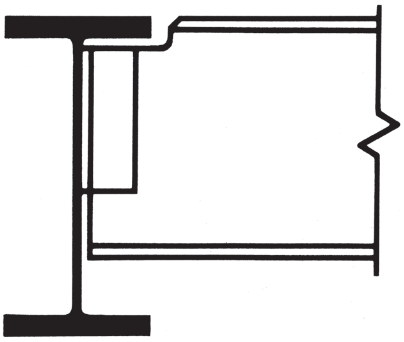
DETAILING



FOR STEEL CONSTRUCTION

THIRD EDITION

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
AISC
FOUNDED 1921

NATIONAL INSTITUTE OF STEEL DETAILING
NISD

DETAILING

FOR STEEL CONSTRUCTION

Third Edition

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PREFACE TO THIRD EDITION

The purpose of the Third Edition of *Detailing for Steel Construction* is to update the Second Edition to be consistent with the most current AISC publications. In particular, this edition references the following:

2005 AISC *Specification for Structural Steel Buildings*

2005 AISC *Seismic Provisions for Structural Steel Buildings*

PREFACE TO SECOND EDITION

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CHAPTER 1
INTRODUCTION

An overview of the structural steel design and construction process, common references, structural materials, fabrication, and erection.

THE CONSTRUCTION PROCESS AND THE STEEL DETAILER'S ROLE

When you look at the outside of a building, what you see is its facade or "skin." Behind that facade (which may be brick, concrete, glass, metal panels, stone or a combination thereof) is a frame or "skeleton" consisting of steel, concrete, masonry, wood or a combination of these materials. This book will address structural steel detailing—the preparation of drawings for the fabrication and erection of this frame.

Traditionally, the steel construction team consists of the owner, architect, engineer, contractor, fabricator, steel detailer, erector and inspectors. Sometimes, the team includes a construction manager, who represents the owner and is responsible for having the project completed on time and within budget. There are several ways that an owner may choose to structure a contract with the steel construction team to deliver a project. The most typical approach, known as Design-Bid-Build is described here. Another popular approach called Design-Build will be described later in this text.

When an owner decides a building is needed to serve their purposes, they usually contact an architect. The owner and architect meet to discuss the function of the building, what the shape and size of the structure should be, how the interior should adapt to the proposed usage, and how the exterior of the building should appear. The architect prepares a set of plans and specifications to show and describe all the features of the building discussed with the owner—the layout and dimensions of the interior spaces, the types of materials to be used, the colors of the interior and exterior, and the details of the skin. The architect then selects a structural engineer to design the supporting structure. The structural engineer determines forces in the components of the supporting structure, sizes elements to resist these forces, and develops design details of connections.

The owner also selects a general contractor to construct the building; the selection method is discussed in Chapter 2. The general contractor is responsible for constructing the structure according to plans and specifications and for delivering the building to the owner for occupancy on schedule and within budget. To do this, the general contractor awards several portions of the building to pertinent subcontractors—HVAC, plumbing, electrical, masonry, foundation, structural steel, roofing and others. The general contractor coordinates the requirements and efforts of these and other related trades. The structural steel subcontract is awarded to a steel fabricator, whose responsibility it will be to accurately fabricate the various structural steel components for on-time delivery to the job site to meet the contractor's construction schedule. The fabricator is responsible to the owner, the owner's agent, or a general contractor and has a duty to keep these parties fully informed of all changes that impact a project's cost and schedule. The AISC *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005a), hereafter referred to as the AISC *Code of Standard Practice,* the standard of custom and usage for structural steel fabrication and erection, stipulates in Section 9.3 the procedures the fabricator and erector are expected to follow in response to revisions to the contract documents.

A person who prepares shop drawings for a steel fabricator is known as a steel detailer. Steel detailers use the design drawings and specifications made by the structural engineer to prepare shop and erection drawings for each piece of a project that their employer has agreed to furnish. In other words, the steel detailer translates design data into information that the fabricator and erector need to actually build the structure. The steel detailer may be either an employee or a subcontractor of the fabricator. To prepare shop and erection drawings the steel detailer works closely with the owner's designated representative for design (ODRD)—normally the structural engineer of record (SER)—who reviews and approves the shop and erection drawings.

At the job site a steel erector receives the material from the fabricator and places it in the proper location in the building. The erector may work for either the general contractor or the steel fabricator. Besides erecting the steel members, the erector must plumb and properly align the structure, ensuring that all joints fit properly and welds are made and bolts installed according to industry standards and specifications. Throughout the process of constructing a building, inspectors may check the materials and workmanship at the job site, and in the shops of the various subcontractors.

The steel detailer has a key role in this process, and it is extremely important that the steel detailer's work be performed completely and accurately. The steel detailer's work is performed early in the construction process and used subsequently by members of the steel construction team and by other subcontractors. Errors can endanger the structure and cause expense to the fabricator.

The steel detailer must be familiar with the fabricator's practices and equipment in the shop. Also, the steel detailer must know what size and weight limits the erector can handle at the job site. This and other erection information can be obtained 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 direct communication 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 earlier 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 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 and 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.

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 *Steel Construction Manual,* 13th Edition (AISC, 2005b), hereafter referred to 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. Universal (UM) plates are rolled between horizontal and vertical rolls and trimmed (sheared or gas cut) on ends only. 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. See the *Manual* for guidance on 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, with particular attention given to the "Remarks" column.

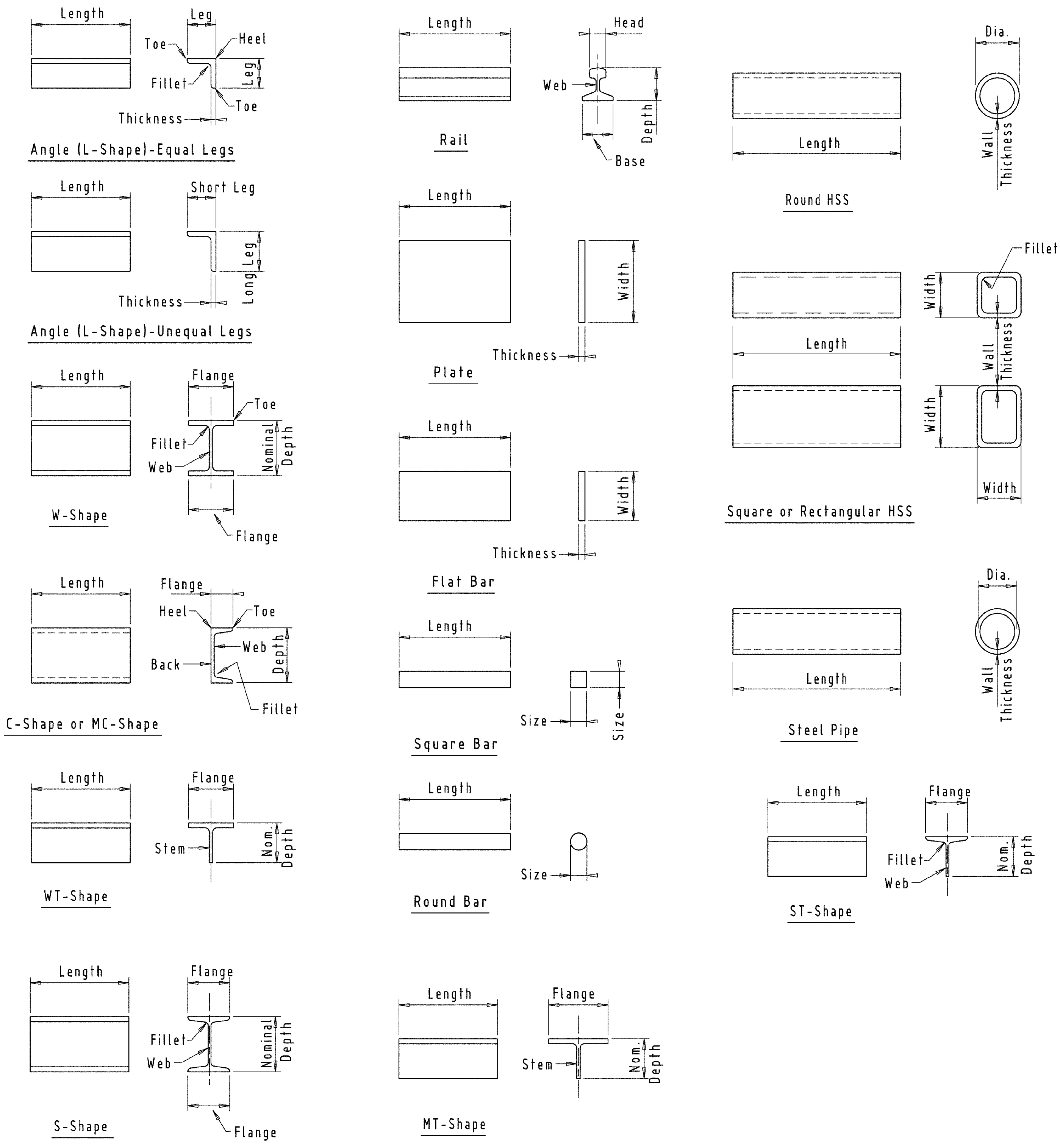


Figure 1-1. Typical cross-sections of raw steel material.

CHARACTERISTICS OF STEEL

Steel, specifically structural steel, is fundamental to building and bridge construction. It is produced in a wide range of shapes and grades, which permits maximum flexibility of design. It is relatively inexpensive to produce and is the strongest, most versatile and economical material available to the construction industry. Steel is essentially uniform in quality and dimensionally stable; its durability is unaffected by alternate freezing and thawing.

Steel also has several unique qualities, which make it especially adaptable to the demanding requirements of modern construction. It can be alloyed or alloyed and heat-treated to obtain toughness, ductility and great strength, as the service demands, and still be capable of fabrication with conventional shop equipment.

PHYSICAL PROPERTIES

The terms yield stress and tensile strength are used to describe the physical properties of steels and their response when subjected to externally applied forces. For example, assume that a rectangular or round specimen of structural steel, having an area of 1 in.^{2} and being any convenient length, is clamped in a testing machine designed to pull the bar apart longitudinally. If the machine is adjusted to pull the bar so that it is resisting a force of 10 kips (1 kip = 1,000 pounds), the bar, with a cross-sectional area of 1 in.^{2}, is said to be stressed in tension at an average intensity of 10 kips per in.^{2} (ksi). If the force is increased to 20 kips, the bar is stressed to 20 ksi, and so on.

