ordinarily, as indicated in Figure 7-86, the slightly shorter distance "out-to-out" of connection angles (c. to c. of supports minus c-distance) between supporting members is sufficient to allow forcing a framed beam into position. Occasionally, however, because the beam is relatively short or because heavy connection angles with wide outstanding legs are required, the diagonal distance A may exceed the distance B by such an amount that the connection at one end must be shipped bolted to the filler beam to permit its loosening or removal during erection.

To permit the use of plain punched filler beams (Figure 7-88), shop fastening one angle of each pair of connection angles to the webs of the supporting girder beams is worthy of consideration. The other angle of each pair, being bolted to the girder beam for shipment, can be removed or loosened in the field to erect the filler beam. Once the filler beam is in position, the angle is bolted permanently to it and to the girder beam web. However, if the design strength of a single-angle or single-plate connection would be adequate, its use is a better approach. In this situation the single-angle or single-plate would be shop attached to the girder beam webs.

To erect a beam seated on the webs of the supporting columns (Figure 7-89), the top angles are removed tem-

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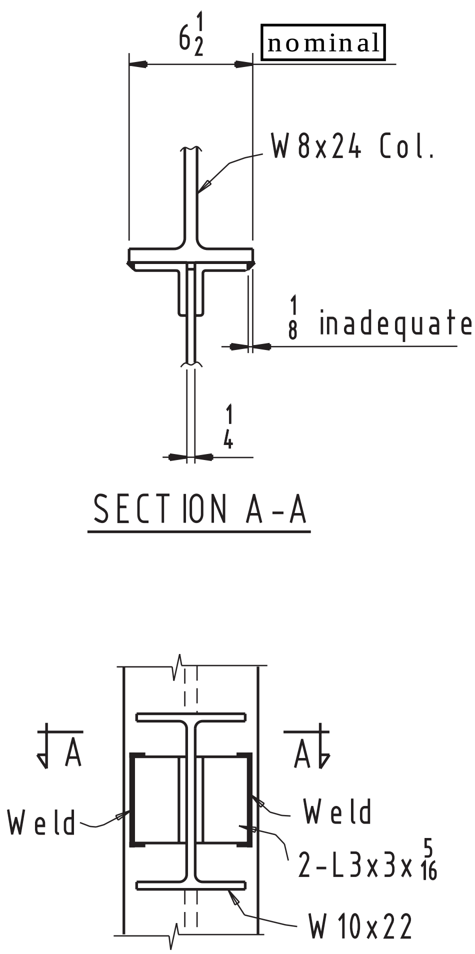


Figure 7-85. Improper connection detail.