The bar, loaded as described earlier, is being elongated, or strained, in direct proportion to the stress being resisted. As the machine load increases, the bar will be stressed and strained proportionally. Within certain limits, the external forces will deform the piece of steel slightly, but on removal of such forces the steel will return to its original shape. This property of steel is termed elasticity. Eventually, a point is reached beyond which the elongation will continue with no corresponding increase in stress. This elongation is characteristic of ductile steels. Within this range, upon removal of the force, the steel does not return to its original shape.

Mechanical testing of most steels produces a sharp-kneed stress-strain diagram, as shown in Figure 1-2. The stress at which this knee occurs is termed the yield point, and varies numerically for different specifications of steel. High-strength steels may not exhibit such a well-defined knee. For such steels, a yield strength is established in conformance with the provisions of ASTM A370, *Standard Test Methods and Definitions for Mechanical Testing of Steel Products* (ASTM, 2008a).

So as not to confuse the issue between these two concepts, the AISC *Specification for Structural Steel Buildings* (AISC, 2005c), hereafter referred to as the AISC *Specification*, has established the common definition yield stress, which is understood to mean either yield point (for steels that have a yield point) or yield strength (for steels that do not show a sharp knee in the stress-strain relationship). The symbol F_y is used to designate this yield stress and it is expressed in kips per in.^{2} (ksi).

In the elastic range, the stress-strain relationship is constant at normal temperatures and is the same for tension or compression loadings. Furthermore, the stress-strain relationship is substantially the same, regardless of yield stress. The ratio of stress to strain is called the modulus of elasticity, designated by the letter E. Numerically:

$$E = \text{Stress/Strain} \approx 29{,}000 \text{ ksi}$$

Figure 1-2 is a theoretical diagram of the stress-strain relationship of ASTM A572 steel. The stresses at yield stress and tensile strength shown on the curve are the minimums specified in ASTM A572, *Standard Specification for Structural Steel Shapes*. Often, actual test results exceed the values shown. Strain is plotted horizontally in units of inches per inch; stress is plotted on the vertical scale in ksi. A straight line, representing the elastic range, starts from the point of zero stress and zero strain and inclines upward to the right. At a stress of 29 ksi, for example, the strain is 0.001 inch for each inch of specimen length. At this stress, a 10-in. length of the 1-in.^{2} bar will be increased in length:

$$10 \times 0.001 = 0.01 \text{ in.}$$

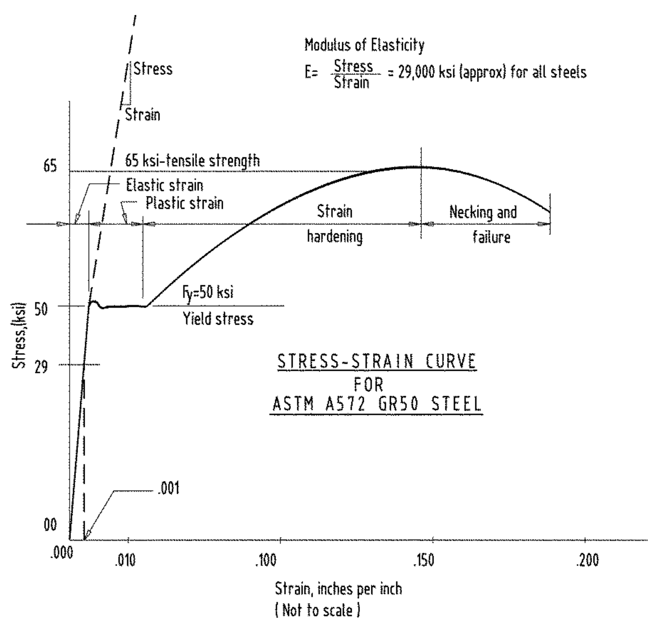


Figure 1-2. Stress-strain diagram for ASTM A572 Grade 50 steel.

At the upper end of the inclined straight line, the yield stress, $F_y = 50$ ksi, is shown graphically by an uneven horizontal line, or plateau, which represents the range of plastic strain. This plastic deformation tends to cold work the steel, causing it to strain-harden sufficiently to require an additional application of load for continual elongation. Throughout this strain-hardening range, the curve makes a long upward sweep until the tensile strength of 65 ksi is reached. Further elongation, or straining, is accompanied by a perceptible thinning or necking-down of the bar, a drop in the stress needed to continue the elongation, and soon thereafter the fracture of the bar.

That portion of the curve immediately following the yield stress illustrates another important property of structural steel—ductility. In this range the metal is said to be in a state of plastic strain; elongation is no longer in direct proportion to stress. Equal increments of stress are accompanied by disproportionately greater strains. Permanent distortion occurs and, on load release, the steel bar no longer reverts to its original length. This characteristic, termed ductility, provides a considerable reserve of strength, a fact that explains the ability of structural steel to absorb temporary overloads safely. The ability of steel to support loads throughout large deformations forms the basis for plastic design. Ductility is measured in percent of elongation at rupture. For ASTM A992 steel this is specified to be at least 20% in a length of 8 in., which means that the steel must have the ability to elongate at least $0.2 \times 8 = 1.6$ in. in 8 in. of specimen length before fracturing.

SPECIFICATIONS FOR STRUCTURAL STEEL

Structural steel is composed almost entirely of iron. Today, most structural steel is made from recycled steel, which was made from iron ore (or scrap iron), limestone, fuel and air. Heated until it liquefies, the steel is then cooled. Small portions of other elements, particularly carbon and manganese, must also be present to provide strength and ductility. Increasing the carbon content makes steel stronger and harder. Decreasing the carbon content makes steel softer or more ductile, but at some sacrifice of strength. The standard grades of steel used for bridges and buildings contain approximately one-fourth of 1% of carbon, with small amounts of several other elements as required or permitted by the particular steel specifications.

All steels are manufactured to specifications that stipulate the chemical and mechanical requirements in detail. Standard specifications for structural steels are established by the American Society for Testing and Materials (ASTM). Committees of ASTM, composed of representatives of producers, consumers, and general interest groups, develop and keep current material specifications to provide and maintain reliable, acceptable and practical standards. Reference to the latest ASTM specifications is recommended for those interested in complete information on all structural steels.

An important specification is ASTM A6*, Specification for General Requirements for Standard Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling* (ASTM, 2008b). It covers in detail all aspects of mill practice and the allowances or tolerances applicable to rolled steel with which the fabrication process must deal.

The AISC *Specification*, as well as most bridge specifications, recognizes several grades of steel for structural purposes. The ASTM specifications list the scope and principal properties of these steels. As these specifications indicate, the tensile strength and yield stress levels within a specific grade of steel may vary with the size of shapes and the thickness of plates and bars.

Tables 2-3 and 2-4 in the *Manual*, Part 2, serve as a quick reference to determine the availability of shapes, plates and bars by steel type, ASTM designation, and minimum yield stress. A brief review shows that:

- ASTM A992 covers rolled steel structural shapes for use in building framing or bridges, or for general structural purposes. A992 is used for wide-flange shapes and has $F_y = 50$ ksi.
- ASTM A36 is a carbon steel with one minimum yield stress, 36 ksi, for all shape groups (but W-shapes are produced to ASTM A992 today) and for plates and bars through 8 in. thick. Plates and bars over 8 in. thick have a minimum yield stress level of 32 ksi.
- ASTM A500 is used for hollow structural sections. For square and rectangular, grade B offers $F_y = 46$ ksi. For round, grade B offers $F_y = 42$ ksi.
- ASTM A53 is the steel used for steel pipe, with $F_y = 35$ ksi.
- ASTM A529, also a carbon steel, has a minimum yield stress of 50 ksi and 55 ksi, but is limited to shapes with a flange thickness of 12 in. and less and to plates 1 in. thick and less. This Specification is not the preferred specification for shapes.
- ASTM A572 is a high-strength, low-alloy steel with four minimum yield stress levels ranging from 42 to 65 ksi. All hot-rolled open shapes are available in 42-ksi, 50-ksi and 55-ksi grades; however, only shapes with a 2-in. maximum flange thickness are available in grades 60 and 65. The limits of availability of plates and bars, by thickness, are given also. This Specification is not the preferred specification for shapes other than HP shapes with $F_y = 50$ ksi.
- ASTM A588 is a corrosion-resistant, high-strength, low-alloy steel with a single minimum yield stress level for shapes and three levels for plates and bars. These stress levels are 50 ksi, 46 ksi and 42 ksi. This steel is unique since the highest yield stress level applies to all shapes and to plates and bars through 4 in. thick. Plates and bars over 4 in. thick have reduced minimum yield stress.

- ASTM A514 is a quenched and tempered alloy steel in the 90- to 100-ksi minimum yield stress range. Note that this specification includes plates and bars only. Special care must be taken in the welding of this steel so as to maintain its characteristics derived from heat treatment.
- ASTM A913 is a low-alloy steel produced by the quenching and self-tempering process. This applies to oversized or “jumbo” steel sections, which are not currently produced in the United States. This steel is produced to a minimum yield stress level of 33 ksi.

Several proprietary steels, so-called because their composition and characteristics are defined by steel producers’ specifications, are available for structural purposes. Producers of these proprietary steels use rigid control of melting processes and careful selection of alloys to achieve minimum yield stresses ranging in excess of 100 ksi. The toughness, weldability and cost-to-strength ratios of proprietary steels compare favorably with those obtainable from standard steels.

Steel making is in a continual state of progress. Metallurgical research in the industry continues to develop new steels for specific purposes and to improve the versatility of existing steels. As time passes and these products prove themselves, writers of ASTM specifications prepare modifications of present specifications or formulate new ones to recognize technological advances.

STEEL PRODUCTION

The processes by which steels are made are complicated and highly technical. Depending upon the end use of steel, several aspects of the processes are subject to variations. Rolling the raw steel into finished products shown in Figure 1-1 involves additional highly technical operations. The steel detailer interested in learning about the steel manufacturing industry is encouraged to read *The Making, Shaping and Treating of Steel* (AIST, 1998). This authoritative reference provides detailed information on the production and rolling of steel.

Commercial practice has established a series of fixed-size shapes with a sufficient range of dimensions and intermediate weights per foot to satisfy all usual requirements. The extent of standardization achieved is evident from a study of the listings under “Dimensions” or “Properties” in the *Manual*, Part 1. Note the relatively small gradations in dimension of the successive shapes included under any one nominal size.

This standardized series of shapes is far from static. From time-to-time, improvements in production technology and changes in construction trends result in the introduction of new shapes and elimination of less efficient shapes, as well as the extension of established popular series of shapes by the inclusion of new lighter or heavier sections.