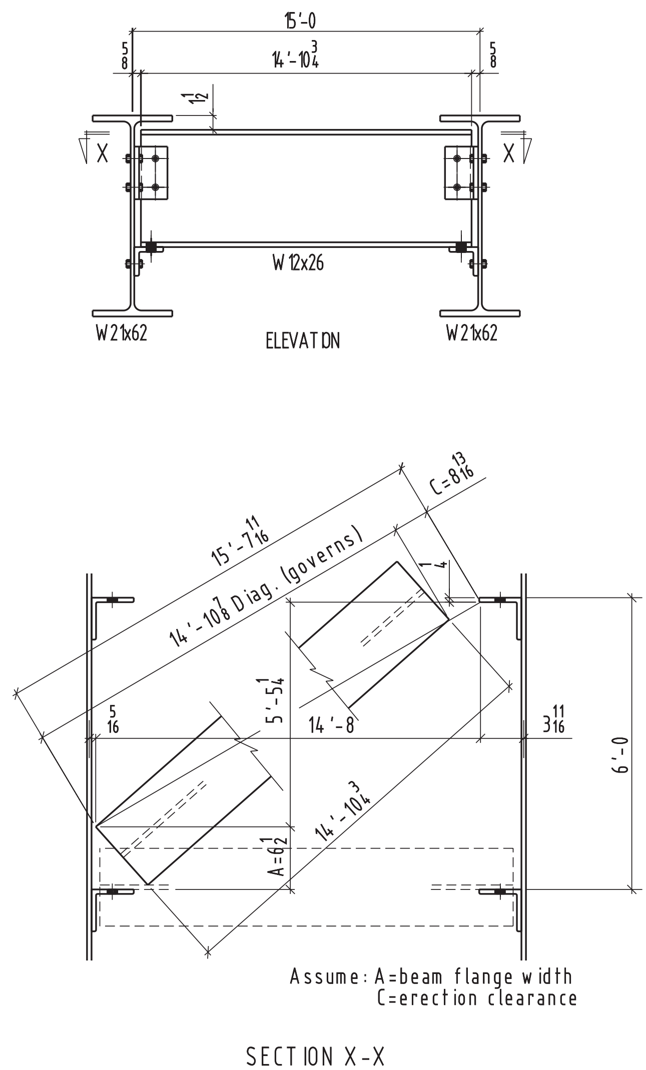
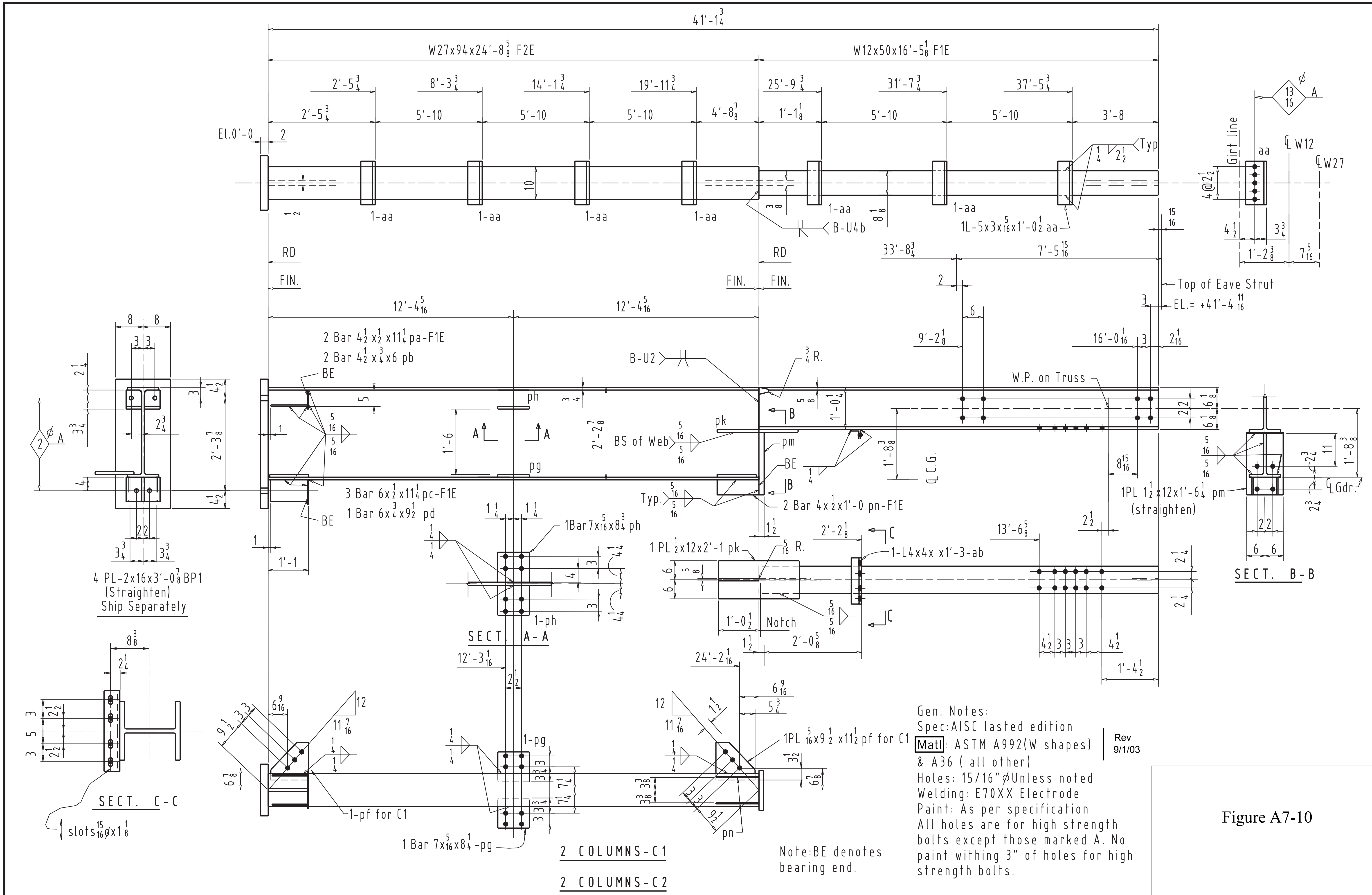


Figure 7-86. Clearance available for erecting seated filler beams where the side angle connections are to be shop-fastened to the supporting girder beams.