MILL TOLERANCES

The term mill tolerances is used to describe permissible deviations from the published dimensions of cross-sectional profiles listed in mill catalogs and in Part 1 of the *Manual*, and from the thickness or lengths specified by the purchaser. Some of the variations are negligible in smaller shapes, but tend to increase and must be taken into consideration in detailing and fabricating connections for members made up from larger shapes. Other mill tolerances permit some variation in area, weight, ends out-of-square, camber, and sweep. Factors that contribute to the necessity for mill tolerances are:

- The high speed of the rolling operation required to prevent the metal from cooling before the rolling process has been completed.
- The varying skill of the operators in adjusting the rolls for each pass, particularly the final pass.
- The deflection (springing) of the rolls during the rolling operation.
- The gradual wearing of the rolls, which can result in some weight increase, particularly in the case of shapes.
- The warping of steel in the process of cooling.
- The subsequent shrinkage in length of a shape that has been cut while still hot.

Rolling, cutting and other tolerances attributable to mill production of structural shapes and plates are discussed in the *Manual*, Part 2 under “Tolerances.” The steel detailer should be familiar with the several tolerances, especially those of camber, sweep, depth of section, and length. A more exhaustive presentation of these tolerances is found in ASTM A6.

An important factor for the steel detailer to understand clearly is the effect of mill tolerances. The steel detailer must know when to take tolerances into account, particularly in ordering mill material and in detailing connections, especially those involving heavy rolled shapes. For instance, when detailing a moment connection (discussed in Chapter 3) the steel detailer must be cognizant of the permissible variations in the depth of the beam and out-of-square of the beam flanges in order to locate the connection material shop welded to the column.

CALCULATION OF WEIGHTS

Listed here are several reasons the weights of finished members must be calculated:

- They provide a check of the accuracy of the original estimated weights against the actual as-built weights.
- Freight is paid on a weight basis.

- The shop, shipping department and the erector must know the weight of heavy pieces to prevent overloading equipment.
- They are used by management in connection with progress controls and cost control.
- The weight of finished parts is required for invoicing purposes. On a unit price contract, where invoices are based on weight of steelwork, the accuracy of calculated weights is extremely important.

When manually prepared drawings are completed, clerks enter the information from the shop bill into a computer to produce a printout that displays the weight of each component of a shipping piece and the total weight of the piece. Shop drawings prepared with CAD systems automatically provide these weights (Figure A1-2). The steel detailer seldom performs the calculation of weights. Later, these weights are entered on the shipping bills (Figure 1-3). Most fabricators use calculated weights and the vast majority of weights used in the industry are calculated in accordance with certain definite, agreed-upon procedures.

Theoretically, determination of the weight of a finished part by calculation is as accurate as using a scale weight. However, simplifying steps, such as the elimination of deductions for material removed by cuts, clips, copes, blocks, milling, drilling, punching, boring, planing, or weld joint preparation (all of which have little effect upon the final weight), are followed as accepted practice in the standard procedure outlined in Section 9.2 of the AISC *Code of Standard Practice*.

BILLS FOR SHIPPING AND INVOICING FINISHED PARTS

Field bolt lists (discussed further in Chapter 8) are part of the shipping bill, shipping memorandum, or bill of finished parts. These bills are prepared by the fabricator's billing department after the shop drawings have been completed. They cover every item of structural steel that must be delivered under the contract. The fastener lists are usually the only part of a shipping list that are prepared by the steel detailer.

As with other forms already discussed, the design and arrangement of the shipping documents vary according to the system of controls in any one plant. In general, however, they provide space for listing the following data:

- The total number of identical pieces to be shipped.
- A brief description of each piece.
- The erection mark and general location of each shipping piece.
- The weight of each finished piece.
- The total weight shipped.

An example of shipping documents is shown in Figure 1-3. As the arrangement and display of information on shipping documents depends upon the preference of a fabricator, those illustrated in Figure 1-3 show the type of information one would expect to find on such documents. If a project requires using more than one crane for erection, the shipping document may list the crane to which each piece is assigned or the sequence number may identify the crane. Some fabricators prefer to list the numbers of the shop drawings corresponding to the shipped pieces if the number is not a part of the shipping mark.

Figure 1-3a (Bill of Materials by Sequence) is a computer-generated list of all shipping pieces for Sequence 1, one of several erection locations into which shipments on Job #1847 have been separated. The weights listed are the total weights of all the pieces in the shipment. Thus, the weight shown for pieces marked C42A is for the three pieces.

Figure 1-3b (Bill of Lading) is a computer generated load list for the first truckload of material on Sequence 1 on Job #1847. On this list both the individual piece weights and the total weights shipped are listed. The steel detailer will note that some of the quantities listed on the Bill of Materials by Sequence exceed those shown on the Bill of Lading (see piece A290, for example). The balance of pieces will go to the job site on another truck. The total weight of 44,100 lb is approaching the limit allowed by law to be shipped by the truck in use. Note that the receiver at the job site is required to sign the Bill of Lading to acknowledge receipt of the material.

CNC FILES

CNC (Computer Numeric Control) is a method by which a steel fabricator sends information to specific semi-automated machinery to perform certain fabrication tasks. These tasks may include cutting members to length; drilling or punching of holes; and cutting plates to size, beam copes, long slots, etc.

CNC is not new to the fabrication of structural steel. It has been provided by what is referred to as interactive methods. In the past, shop drawings were sent to the fabrication shop and numeric information was entered into a computer by hand or interactively. The classical method can and does provide for the possibility of making mistakes. The programmer/operator, typically someone in what is called the "template shop," would then provide tapes or some other means of transferring the information to the individual CNC pieces of equipment. With this digital information the machinery would, when the material is loaded, perform the indicated operation.

In today's world of electronically produced shop drawings, CNC information can be provided automatically by the detailing software. If the detailing software being used is capable of providing CNC information, the need for a

Bill of Materials by Sequence

Job # 1847 **Page # 1**

Property of ABC Fabricators **09/01/00 10:13:21**

Mark	Quan	Type & Size		Grade	Length	Weight
Sequence 1						
C42A	3	W	12x65	A992	32' 10-1/4"	6,406.56#
C43B	1	W	12x45	A992	35' 10-1/2"	1,614.38#
C44B	1	W	12x152	A992	35' 10-1/2"	5,453.00#
C45A	1	W	12x152	A992	38' 4-1/2"	5,833.00#
C46A	1	W	12x152	A992	35' 10-1/2"	5,453.00#
C51B	1	HSS	10x10x3/8	A500-Gr B	19' 5-7/8"	933.55#
C54A	1	HSS	8x6x1/2	A500-Gr B	28' 9-3/8"	1,210.25#
C59A	5	W	12x96	A992	19' 6"	9,360.00#
C82B	1	W	12x152	A992	35' 4-1/2"	5,377.00#
B103B	3	C	8x11.5	A36	12' 6-1/4"	431.97#
B105A	4	W	16x40	A992	31' 6-3/4"	5,050.00#
B106B	2	W	14x22	A992	18' 9-1/2"	826.83#
B106E	1	W	21x44	A992	28' 10"	1,268.67#
B107A	6	W	14x22	A992	18' 10-1/2"	2,491.50#
B209A	2	W	24x76	A992	20' 10-3/8"	3,171.42#
A290C	5	L	4x4x3/8	A36	16' 9-1/8"	821.26#
A290D	4	L	7x4x5/8	A36	8' 11-7/8"	794.68#
Total:	**42**	**Shipmarks**			**Material:**	**56,497.07#**

Figure 1-3a. Sample bill of materials.

Bill of Lading

ABC Fabricators		Job #	1847
P.O. Box 000		Load #	1
Weldersville, USA 12345		Trailer #	001
Phone:	123-456-7890	Load Date	09/14/00
Fax:	123-456-7891	Ship Date	09/19/00

SOLD TO:	SHIP TO:
XYZ Building Company	AAA Erectors
45 Joist Lane	325-N Connection Drive
Girderville, USA	Gussetville, USA 11111
Phone: (222)-222-2222	Phone: (111)-111-1111
Fax: (333)-333-3333	Fax: (555)-555-5555
	Attn: Jeff Doe

Page # 1

Quan	Mark	Seq	Description	Grade	Length	Wt/Each	Weight
2	A290C	1	L 4x4x3/8	A36	16' 9-1/8"	164.25#	329#
4	A290D	1	L 7x4x5/8	A36	8' 11-7/8"	198.67#	795#
3	B103B	1	C 8x11.5	A36	12' 6-1/4"	143.99#	432#
2	B105A	1	W 16x40	A992	31' 6-3/4"	1,262.50#	2,525#
1	B106B	1	W 14x22	A992	18' 9-1/2"	413.42#	413#
1	B106E	1	W 21x44	A992	28' 10"	1,268.67#	1,269#
4	B107A	1	W 14x22	A992	18' 10-1/2"	415.25#	1,661#
1	B209A	1	W 24x76	A992	20' 10-3/8"	1,585.71#	1,586#
3	C42A	1	W 12x65	A992	32' 10-1/4"	2,135.52#	6,407#
1	C43B	1	W 12x45	A992	35' 10-1/2"	1,614.38#	1,614#
1	C44B	1	W 12x152	A992	35' 10-1/2"	5,453.00#	5,453#
1	C45A	1	W 12x152	A992	38' 4-1/2"	5,833.00#	5,833#
1	C46A	1	W 12x152	A992	35' 10-1/2"	5,453.00#	5,453#
1	C54A	1	HSS 8x6x1/2	A500-Gr B	28' 9-3/8"	1,210.25#	1,210#
2	C59A	1	W 12x96	A992	19' 6"	1,872.00#	3,744#
1	C82B	1	W 12x152	A992	35' 4-1/2"	5,377.00#	5,377#
29							44,100#

NOTICE TO RECEIVERS: Please check each item on Bill of Lading carefully.
Shipper will not be responsible for any shortages unless noted above.

Received by: ______________________ Date: ___/___/_____
Name in Full

Complete: Partial: 44,100#

Figure 1-3b. Sample bill of lading.

programmer in the shop to transfer the required data from the shop drawings to the computer is eliminated. CNC also reduces the possibility of an error in data transfer. This will, for the most part, eliminate the need for a programmer in the shop, but it also means that the shop drawings must be made accurately and to scale. Furthermore, all holes, cuts, lengths, and other fabrication criteria must be incorporated electronically for inclusion with the CNC information. If shop drawings are plotted and changes are made to these plotted/hard copies, then the automatic CNC information may be rendered useless. In today's market these hand changes are rarely performed when accurate CNC information is required. If for some reason drawings are not made to scale, the CNC information is corrupted and cannot be sent to the shops for fabrication.

CNC is a great tool providing speed of fabrication and better quality control. If fabrication information is transferred digitally from the detailing computer directly to the CNC control computer, either through a network system or stored data on some sort of digital media, there is little room for error and quality control is greatly improved.