Gen. Notes:
Spec:AISC lasted edition
Matl: ASTM A992(W shapes)
& A36 (all other)
Holes: 15/16"ø Unless noted
Welding: E70XX Electrode
Paint: As per specification
All holes are for high strength bolts except those marked A. No paint withing 3" of holes for high strength bolts.

Figure A7-10

recovered. The permanent increase in length that remains is called permanent set.

For an idealized model such as shown in Figure B-17, the member will stretch until it finally breaks in the region where the cross-section contraction is greatest. The elongation at which it breaks (fractures) is several times the elongation at which yielding begins. The ability of steel to undergo significant yield deformation before it fractures is called ductility.

Ductility is one of the most important a properties of steel. Most engineering models that are used to predict the strength of steel members make use of steel's ductility. Ductility is one reason that steel can be successfully welded. Ductility also means that as a steel member yields it absorbs the energy applied to it by the external force producing the yielding deformation. Any structure that is subjected to an earthquake has considerable energy imparted to it by the earthquake ground motions. In steel structures, this energy is absorbed by yielding of overloaded components in the structure. Even though the components may be deformed to the point where they must be repaired or replaced before the structure can be put back into service, the steel does not fracture during the earthquake and the structure does not collapse. This protects the lives of the occupants. Special steel connections are now in use that deform and yield under the extreme effects of earthquakes, thus protecting the remaining components in the structure.

The force required to cause elongation ΔL in a tension member (such as shown in Figure B-17) depends upon the size of the member (its cross-section area) and its length. It has already been shown that dividing the force by the cross-section area gives the stress, which is a measure of the force intensity. Dividing the elongation ΔL by the total length L gives strain (denoted by Greek symbol ε):

$$\varepsilon = \frac{\Delta L}{L}$$

The units of ε are in. / in. Strain (ε) is a measure of how much each inch length of the member stretches, and hence it is the "rate" of elongation. The greater the elongation per in. (ε), the greater the overall elongation ΔL caused by the force. Figure 1-1 (Chapter 1) is a typical stress versus strain diagram for low-carbon steel.

LOAD AND RESISTANCE FACTOR DESIGN: LRFD

Load and Resistance Factor Design (LRFD) is a structural design procedure for selecting each steel member in a structure so that it most economically resists the loads applied to it. In LRFD it is first necessary to identify every possible condition for which the component would fail to perform acceptably, and then ensure that the member is adequate for these conditions. Such conditions are called limit states. Service limit states are based on the maximum expected day-to-day loads (service) loads. For example, if a beam supporting a floor deflects excessively or if it vibrates excessively during day-to-day usage, then deflection or vibration would be a serviceability limit state. Similarly, if a load applied to a beam were to cause it to break, then the beam's strength would be exceeded, and the beam would be said to have reached a strength limit state. Strength limit states are based on maximum expected day-to-day loads that have been increased by "factors of safety" (called load factors in LRFD) so that there is an acceptable margin of variability in the applied loads.

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The load factors are necessary for several reasons. Perhaps the most important reason is that it can be very difficult to predict the true maximum loads that will be applied to a structure. Load factors serve as protection against overloads, whether accidental or deliberate. Thus, load factors provide a margin of error between the expected service loads and the loads at which a member will reach its strength capacity.

One of the underlying concepts in LRFD is that the same margin need not apply to all loads. For example, actual dead loads can be better predicted than actual live loads, so dead loads do not require as great a load factor as live loads. For steel beam that support a combination of dead and live loads, the live load is multiplied by a factor of 1.6, whereas the dead load is multiplied by a load factor of 1.2. The *LRFD Specification,* 3rd Edition, does not include load factors (as was done in earlier editions), but refers to ASCE 7, which is the national standard for loads on structures.

The structural engineer uses engineering models that predict how a component will reach its strength capacity and the internal force at which failure will occur. In these models, tension members fail by yielding, columns fail by buckling and connections fail by either yielding or fracture (or a combination of both). Beams can fail by either yielding or buckling, both of which can be caused by the internal shear or moment produced by the external load. All limits states must be investigated to determine the smallest externally applied loads that would cause the member's strength to be exceeded. The failure condition that requires the least externally applied load to reach its capacity is the critical limit state. The nominal strength of a member is defined as the strength predicted by the model appropriate for the way the member is used. Section 16 of the *LRFD Manual* (3rd edition) is the Specification that contains limit state engineering models in the form of design equations that are used to predict the nominal strength for each limit state by which a steel component can fail.

The tension member shown in Figure B-1 will be used as a simple introduction to computing the nominal strength of a steel member. As can be seen in Figure B-14, the stress project such as a stadium or arena, where a highly specialized design relies on computer methods to complete a very complex structural analysis. The incorporation of CIS/2 data formatting and electronic information sharing into the design process will lead to a more wide-spread use of structural steel in the construction market and will ultimately create greater profits for steel fabricators and detailers, greater savings to owners, added value for architectural services, and greater efficiency and value for engineers. For more information on CIS/2 data formatting, please visit AISC's web resourse on electronic data exchange at: http://www.aisc.org/edi.html.

SCALE

Scale is a critical factor for information obtained electronically from outside parties on any given project team. Architects, Construction Professionals and Engineers occasionally modify scales in drawings in order to make information fit or to make details more clear. The slightest deviation in the scaled versus actual dimension of a drawing demands a thorough review of the drawing and adjustment of the data to accommodate the scale variation. This can cause detailing errors, which can lead to major problems for steel erection and major cost increases to a project.

When using drawings from outside parties who do not assume the responsibility of drawing precision, detailers must beware of the possibility that drawings will contain "exploded dimensions"– dimensions in which the actual scaled distance measurement of the CAD-drawn line is different from the numeric dimension shown. Design professionals may employ imprecise dimensions in the development of drawings for conceptual applications of design. When the dimensions have been violated, the usefulness of digital information sharing for detailers becomes highly limited. When using information created by an outside party, it is the responsibility of the user of that information to make a judgment on the accuracy of the information. Generally, structural models are much more accurate than drawings as the models themselves rely on members drawn between defined node locations to complete a technical design rather than a conceptual or aesthetic presentation of information.

Often a designer will not allow the distribution of electronic files in an attempt to avoid the responsibility of file precision or to avoid potential liability in the event that the user modifies the drawings. Common practice at this time is for CAD drawings to be considered informational only and are superceded in their validity by contract drawings. For more information on liability issues with respect to the sharing of electronic information please see the AISC Code of Standard Practice, Section 4.3. As electronic data exchange develops, standards for format and precision of data will evolve and information processing and distribution will provide increased productivity.

QUALITY CONTROL

With the use of new technologies comes an increased responsibility to ensure that steel detailing does not circumvent the thought process necessary for the development of an accurate and useful product. As is true for any computer software application, the "Garbage In, Garbage Out" principle applies - the quality of the end product is only as good as the information entered to develop it. The use of CIS/2 data formatting will help to improve the quality of information by promoting effective checking by the drafter and verification of compliance with design intent by the engineer.

WHERE WE ARE TODAY

Today only the most advanced building projects make use of the tools available with the development of technology. The most sophisticated proponents of this process can even enter electronic information into a CNC (Computer Numeric Control) automated fabrication system that allows shop drawing information to be accurately uploaded directly to the machinery that fabricates the steel.

In an industry that thrives on tradition adoption of this type of technology will take time. Inefficiencies will persist until competition makes the technology necessary. The thrifty professionals of today should take steps now to prepare themselves for the day when the technology of tomorrow becomes the reality of the present day. Rev. 9/1/03

GLOSSARY

Some of the terms listed herein are extraneous to the text of this manual. However, they are terms which the steel detailer may expect to encounter during their involvement with structural steel detailing.

Advance bill or Advance bill of material. A list prepared by the Detailing Group showing the steel mill products to be ordered expressly for the requirements of a specific project.

AESS. Architecturally Exposed Structural Steel. See the AISC Code of Standard Practice.

AISC Specification. The AISC Specification for Structural Steel Buildings.