FABRICATING STRUCTURAL STEEL

The versatility of a structural steel fabrication shop is its most notable characteristic. Few other types of industrial shops are called upon to perform such a variety of work. For example, the fabrication of a long-span bridge may be concurrent with the fabrication of an industrial facility or a multi-story building. The speed and accuracy with which these structures are fabricated and erected is a tribute to the steel detailers who detail the work and the shop workers who perform it.

Knowledge of shop operations will help the steel detailer to understand the reasons for many conventional practices used in the preparation of shop drawings. Also, knowledge of the available shop facilities and equipment will enable the steel detailer to detail pieces that can be fabricated and erected easily and economically. Drawings must be made to suit the capacities and requirements of shop machines.

Fabricating shops differ considerably in size and layout. Nevertheless, most conform to the same general pattern of operations. A typical fabricating plant consists of one or more bays or aisles, which are often called shops. The lengths of the bays vary to accommodate required equipment and provide the desired capacity. Usually, bays average 60 to 80 ft in clear width and are serviced by overhead traveling bridge cranes spanning the full width of the bay. Often jib cranes are attached to and swing in an arc about individual building columns for servicing various machines placed within reach.

In large multiple-bay shops, various classes of work are segregated and passed through that bay which is equipped to handle the particular type of work required. In small shops, all classifications of work usually pass through one bay. Repair work, minor fabrication, and storage of bolts and small parts are handled, generally, in lean-tos or a small section of the shop normally serviced by monorail hoists or fork lift trucks.

At the receiving end of the shop, an area is provided where incoming raw material can be unloaded from railroad cars or trucks, sorted, and stored until fabrication. At the shipping end of the shop, a similar area is provided where fabricated members can be loaded onto railroad cars, trucks or barges.

Structural steel must pass through several operations during the course of its fabrication. The sequence and importance of shop operations vary, depending on the type of fabrication required. This wide variation in operations distinguishes the structural steel fabrication shop from a mass production shop. A list of typical fabrication shop operations follows. A brief description of the work performed is then given under subheadings identifying each operation.

- Material handling and cutting
- Template making
- Laying out
- Punching and drilling
- Straightening, bending, rolling and cambering
- Fitting and reaming
- Fastening methods
- Finishing
- Machine shop operations
- Cleaning and painting (if required)
- Shipping

MATERIAL HANDLING AND CUTTING

Three broad classifications describe the sources of steel used in a structural fabricating shop: mill order steel, stock steel, and warehouse steel.

Mill order steel is purchased from the rolling mills for specific jobs at specific quantities, sizes and lengths from lists prepared by the steel detailer or fabricator's purchasing department. It provides most of the material used in the fabrication shop. While material used to be ordered cut to length and ready for fabrication, material today is almost exclusively ordered in standard lengths (and widths for plates) with cutting to length done in the shop.

Stock steel is stored at the fabricator's plant and used to handle requirements beyond those covered by mill order steel. Also, it is used to fill small orders and rush orders and to supply quantities too small to order economically from the mill.

Warehouse steel is purchased from established warehouses (steel service centers), usually at a premium price. Normally, warehouses purchase steel from rolling mills in stock lengths, such as 40 ft, 50 ft or 60 ft. Warehouse steel generally costs more and the fabricator may have a greater waste factor if the available lengths are more limited than those for a mill order.

It is used either for jobs where a customer desires a quicker delivery than is possible with mill order steel and is willing to pay extra for the service or for when quantities are too small for a mill order.

When steel arrives at the plant, it must be identified and checked against the fabricator's order list and segregated for a particular job or stock.

ASTM A6 specifies that steel, as shipped from the rolling mill, must be marked with the heat number, manufacturer's name, brand or trade mark, and size. In addition, when a yield stress of more than 36 ksi is specified, each plate, shape or lift (a bundle of several pieces) is marked with the applicable material specification number and color code. Mill test reports show the results of physical and chemical tests for each heat number and are furnished to positively identify the steel.

Sections A3.1 and M5.5 of the AISC *Specification* provide for identification of high-strength steels during fabrication. These systems of identification and control of high-strength steel identification during fabrication ensure that the materials specified for the various members are identified in the fabricator's plant.

Most material passing through a structural shop is too heavy to lift and move by hand. Overhead cranes, buggies operating on tracks, motorized tractors, fork lifts, and straddle carriers take the material as received in the shop and deliver it to the various machines. Also, they handle the material during its movement through the shop and finally deliver the finished fabricated members to the shipping yard.

Material not cut to length at the mill must be sent to the shears, saws or cutting tables. Plates or flat bars under a certain thickness are cut on a guillotine-type machine called a shear. Angles are cut on a similar machine capable of cutting both legs with one stroke. Automated angle punching and shear lines can cut and punch angles from information furnished to it by the computer. Material is fed into the machine on a bed of rollers. Beams, channels and light column shapes are usually cut on a high-speed friction saw, a slower speed cold saw or a band saw.

A gas torch is used to cut curved or complex forms and material of a size or thickness beyond the capacity of the aforementioned cutting machines. This operation is termed flame cutting. The cutting torch provides a most useful and versatile means of cutting steel. The portable type can be taken to the material, either in the shop or in the yard. One stationary model has a pantograph arm with cutting nozzle at one end, directed by a guide template at the other end. Some gas cutting machines are mounted on power-driven carriages designed to run on small guide tracks. For relatively straight cutting, a guide rail on an adjacent table controls the cutting torches. For complex cutting, an electronic guide tracer follows a full scale template laid on the adjacent table. More often, though, fabricators use CNC machines controlled by computers that automatically control the cutting head and eliminate the need for templates.

TEMPLATE MAKING

A template is a full-size pattern or guide, made of cardboard, wood or metal, used to locate punched or drilled holes, and cuts or bends to be made in the steel. It is used when layouts are not made by CNC equipment.

Unless the fabrication operations are CNC-machine based, template making is the first major shop operation required when a new job starts. Detail drawings should be sent to the shop early enough to ensure an ample supply of templates before actual shop operations begin. The template is the sole guide to many subsequent operations, such as the cutting of plates, fabrication of bent work, and punching or drilling of holes.

Each template is marked with the size of required material, number of pieces to be made, the job number, the piece identification mark and the drawing number on which the part is detailed.

Computer plots have eliminated the need for manual template making in some operations. In addition, patterns for templates of complicated curves in plate work can be made using plots of data supplied to a computer by a steel detailer.

LAYING OUT

Unless the fabrication operations are CNC-machine based, a substantial portion of the steel routed through the shop for fabrication passes through the hands of the layout crew. Some layout work is performed without the use of templates. This is true when there is little duplication and layout work is more economical. Construction lines are marked directly on the steel with chalk lines or soapstone markers. Then, a centerpunch is used to locate the centers of holes to be punched and the lines along which cutting must be done.

The layout crew checks the raw material for size and straightness. If a piece needs to be straightened, it must be sent to straightening machines, which are discussed later in this chapter.

Material that is to be laid out from templates is placed on skids and the templates are clamped in place. All holes are centerpunched and all cuts are marked with a soapstone marker. All centerpunch marks and cuts are "rung-up" (outlined with painted lines) to prevent their being overlooked in later operations.

PUNCHING AND DRILLING

Punching is a common method of making bolt holes in steel (refer to AISC *Specification* Section M2.5). High-strength steels are somewhat harder and punching may be limited to thinner material. Except when holes other than standard holes are specified, round holes are punched with a diameter $^1/_{16}$ in. larger than the nominal diameter of the bolt to be used. This provides clearance for inserting fasteners with some tolerance for slightly mismatched holes.

Light pieces of steel, such as short-length angles and small plates, may be single-punched, that is, punched one hole at a time. Machines for this purpose are known as detail punches.

A multiple punch has a number of punches arranged in a transverse row over a spacing table. The table extends beyond both sides of the punch and has adjustable rollers to support the steel. A hand- or power-driven carriage moves the steel through the punch, and hole locations are determined by stops set by a template or by a steel tape. Several holes can be punched simultaneously.

A hand- or power-operated spacing table is used for medium-weight beams, channels, angles and plates. An automatic spacing table handles larger and heavier pieces. The introduction of electronic controls in some shops permits fully automatic operation of the spacing table carriage.

Drilling of structural steel is confined, largely, to making holes in material thicker than the capacity of the punches, or to meet certain job specification requirements. Drilling equipment includes the standard machine shop fixed-drill press, radial arm drills, multiple-spindle drills, batteries of drills on jibs used for mass drilling and reaming, and gantry drills.

The fixed-drill press and radial arm drill usually drill one hole at a time. For pieces requiring numerous holes, a multiple-spindle drill may be used. One type has rows of spindles with the longitudinal spacing between them fixed at 3 in. center-to-center. With this type of equipment the material must be moved into position under the drills. In contrast, horizontally movable drills on jibs and radial drills mounted on a gantry frame permit the drills to be moved over the material.

Machine manufacturers have combined many formerly separate functions into continuously operating lines for the processing of main material. One such machine, commonly called a beam line, moves the material on a conveyor through a saw, then punches or drills all holes. In this equipment, the drill or punch equipment may consist of one spindle or punch, or several spindles or punches, arranged to drill or punch beam or column flanges and webs simultaneously. Another machine is the single-spindle, CNC-controlled high-speed drill, which will drill holes in gusset plates without the need for templates or layout, including those for skewed connections. One advantage of these highly automated machines is their inherent accuracy. The associated elimination of dimensional errors greatly simplifies successive shop operations, as well as erection.

STRAIGHTENING, BENDING, ROLLING AND CAMBERING

Material not meeting ASTM A6 tolerances and material that may have become bent or distorted during shipment and handling, or in the punching operation may require straightening before further fabrication is attempted. In addition, members may become distorted when they are trimmed or, in the case of W, S and M shapes, when they are split into tees. The bend press, generally used for straightening beams, channels, angles and heavy bars, is known commonly as a bulldozer, gag press or cambering press. This machine has a horizontal plunger or ram (or a set of rams or plungers) that applies pressure at points along the bent member to bring it into alignment. Also, the press is used to form long-radius curves in various structural members.

Long plates, which are curved slightly or cambered out of alignment longitudinally, are frequently straightened by a roll straightener. The plates are passed between three rolls. The resulting bending increases the length of the concave side and brings the plates back to acceptable tolerances of straightness.

Misalignments in structural shapes are sometimes corrected by spot or pattern heating. When heat is applied to a small area of steel, the larger unheated portion of the surrounding material prevents expansion, causing a thickening of the heated area. Upon cooling, the subsequent shrinkage produces a shortening of the member, thus pulling it back into alignment. Commonly, this method is employed to remove buckles in girder webs between stiffeners and to straighten members. Heating must be controlled. A special crayon that changes color or melts at a predetermined temperature is often used as a temperature check.

A press brake is used to form angular bends in sheets and plates. Curved plates used in tanks and stacks are formed in a plate roll machine.