AWS D1.1. American Welding Society Structural Welding Code - Steel.

Alloy steel. Steel to which has been added silicon, manganese, nickel or another element or elements.

Anchor. A device for fastening steelwork to masonry or concrete.

Anchor rod. A rod used to fasten steel columns, girders, etc. to masonry or concrete.

Angle. A common structural steel shape, the cross section of which is in the form of a right angle.

Anneal. The process of softening metal or making it more uniform by heating and cooling slowly.

Approval. A process whereby shop and erection drawings are submitted to the owner's designated representatives for design and construction for review and approval. See AISC Code of Standard Practice.

Arrow-side. That part of the welding symbol below the reference line, which describes the weld to be placed on the side of the joint to which the arrow points.

Assembling. Putting the component parts of a member together in the shop preparatory to bolting or welding.

Assembly marks. A system of identifying marks used on the component parts of a member to facilitate assembling in the shop.

ASD. Allowable Stress Design.

ASTM. American Society for Testing and Materials.

AWS. American Welding Society.

Axial forces. Forces that are applied longitudinally at the center of a member.

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Back-charge. Costs directed to a fabricator or steel detailer for errors in fabrication or detailing.

Back-check. A steel detailer's checking of the comments and corrections made by a checker to a document prepared by the steel detailer.

Bar. A round or square rod. Also, a rolled flat (1) up to and including 6-in. in width by 0.203 in. and over in thickness and (2) over 6-in. to 8-in. in width by 0.230 in. and over in thickness. Bars have rolled edges.

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Base angle. An angle used to connect the bottom of a column to the base plate.

Base plate. A load-distributing plate upon which a column bears.

Batten plate. A plate element used to join two parallel components of a built-up column, girder or strut rigidly connected to the parallel components and designed to transmit shear between them.

Beam. A structural member whose primary function is to carry loads transverse to its longitudinal axis.

Bearing. The support upon which a member rests. Also, the resistance to crushing offered (1) by a member that bears against another or upon a support, or (2) by a component part of a member that bears on a bolt or a pin.

Bearing plate. A load-distributing plate, as at the end of a wall bearing beam.

Bearing stress. The stress that occurs when two metal surfaces are in contact and a compressive force is applied perpendicular to the contact surfaces. Also, the stress that occurs where the shanks of bolts in holes or slots come into contact with the surrounding material.

Bearing value. The amount of pressure in bearing, either total or per unit of area.

Bending moment. A term that expresses the measure of the tendency of a beam, girder or column to bend. It is the sum of the moments of all external forces on one side of the point of moments.

Bent. A planar framework of beams or trusses and the columns that support these members.

Bevel. The slope of a line with respect to another line.

Beveled washer. A washer beveled on one side.

such movement.

Deformed bar. One of many types of reinforcing bars which are rolled with projections to increase the bond to the surrounding concrete.

Derrick. A hoisting machine so pivoted that a load may be swung horizontally. Two types are a guyed derrick and a stiff-leg derrick.

Design-build. A type of contract in which an owner retains a general contractor to assume responsibility for the design and construction of the structure.

Design drawing. A drawing prepared by the owner's designated representative for design to show the dimensions, configuration, sizes, connections and other aspects of the structure.

Design strength. Resistance (force, moment, stress, as appropriate) provided by a structural element or connection. In LRFD the product of the nominal strength and the resistance factor.

Designer. The owner's designated representative for design.

Detail. To make shop and erection drawings. Also, a connection or other minor part of a member in contrast to the main member.

Detailer. See Steel Detailer

Detailing group. An organization of structural steel steel detailers and checkers whose purpose is to supply a fabricator with accurately prepared shop drawings.

Detailing manager. One who manages a detailing group, and whose responsibilities include becoming familiar with the project plans and specifications and scheduling the detailing work to meet the fabricator's schedule.

Develop. To represent on a drawing a bent or curved piece as if it were flattened into place.

Diagonal bracing. Inclined structural members carrying primarily axial load employed to enable a structural frame to act as a truss to resist horizontal loads.

Diagram. A drawing in which each member usually is represented by a single line, as in a erection drawing or a load diagram.

Diaphragm. An element that stabilizes a system or elements of a system, primarily through shear; e.g. floor slabs and shear walls. Also, a stiffening plate or similar part placed between the webs of a member, or from one member to another.