The foregoing operations can also be used to induce curvature, rather than remove it.

FITTING AND REAMING

Before final fastening, the component parts of a member must be fitted-up; that is, the parts assembled temporarily with bolts, clamps or tack welds. During this operation, the assembly is squared and checked for overall dimensions. Then, it is bolted or welded into a finished member.

The fitting-up operation includes attachment of assembling pieces (such as splice plates, connection angles, stiffeners, etc.) and the correction of minor defects found by the inspector.

On bolted work some holes in the connecting material may not be in perfect alignment, and small amounts of reaming may be required to permit insertion of the fasteners. In addition, holes may be formed by subpunching and reaming. In this operation, the holes are punched at least $^1/_8$ in. smaller than final size. After the shipping piece is assembled, the holes are reamed with electric or pneumatic reamers to the correct diameter to produce well-matched holes. The resulting elongation of holes in some of the plies is acceptable, provided the resulting hole size does not exceed the tolerances for the final hole sizes given in the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, hereafter referred to as the RCSC *Specification*. If reaming results in a larger round dimension or a longer slot dimension, the rules for the larger hole size must be met.

To ensure precise matching of the holes, some specifications require that field connections be reamed to a metal template or that connecting members be shop assembled and reamed while assembled. Either of these operations adds considerably to the cost of fabrication and are generally specified only for unusually large and important connections, most often encountered in bridge work. The use of CNC-controlled drilling virtually eliminates the need for such operations.

FASTENING METHODS

The strength of the entire structure depends upon the proper use of fastening methods. Where options are permitted by the specifications, a steel detailer should select the most economical fastening method suited to the shop.

Bolting

Bolted connections are used in both the shop and field. Connections are usually made using high-strength bolts, ASTM A325 or A490, depending on strength requirements. Ordinary machine bolts (ASTM A307) are seldom used today, perhaps only in minor structural applications such as connections for girts and purlins. Installation and strength requirements for high-strength bolts are specified in the RCSC *Specification*.

The RCSC *Specification* and Section J3 of the AISC *Specification* specify that the required joint type for high strength bolts be identified in the design drawings as snug-tightened, pretensioned or slip-critical. Snug-tightened joints and pretensioned joints resist forces through bearing of the fasteners. Slip-critical joints resist forces in much the same way, but also have frictional resistance to slip on the faying surfaces. In building structures, snug-tightened joints are most common; see RCSC *Specification* Section 4.1. Applications where pretensioned joints are required are listed in RCSC *Specification* Section 4.2 and AISC *Specification* Section J3.1. Applications where slip-critical joints are required are listed in RCSC *Specification* Section 4.3. Note how rarely slip-critical joints are required in building design.

Welding

Welding generators, transformers and automatic welding machines are provided with adjustable controls. These controls are used to obtain welding power characteristics and rates of weld deposit best suited to the electrodes and to the type and position of work being welded. The welding current is conducted from the generator or transformer through insulated cables. These are connected to complete a circuit between the work and the machine when an electric arc is struck between the electrode (the conductor that delivers the electric current used in welding) and the work to be welded. Long welds of uniform size are deposited, generally, by automatic welding machines that feed welding wire and flux into the arc at an electronically controlled speed. Other methods, more completely described in Chapter 4, may be used.

When a number of identical welded assemblies are to be fabricated, special devices known as fixtures or jigs are used to locate and clamp the component parts in position.

The layout work for welded fabrication consists, chiefly, of marking the ends and edges of components for accurate cutting. Drilling or punching of main material is avoided and holes for erection bolts are confined to fitting or connection material, when practicable. Subassemblies are placed on level skids and tack-welded together. This holds the part in alignment, facilitates completion of the final welding operations, and reduces distortions.

An inspection of each shipping unit prior to final shop welding is made to check overall dimensions and the proper location of all connections. This inspection also includes a check of the fit-up of all joints to ensure that they can be welded properly. When the welding has been completed, a final inspection is made and each piece is cleaned and painted, if required. When shop painting is required, the surface areas adjacent to future welds may need to be left unpainted until after these welds have been made. This provides surfaces free of materials that might prevent proper welding or produce objectionable fumes during welding. Shop drawings must show such unpainted surfaces.

FINISHING

Structural members whose ends must transmit the weight and forces that they are supporting by bearing against one another are finished to a flat surface with a roughness height value less than 500 μin., per ASME B46.1 (ASME, 1996). Such finishing is normally obtained by sawing, milling, or other suitable means.

Several types of sawing machines are available, all of which produce very satisfactory finished cuts. One type of milling machine employs a movable head fitted with one or more high-speed, carbide-tipped rotary cutters. The head moves over a bed, which securely holds the work in proper alignment during the finishing operation.

When job specifications require that sheared edges of plates over a certain thickness be edge planed, the plate is clamped to the bed of a milling machine or a planer. The cutting head moves along the edge of the plate, planing it to a neat and smooth finish.

Column base plates over certain thickness limits are required by the AISC *Specification* to be finished over the area in contact with the column shaft. This finishing is usually done on a machine known as a bed planer.

The term “finish” or “mill” is used on detail drawings to describe any operation that requires the steel to be finished to a smooth, even surface by milling, planing, sawing, or other suitable means.

MACHINE SHOP OPERATIONS

Some plants may be equipped with a machine shop as an auxiliary facility to the main fabrication shop. Special operations of machining are performed here as required in connection with the general run of structural steel fabrication.

One of the important functions of the machine shop is the maintenance and repair of plant equipment. In addition the machine shop may bore holes in parts for pin connections, turn out pins and other lathe work, plane or mill base plates, and cut and thread tie rods and anchor rods. In larger plants the machine shop may be equipped to manufacture machinery for movable bridges, railroad turntables, rockers and rollers for bridge shoes, and similar special items.

CLEANING AND PAINTING

All steel that is to be painted is so indicated on the design drawings and the shop drawings. Before painting, the steelwork must be cleaned thoroughly of all loose mill scale, loose rust, and other foreign matter. The cleaning may be done by hand or power-driven wire brushes; by flame descaling; or by sand, shot or grit blasting. Certain specifications may require a specific type of treatment, as in the case of paints requiring a surface free of mill scale. The kind and color of paint, as well as the method of painting, are controlled by job specifications, which are part of the contract documents. For an expanded discussion on steel coatings, refer to Chapter 4.

SHIPPING

The shipping dock or yard requires a large area serviced by cranes or other material handling equipment. Here, the fabricated members are sorted, stored and shipped to the field as required.

Material destined for distant points is transported by railroad cars, trucks or barges. Material for local structures is almost always hauled by truck. This requires loading facilities for each type of transportation used.

Long members, which slightly exceed the length of a railroad car, are loaded with the overhanging length at one end; an idler car goes with the load to provide clearance for the overhanging end. Longer members, which approximate the length of two cars, are loaded to rest on a bolster on each car. The bolsters are arranged to rotate slightly and to move lengthwise at one end to permit the cars to go around curves. Even longer members are loaded on three cars; bolsters support the load on the two end cars, which are separated by an idler car. Sketches of large pieces are submitted to railroads for loading instructions and clearance confirmation. These sketches are sometimes prepared by the steel detailer.

Shipping foremen must be familiar with railroad and highway regulations. They must have information on maximum permissible loads and bridge clearances. When material is wider, longer and heavier than is permitted on streets or highways, permission for special routing must be obtained from the proper local, state or federal authorities.

CHAPTER 2

CONTRACT DOCUMENTS AND THE DETAILING PROCESS

Summary and definition of the information needed on design drawings and the typical steps involved in the detailing process.

A NEW PROJECT

When a steel fabricator supplies the structural steel for a project, the fabricator must be aware of their responsibilities as a member of the project team. The AISC *Code of Standard Practice* outlines the normal fabricator obligations that become applicable when the AISC *Code of Standard Practice* is referenced in the contract documents. Explicit requirements in the contract documents may be included that tailor the AISC *Code of Standard Practice* requirements to meet the needs of a specific project. Such requirements are in addition to (or may supersede) those in the AISC *Code of Standard Practice*.

As noted in Chapter 1, the major portion of work placed under contract by a structural steel fabricator is with an owner, normally through the owner's designated representative for construction, to provide the structural steel indicated in the design drawings and specifications prepared by the owner's designated representative for design. One common alternative system is a design-build project, which provides a way for the owner to retain a single representative who assumes responsibility for both the design and the construction of the structure.

Typically, the owner or the owner's representative advertises in construction and contracting periodicals that a structure is proposed for construction and requests bids. The advertisement describes the scope and location of the project, states the date bids are due, and gives the location where design drawings and specifications can be obtained by contractors for bidding. Interested contractors obtain sets of design drawings and specifications for their own use and for distribution to subcontractors who are invited to bid to the general contractor on their (the subcontractor's) portion of the work. Thus, the structural steel fabricator obtains a set of design drawings and specifications pertaining to the portion of the project in which the fabricator is interested. This interest could be in the structural steel only or, if requested by the general contractor, could also include other construction items such as miscellaneous steel (ladders, stairs, handrails, relieving angles, curb angles, loose lintels, etc.), open-web steel joists, steel sash, corrugated steel siding and roofing, steel decking, and/or erection of any or all of these items. The fabricator will usually sublet the work of these other construction items to specialty subcontractors who perform these types of work.

ESTIMATING

When a project is advertised for bidding, the owner must provide sufficient information in the form of scope, structural design drawings, specifications, and other descriptive data to enable the fabricator and erector to prepare a bid. As the first step in preparing a bid to furnish structural steel for a given project at an agreed price, the fabricator's estimating department prepares a detailed list, or "takeoff," of all pertinent material shown in the structural design drawings and determines the associated costs and labor.

Where the basis of payment is lump sum, it is particularly important that this takeoff be accurate and complete. A lump-sum price covers a specific amount of work explicitly shown on the design drawings and covered in the project specifications. The omission or addition of items may result in taking a contract at a loss or losing a contract.

Another basis of payment is unit-price. Frequently, this method is used when a design is incomplete or when additions and changes are expected. In unit-price contracts, the final calculated weight of the structural steel in pounds (or tons) multiplied by the bid price per pound (or ton) determines the total cost. Unit-priced payment is most common in industrial work.

Occasionally, the basis for payment is the actual cost of material and all labor plus a percentage of these costs. This is termed a cost-plus price.

The estimator, from past experience and with the aid of cost data from previous similar jobs, determines the cost of preparing shop drawings and fabricating the structural steel. Cost estimates are prepared either by:

- Applying appropriate cost factors to the estimated steel weight; or,
- Estimating the cost of preparing shop drawings from analyzing the quantity, sizes and shapes of pieces to be fabricated, and making a complete and detailed analysis of shop costs.