Die. A steel form used in forging or cutting any piece.

Dimension. A linear measurement indicated on a drawing upon a dimension line to show its extent and significance.

Direct tension indicators. A washer-type element inserted under the bolt head or hardened washer, having several small arches, which deform in a controlled manner when subjected to load. See the RCSC Specification for further information.

Double shear. The tendency to shear, or the resistance to shear, a single element or group of elements on two planes.

Drafting. Making working drawings, usually including the design of the details.

Drafting project leader. A member of the detailing group who plans and organizes work assignments and supervises a group of structural steel detailers and checkers in the work of the group. Rev. 9/1/03

Draw. The tension induced in diagonals of bracing systems to insure initial tightness.

Drift. Lateral deflection of a building.

Drift pin. A tapered pin used in assembling members in the shop or during erection. It is driven into the connection holes to bring them into alignment to permit installation of permanent bolts.

Drill. To make a hole by means of a rotating cutting tool or drill bit.

Ductility. A characteristic of steel in which, when steel is under load and stressed beyond its yield strength, strain increases disproportionately greater than stress, resulting in permanent distortion. When the load is released, the steel does not revert to its original shape.

Eave strut. A longitudinal member between the tops of columns at the eaves of a building.

Eccentric connection. A connection in which the line of action of a resultant load does not pass through the centroid of a group of connecting bolts or welds.

Eccentricity. The perpendicular distance from a resultant force to some other point or line.

Edge distance. The perpendicular distance from the center of a hole to the edge of the piece that contains it.

Effective weld length. The distance from end to end of a fillet weld measured parallel to its root line. For curved fillet welds the effective length is measured along the centerline of the throat.

Elasticity. A property of steel in which steel will return to its original shape following deformation by an exter-

Filler. A plate or combination of plates used to fill the space between two surfaces.

Fillet. The additional metal that forms the curve at the junction of the flange and web of a rolled shape.

Fillet weld. A weld having a theoretically triangular cross-section joining two surfaces that abut perpendicular to each other.

Finger shim. Shim consisting of a narrow piece of structural steel with slots open through the edge.

Finish. To smooth a surface by milling or other suitable means. To face.

Fit check. A partial checking of the shop drawings to ensure the proper connection of the members in the field.

Fitter. A shop workman who assembles the component parts of a member and bolts or welds them in position.

Fixed-ended beam. The term describing a beam or girder in which the connections at the ends are rigid and end rotation is prevented.

Fixture. A special device built in the fabricating shop to locate and clamp component parts of a welded assembly prior to welding. Usually used for fabricating several members having the same configuration, such as a group of roof trusses.

Flame. An gas flame or torch used for cutting steel by melting a narrow slot by means of an intense heat.

Flame-cut plate. A plate in which the longitudinal edges have been prepared by gas cutting from a larger plate.

Flange. The wide part of a rolled W, M, HP, S or channel shape at each edge of the web. Also, the corresponding portion of a built-up girder or H-shaped column, each flange being composed of plates.

Flange plate. A plate used as the flange of a built-up girder or similar member. Or, a plate used on the flange to reinforce of connect the flange.

Flange weld. A weld that attaches a flange. Also, a weld that attaches a flange plate to the web plate of a built-up girder or column.

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Flat. A narrow plate with rolled edge.

Flexible connection. A connection permitting a portion, but not all, of the rotation of the end of a member.

Flexure. Bending. Commonly applied to describe the bending of a beam, girder or column.

Flitch plate. Steel plates used as reinforcement for timber beams.

Floor beam. A beam participating in the support of a floor.

Floor plan. A plan showing the arrangement of beams, girders, bracing, columns, etc. in a floor.

Floor plate. See Raised pattern floor plate.

Floor slab. A reinforced-concrete floor supported by beams, girders and columns.

Flush top. The tops of connecting members, such as beams and girders in a floor arrangement, which are at the same elevation.

Footing. The concrete pier or foundation for a column.

Force. Resultant of distribution of stress over a prescribed area. A reaction that develops in a member as a result of load. Generic term signifying an axial load, bending moment, torque or shear.

Forging. An article formed by being hammered while hot. Often, a die is used for shaping the article.

Foundation plan. A plan showing the layout of the foundations that support a structure.