If the bidding fabricator has an in-house detailing group (Figure 2-1), the estimator may request that the group create an estimate of the costs to produce shop drawings. On the other hand if bidding fabricators rely on subcontract steel detailers to produce their shop drawings (and time permits) they may ask these steel detailers to prepare an estimate on the preparation of shop details and erection drawings. In addition, a cost analysis is prepared for the other construction bid items (miscellaneous steel, joists, decking, erection, etc.) when they are to be bid by the fabricator. If time permits, the subcontractors for these items may be invited by the fabricator to submit bids. Usually, the lowest price the estimator

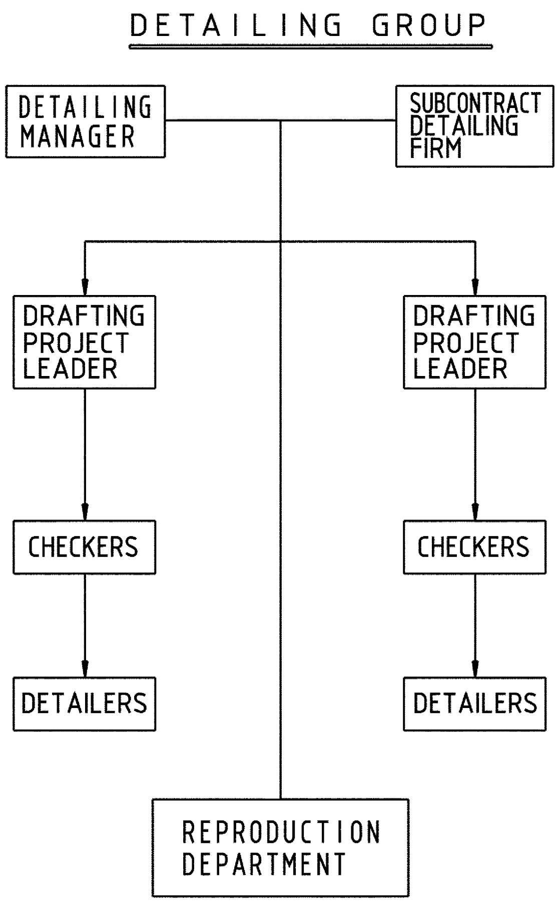


Figure 2-1. Detailing group hierarchy.

receives for producing shop and erection drawings and supplying any of the other construction items will be included in the fabricator's bid prices to the general contractor. However, sometimes the lowest price will be rejected for some reason such as the bidder's inability to perform within the allotted time frame.

Before an award, the sales manager (or "contracting manager" as some fabricators call it) usually has the only contact with the customer (the owner, owner's designated representatives for design, and/or owner's designated representative for construction). When the award is made, all of the information required to perform the work is forwarded to the steel detailer and shop in accordance with an agreed-upon schedule. A project manager or coordinator is assigned to schedule the work and to provide contact between the fabricator's departments and the customer.

CONTRACT BETWEEN THE FABRICATOR AND THE CUSTOMER

The contract documents normally detail what the fabricator is to furnish, the delivery schedule, and the manner and schedule by which the fabricator will receive payment. Having won the contract to furnish the items bid, a fabricator informs its winning subcontractors and sets its system of production controls into motion. As the first step, a contract number is assigned to the job and used to identify all shop and erection drawings, documents, raw material, and finished parts relating to the project.

For reasons relating to price, delivery time or character of the work involved, the project may be divided into multiple contracts. In such cases, a separate number is assigned to each contract. This establishes a separate identity for the work throughout the drafting, production and erecting operations. In most shops, the sales department prepares an operating data sheet (sometimes referred to as a job data sheet, production order, or contract memorandum) similar to the form illustrated in Figures 2-2a, b and c. As noted elsewhere in this manual, the arrangement and presentation of an operating data sheet will vary depending on the preference of the fabricator.

The data usually lists basic information such as, but not limited to:

- Project
- Customer
- Owner
- Structural Engineer of Record
- Architect
- Contract design drawings
- Contract specifications
- Location
- Job number

Also, it may briefly summarize information such as:

- Grade(s) of steel to be used.
- Type of paint required (if any) and type of surface preparation.
- Type of field connections to be furnished.
- Inspection required and by whom.
- Agreed-upon schedule for drawing submittals, fabrication, delivery, and duration of erection.
- Method of delivery of steel to the job site.
- Basis of payment.
- Any additional information needed to prepare shop drawings.
- The requirement that certified copies of all mill test reports are to be furnished.
- Individual or company to whom shop drawings are to be submitted for approval.
- Scope of work and exclusions.
- Fabrication and shipment sequences (divisions) of steel.
- Accepted deviations from design drawings.
- Erection requirements.

Operating Data Sheet

JOB NAME:	ABC High Rise	JOB # :	1847
LOCATION:	Anywhere, U.S.A.	PROJECT MGR.:	
CUSTOMER:	XYZ Building Company		
Home Office:	45 Joist Lane Girderville, U.S.A. 00000	Job Site:	325-N Connection Drive Gussetville, USA 11111
Home Office Phone:	(222) 222-2222	Job Site Phone:	(181) 818-1818
Home Office Fax:	(333) 333-3333	Job Site Fax:	(191) 919-1919
E-Mail:	www.fantasticbuilders.com	Project Supt. :	Jane Doe
		Project Manager:	Bill Doe
		Project Engineer:	Joe Doe
ARCHITECT:	Outstanding Architects		
Contact:	Matt Doe	Phone:	(444) 444-4444
Address:	90 Drawing Road	Fax:	(777) 777-7777
	Beamville, U.S.A. 11111	E-Mail:	www.outstandingarchitects.com
ENGINEER:	Excellent Engineers		
Contact:	Jonathan Doe	Phone	(888) 888-8888
Address:	120 Design Street	Fax:	(999) 999-9999
	Columnville, U.S.A. 22222	E-Mail:	www.excellentengineers.com
DETAILER:	Superb Detailers		
Contact:	Beth Doe	Phone:	(121) 212-1212
Address:	180 Anchorbolt Drive	Fax:	(131) 313-1313
	Baseplate, U.S.A. 33333	E-Mail:	www.superbdetailers.com
JOISTS:	Marvelous Joist Company		
Contact:	Jim Doe	Phone:	(141) 414-1414
Address:	240 Bridging Street	Fax:	(151) 515-1515
	Channel, U.S.A. 44444	E-Mail:	www.marvelousjoist.com
DECK:	Fabulous Deck Company		
Contact:	Anne Doe	Phone:	(161) 616-1616
Address:	360 Blueprint Drive	Fax:	(171) 717-1717
	Angle, U.S.A. 55555	E-Mail:	www.fabulousdeck.com
ERECTOR:	Very Good Erectors		
Contact:	Jeff Doe	Phone:	(111) 111-1111
Address:	420 Craneville Lane	Fax:	(555) 555-5555
	Boom, U.S.A. 66666	E-Mail:	www.verygooderectors.com
OTHER:	Super Stud Welders		
Contact:	Dave Doe	Phone:	(212) 121-2121
Address:	540 Ferrule Way	Fax:	(313) 131-3131
	High Power, U.S.A. 77777	E-Mail:	www.superstud.com
Tonnage:	800 Tons	Shop Bolt	$^3{}_4$ ϕ A325-N Bearing Type u.n.o.
Material Grade:	A992	Field Bolt	$^3{}_4$ ϕ A325-N Bearing Type u.n.o.
Paint:	One Shop Coat Standard Primer	Submittals:	Prints: 3 Transparencies: 1

Figure 2-2a. Sample operating data sheet.

Operating Data Sheet

SCHEDULE	
Award (not later than):	05/30/00
Mill order placed:	06/12/00
Connection details submitted for approval:	06/12/00
Anchor bolt plan submitted for approval:	06/14/00
Anchor bolts & leveling plates delivered to site:	07/14/00
First shop drawing submittal:	07/05/00
Final shop drawing submittal:	08/07/00
Commence fabrication:	07/31/00
Complete fabrication:	09/15/00
Commence erection:	09/11/00
Crane leaves site:	10/20/00
All work complete:	11/10/00

CONTRACT DOCUMENTS	
Drawings:	S-1 thru S-14 All dated: 04/26/00 (Rev. 1)
Specifications:	Structural steel – 05120 dated: 04/26/00 Joists – 05220 dated: 04/26/00 Metal Decking – 05300 dated: 04/26/00
Sketches:	SKS-1 thru SKS-5 All dated: 05/01/00
Addenda/Supplements, etc. :	Supplement 1 dated: 05/03/00 Supplement 2 dated: 05/08/00

Figure 2-2b. Sample operating data sheet, continued.

Operating Data Sheet

SCOPE OF WORK
FURNISH AND INSTALL 1. Structural steel 2. Joists / joist girders w/ accessories 3. Metal deck w/ accessories 4. Perimeter hung lintel system (lintel angles galvanized) 5. Moment connections where shown 6. Galvanized roof screenwall framing
DELIVER ONLY: 1. Column anchor bolts 2. Leveling plates
TERMS AND CONDITIONS : 1. A mutually acceptable contract 2. Good truck and crane access inside and around structure on firm, level ground 3. Line and grade provided by others. 4. Hung lintel system to be aligned and welded off brick masons scaffolding in conjunction with the brick installation. 5. A) Steel joists / joists girders: one shop coat standard gray primer B) Floor deck: galvanized, G60 C) Roof deck: painted (standard w/ manufacture) D) Structural steel: unpainted (no surface preparation)
EXCLUSIONS: 1. Anchors and bolts for other trades 2. Field touch-up painting 3. Grout / grouting 4. Shoring 5. Loose lintels 6. Embedded items other than mentioned above 7. Openings, penetrations or reinforcement of same unless shown and located on structural drawings. 8. Elevator sill angles 9. Masonry ties, anchors or CMU seismic clips 10. Miscellaneous metals of any kind 11. Reinforcement of joists at point loads

Figure 2-2c. Sample operating data sheet, continued.

Most of this data is given by the owner's designated representative for design on the design drawings or in the project specifications, which are a part of the contract documents.

Certain practices relating to the design, fabrication and erection of structural steel have become standardized, such as the furnishing of incidental materials (bolts, weld electrodes, etc.), method of weight calculation, use of stock, etc. These standards are described fully in the AISC *Code of Standard Practice*, which normally is included in the contract documents by reference. Suppose, for example, the scope of a contract is defined in the job specification as:

> Furnish and deliver all structural steel shown on Drawings S101 to S109, inclusive, in accordance with the AISC *Specification for Structural Steel Buildings*, and AISC *Code of Standard Practice for Steel Buildings and Bridges*.