Gable. The triangular portion of the end of a building between the opposite slopes of the roof.

Gage. The transverse distance that locates the line(s) of fasteners in a rolled or built-up shape. The gage is measured from the back of an angle or channel but between the gage lines in a symmetrical shape such as a W, M, HP or S shape.

Gage lines. The lines on which bolt holes are placed in a rolled or built-up shape.

Galling. Scouring of base metal under the head or nut of a high-strength bolt assemble during installation.

Galvanize. The process of applying a coating of zinc to steel products where protection of the surface from corrosion is required.

Gantry. A self-propelled crane supported on two bents and traveling on two rails.

General contractor. The entity with full responsibility for construction of the structure according to project plans and specifications, on schedule and within budget or the contracting entity who contracts for the general trades portion of a project.

Girder. Historically, a member, usually made with a web and flanges composed of plates, used to resist bending due to transverse loads as a beam. In modern usage, this term refers to a flexural member that supports other beams.

Girt. A horizontal member in the side or end of a building used to support side covering.

Nailing strip. A strip of wood bolted to a steel beam or other member, to which wooden flooring or sheathing is nailed.

Necking-down. During a test in a tension machine of a bar of steel, the perceptible thinning of the bar prior to rupture. It occurs when the bar continues to elongate despite a drop in the stress required to continue the elongation.

Net area (or Net section). The reduced area in a cross section in which the rectangular areas of all bolt holes and other material removal cut by the section are deducted from the gross area of the member or part of member under consideration.

Nominal loads. In LRFD the magnitude of the loads specified by the applicable code.

Nominal strength. In LRFD the capacity of a structure or component to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or by laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

Ordered length. The length of a steel piece as ordered from the mill or steel distribution center.

Orthographic projection. The method of representing the exact shape of an object in two or more views on planes generally at right angles to each other by dropping perpendicular projections from the object to the plane.

OSL. Abbreviation for outstanding leg of an angle.

Other side. That part of the welding symbol above the reference line, which describes the weld to be placed on the side of the joint opposite from the side to which the arrow points.

Outlooker. A small angle or similar piece fastened to an end purlin of a building to support the roof, which overhangs the gable end.

Overrun. The amount of increase in the actual length of a structural shape over the theoretical dimension indicated on the drawing or advance bill. Also, the amount of increase in the actual cross section dimensions from those published in ASTM A6, A500 or A53 (as summarized in the AISC Manual of Steel Construction).

Owner's designated representative for construction. The Owner or the entity which is responsible to the Owner for the overall construction of the project, including its planning, quality and completion. This is usually the general contractor, the construction manager or similar authority at the site.

Owner's designated representative for design. The Owner or the entity which is responsible to the Owner for the overall structural design of the project, including the Structural Steel frame. This is usually the Structural Engineer of Record.

Oxyacetylene flame or torch. Equipment used for cutting steel by burning a narrow slot by means of an intense heat.

Panel point. The intersection of the working lines of different members of a truss.

Parabola. A curve in which the coordinates vary as the squares of the abscissas, or conversely.

Peak. The top point of a roof truss where the top chords meet.

Piece mark. See Assembly Mark.

Pier. A concrete column footing

Piles. Logs or steel shapes driven into the ground to give greater support for the foundations of a structure.

Pin. A solid steel cylinder used for connecting the members of a truss.

Pin plate. A reinforcing plate bolted or welded to a truss member to provide greater bearing on a pin.

Pitch. The longitudinal distance between adjacent bolts in the main part of a member. Also, the ratio of the center height of a roof truss to the half-span.

Plan. A drawing that represents the horizontal projection of a structure or part of a structure. Often less accurately used for any general drawing of a structure whether plan or elevation.

Plans. Design drawings furnished by the party responsible for the design of the structure.

Plane. To smooth to a planar surface.

Plate. Rolled steel of flat, rectangular cross section with sheared or gas-cut edge. Rev. 9/1/03

Plate girder. See girder.

Pneumatic tools. Tools made to operate with compressed air, as a chisel for removing projecting parts, a wrench for tightening bolts or a fluted reamer for enlarging holes.

Point of Inflection. See Contraflexure.

Post. A comparatively short vertical, or near vertical, compression member. Also, a compression member less than 300 lbs. in weight per OSHA.

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