Such a contract provision establishes a commonly accepted and well-defined line between what is, and what is not, to be furnished under a contract for structural steel. Without such a provision, the contract would need to spell out in considerable detail what is expected of both parties to prevent a misunderstanding.

Under the preceding contract provisions, the categories of material to be furnished by the fabricator are defined in AISC *Code of Standard Practice* Section 2.1. The items listed therein are produced in the fabrication shop or are related directly to these items. Unless specifically called for in the contract documents, other non-structural-steel items—such as steel sash, corrugated steel siding and roofing; open-web steel joists, and other items listed in AISC *Code of Standard Practice* Section 2.2—even though they may appear on the design drawings or in the project specifications, are not included in the contract, even though some actually may be made of steel. However, as noted earlier when requested to do so, some fabricators will include in their bid package the costs of purchasing and delivering these "other items."

Because all of this contract information must be available to many individuals, summarizing it in a single memorandum is advisable. Its importance requires that the data sheet be revised and kept up to date throughout the period of the contract.

PLANS AND SPECIFICATIONS

The most important contract documents are the design drawings and specifications, which define the work. Generally, the specifications describe how the work is to be accomplished, while the design drawings show how the structure will look. They show the shape of the structure, sections and sizes of members, location and arrangement of the members in the frame, beam/girder camber, floor levels and roof, and column centers and offsets, with adequate dimensions to convey accurately the quantity and character of the structural steel to be furnished. Also, details of structural joints, bearing stiffeners on beams and girders, beam web reinforcement, openings for other trades, connections between the curtain wall and the supporting frame, column stiffeners, column web doubler plates, column anchorage, and column splices are given. Notes listing the types of fasteners, applicable design specifications (for example, AISC, RCSC, AWS, ASTM), grade of steel, and type of paint (if any) to be used are included with other instructions specific to the construction of the structure.

Sufficient information concerning loads and forces to be resisted by the individual members and their connections should be given in the design drawings. Such forces include shear and axial forces in beams and girders, shear forces and moments at column splices and bases, moments at beam ends, and axial forces in diagonal bracing. See AISC *Manual* Part 2 and AISC *Code of Standard Practice* Section 3.1 for further information.

DESIGN INFORMATION

Figure A7-66 in Appendix A of this manual is a design drawing of a light industrial building. This drawing, prepared by the designer, gives the fabricator the necessary information to prepare shop drawings for the structural frame.

The composite plan view (Figure A7-66) shows both the top and bottom chord bracing and the braced bays requiring sway frames. The size of the eave struts is indicated on the plan, but their location is shown on the Typical Wall Section. The size and location of the purlins and girts are shown in the top chord plan and in the Side Elevation. Note that sag rods are used to align the purlins and girts.

The cross section is taken through the 60-ft width of the structure and shows the sizes of the columns, knee-braces and truss components. The designer has indicated the required forces and loads in the truss and knee-braces. These are needed by the steel detailer to develop adequate connections. Two sets of forces are indicated: those produced by gravity (vertical) loads and those caused by wind. The wind forces are given by the designation ($\pm$) to indicate tension or compression because the wind may blow in either direction against the sides of the building. The gravity forces, because they are produced by loads which act in only one direction (downward), are either positive ($+$) or negative ($-$), never both. Pages 2-8 and 2-9 of the *Manual* define the several kinds of loads and their combinations to be applied in designing truss joints.

One of the advantages of listing the forces as in Figure A7-66 is that the design drawing indicates whether any of the double-angle truss members may be subject to both tension and compression. If the magnitude of the reversible force is such that a dead load tensile force is less than the compressive wind force, the spacing of the stitch fasteners or welds connecting the two angles would be governed by the more restrictive requirements for tension or compression members (AISC *Specification* Chapter D or E).

Design drawings of trusses should show all dimensions that are required to establish the necessary working points and distances between working points.

The columns in Figure A7-66 have been proportioned by the designer to resist bending (acting in conjunction with the roof truss) from the moderate amount of wind load against the wall siding. The column bases are assumed free to rotate unless otherwise specified by the designer. Therefore, the required column details are relatively simple.

Figure A7-52 is a design drawing of an industrial building that must support an overhead traveling crane having a lifting capacity of 15 tons. In this building, the columns are subject to large bending forces because, in addition to the bending moments induced by wind, the operations of the crane will impose horizontal forces at the crane girder level, which must be resisted by the column in bending.

In designing this structure, the engineer had to give special attention to the problem of developing suitable connections for the stepped columns, where the upper shaft is spliced to the lower shaft and where the lower shaft is fastened to the foundation. These connections form a very important part of the structure.

As required by the AISC *Code of Standard Practice*, the designer has indicated the desired make-up of these connections. The steel detailer will follow the design drawing in detailing these connections or, in special cases, obtain approval from the designer before varying any details.

ENGINEERING DESIGN DATA

The information needed for detailing columns, as well as other structural members, is normally found on the structural design drawings. These drawings show the size and location of all parts of the structural frame using plan views, elevations, sectional views, enlarged details, tabulations and notes. They should include all information necessary for complete detailing.

Plan views show the locations of column centers and indicate the orientation of column faces. Beams and girders shown on column centers are assumed to connect at the center of the column web or flange. Because the structural design drawings generally are small-scale line diagrams, enlarged sections are sometimes employed to locate off-center beams and to clarify special framing conditions. This is true, particularly, for perimeter (spandrel) framing, beams around stairwells and ramps, and members at elevator openings. Enlarged parts of the design drawings, such as those adjacent to corner columns, may be used to indicate the designer's solution or to alert the steel detailer to complex situations.

Beam connections to columns may be designed to resist wind or seismic forces in addition to vertical floor loads. Such special connections are sometimes sketched and tabulated on the design drawings and keyed to the beams by numbers and symbols. Ordinary framed or seated connections are usually designated by note or specification reference, as are the bolts or welds to be used. When vertical bracing, trusses or built-up girders are required, the necessary views are shown in vertical sections or exterior elevations.

TYPES OF COLUMNS

The most frequently used columns consist of 10-in., 12-in. and 14-in. W shapes. Even though design conditions sometimes require sections built up of several components, designers utilize W shapes, as rolled, whenever practical. In Figure 2-3, W-shape columns, cover-plated W-shape columns, and several types of built-up columns are shown. Special I- and H-shaped columns and box sections, sometimes with interior webs, can be made by welding plates together. Double- or triple-shaft columns, laced, battened or connected with diaphragms, may be used in mill buildings where crane runways and roof supports are combined in one member. Tubular columns of round, square or rectangular shape are used in light structures and, for architectural reasons, often supplant W sections in schools and small commercial buildings.

COLUMN SCHEDULES

To furnish the fabricator information on the size and length of columns required in a tier building, the designer prepares a column schedule, similar to the one shown in Figure 7-1f. Columns are identified and oriented on the design drawings by an appropriate symbol, usually the column shape in cross section, and are located by a system of numbering. Their location may be established using either a simple numerical sequence, as 1, 2, 3, etc., or a two-way grid system, with column centerlines assigned letters in one direction and numbers in the other direction. Thus, a column at the intersection of D and 4 would be column D4. The column schedule sometimes contains member loads, which should be included when required for the selection of column splice connections.

The required size and makeup of a particular column, including (usually) loading, is given in the column schedule. As the total load supported by a column increases through an accumulation of loads from each level of framing, the size of the column usually increases from roof to footing. The schedule shows the column sizes and specifies the elevation at which the sizes must change. For reasons of economy in fabrication and handling, splices usually occur at every second (or sometimes every fourth) level. Thus, each individual column length supports two (or four) floors, termed a tier. Horizontal reference lines in the column schedule represent finished floor lines or some other reference plane. Elevations of floor framing, as well as column splices, are referred by note or dimension to these lines. Bottoms of columns (or tops of base plates) and the "cut-off points" at the column tops are located similarly. Conditions do exist when it is proper to provide a column splice after the first level, and the erection logic of a project should be considered when choosing the column splice locations.

The size and length of columns in low buildings of one or two stories, where the same section may be used from top to bottom, are usually shown on the plans and in elevations or typical sections.

Locations of column splices can affect the cost of a high-rise structure. The following situations are cited for consideration:

- Because the lower tier is normally heavier, the column splice level is kept as low as possible in order to reduce weight of materials.
- Splices must be made at least 4 ft above finished floor level on perimeter columns, as required by OSHA, 1926 Subpart R, to permit the installation of safety cables. More specific information about OSHA requirements is outlined later in this chapter.
- The elevation of the splice must provide sufficient space to allow for the splice plate and beam connection to be made without interfering with each other. If the structure is braced, sufficient space for the bracing connection should be provided. It is a very undesirable situation for the column splice to share fasteners with or be dependent upon some other connection.
- The splice elevation should accommodate the erector who will make the connection. To splice a column at the mid-height or point of contraflexure (a change in the direction of bending in any member) may appear desirable, but, as this is several feet above the steel framing, such a splice can require additional expense in initially connecting the next higher tier, installing and tightening permanent bolts, or in field welding the splice, because scaffolding can be required for access.

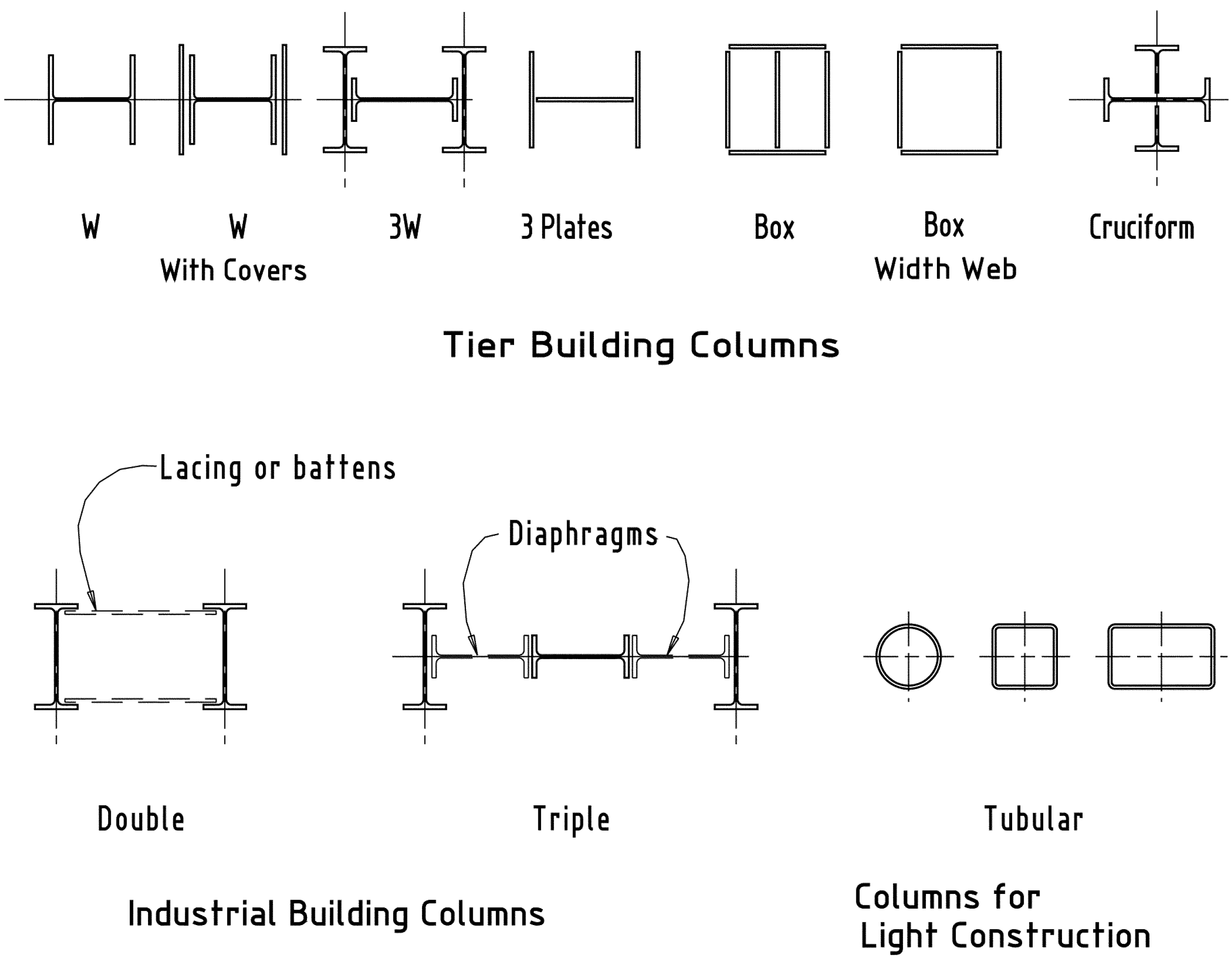


Figure 2-3. Typical building column sections.

This is troublesome, particularly during erection of the next tier, and is sometimes an unsafe procedure.

DISTRIBUTION OF PLANS AND SPECIFICATIONS

Immediately upon receiving notice to proceed with structural steel fabrication, the fabricator obtains from the general contractor either several sets of prints of the design drawings (architectural and/or engineering) or a set of reproducibles or electronic files, which the fabricator uses to make the required number of sets of prints. These design drawings are usually marked "Released for Construction" or with a similar note to differentiate them from the design drawings used when the estimate was made and from which the project was bid. As stated in the AISC *Code of Standard Practice*, this note permits the fabricator to commence work under the contract, including placing orders for material and preparing shop and erection drawings, except where the design drawings designate hold areas to be avoided due to a design that is incomplete or subject to revision. One set of design drawings and specifications is given to the estimator to compare with the design drawings used during the bidding. If differences between the bid and contract sets are detected, the estimator determines the cost and schedule impact and advises the sales manager. The sales manager must decide if the differences are acceptable without adversely affecting job costs and schedules or if they require contractual changes. If the latter is the case, the cost and schedule changes to which the fabricator and general contractor agree can be included in the contract documents before they are signed by both parties.

Another set of design drawings and specifications is issued to the fabricator's production manager, usually with a copy of the summary of the estimate. With these documents the production manager can see what kinds of pieces will be fabricated (beams, columns, trusses, etc.), their weights and their sizes. If the production manager recognizes that some shipping pieces must be limited in size or weight or that two or more relatively small shipping pieces can be combined into a larger one, the matter is discussed during an in-plant pre-production meeting. This recognition on the part of the production manager comes from experience, familiarity with shop equipment and capacity, familiarity with transportation regulations, and from knowledge of the erector's capabilities.

If the fabricator elected in the bid to furnish such items as miscellaneous steel, decking, joists, etc., sets of design drawings and related job specifications are distributed to each of these fabricator's subcontractors along with a purchase order from the fabricator. The purchase order to each subcontractor describes the items to be supplied by that subcontractor. Sometimes the subcontractor will require information in the form of a drawing or a list prepared by the fabricator's structural steel detailer.

Depending upon the size of the project and the number of steel detailers assigned to it, sufficient quantities of design drawings and related specifications are issued to the structural steel detailing group. The detailing manager studies the design drawings and specifications and schedules the work to be done to meet the fabricator's schedule for the project. At the in-plant pre-production meeting, the detailing manager has the opportunity to discuss and resolve with the sales and/or production managers any questions or concerns prior to beginning detailing functions. The detailing manager's accumulated experience and knowledge of steel fabrication and erection leads to valuable suggestions to the sales and production managers of ways to expedite fabrication and erection.

STEEL DETAILING GROUP

The production of fabricated steel starts with the steel detailing group, which follows an established procedure to ensure an orderly flow of work through the shop. The organization of the group resembles that shown in Figure 2-1. It could be a group of steel detailers that forms either a department in-house with the fabricator or a separate company under contract to the fabricator. A tremendous amount of paperwork is involved. Drawings and bills (standard forms) prepared by the steel detailer form an important part of this paperwork. Therefore, each steel detailer must understand thoroughly the system used by the employing fabricator.

The constantly increasing use of data processing equipment causes revision of the various forms used by individual companies. Understanding the purpose of each form, the steel detailer will have little difficulty in adapting quickly to the use of the particular forms used by the fabricator.

To assist the steel detailer in understanding the functions of a detailing group, a list of the various operations in their approximate sequence follows. Figure 2-4 is a flow chart illustrating the sequence of operations. Its purpose is to give the steel detailer an idea of the relationships of the several functions listed. Of course, the relationships may change depending upon the type of project, its size, the schedule, the size of the detailing group, and other factors. A description of the work required for each operation is given in later chapters.

A typical detailing group would perform its procedures in approximately the following sequence:

- Initiate job and fabricator setup (i.e., pre-planned checklists).
- Prepare typical details, job standard sheets, layouts, and calculation sheets.
- Prepare system of assembling and shipping piece marks.
- Prepare and check advance bills for ordering material.
- Make and check anchor rod/embedment drawings.
- Make and check erection drawings.
- Make and check detail shop drawings, including bills of material.
- Secure approval of shop drawings.

- Incorporate approval comments.
- Issue shop drawings to the shop.
- Prepare lists of field fasteners.
- Fit check (discussed in Chapter 8).
- Issue shop and erection drawings to field.

Detailing groups involved with 3D modeling detailing may use the following list of procedures:

- Initiate job and fabricator setup (i.e., pre-planned check-lists).
- Prepare typical details, job standard sheets, layouts, and calculation sheets.
- 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.

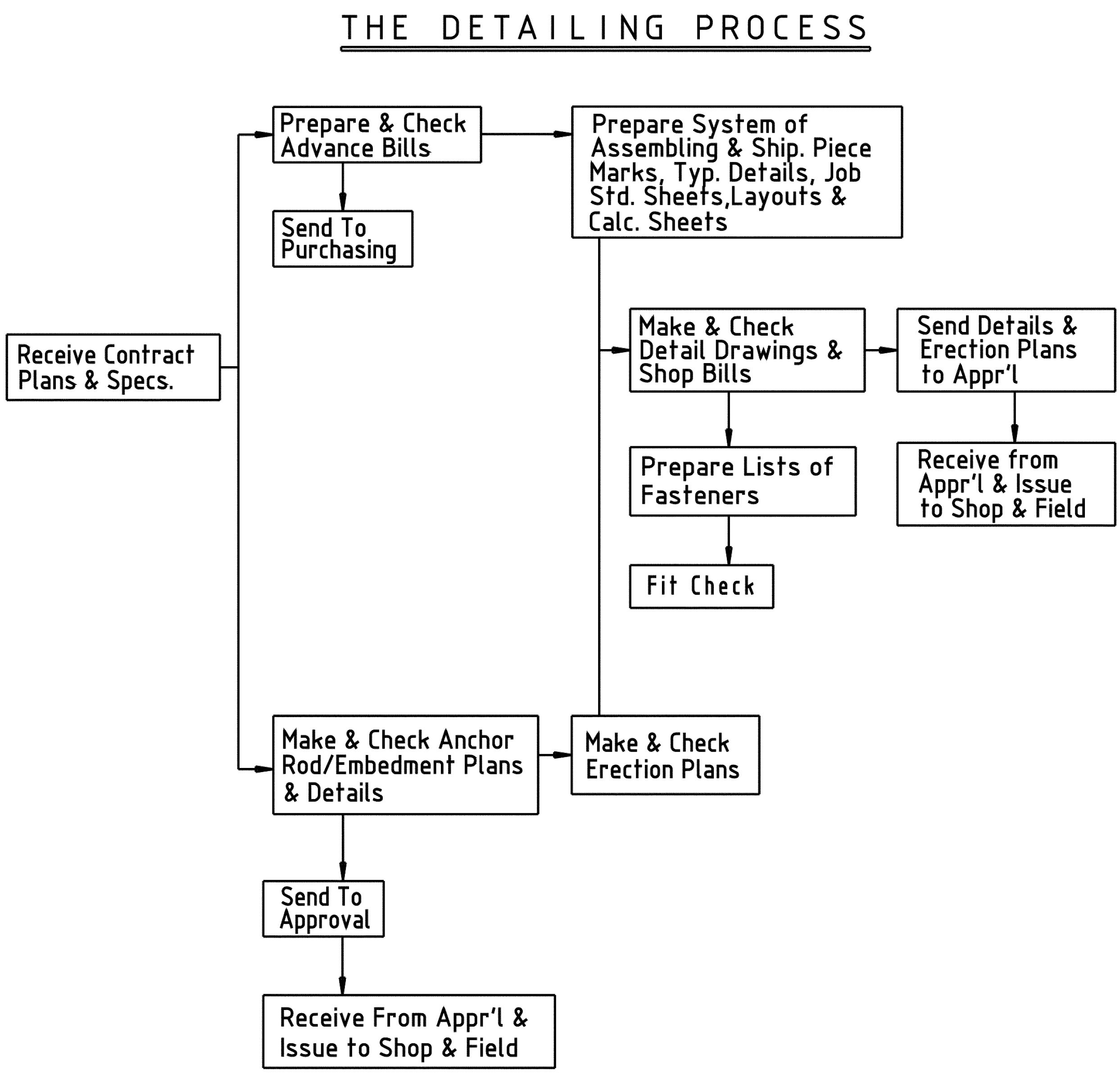


Figure 2-4. Detailing process sequence of operations diagram.

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.

CONTRACT DOCUMENT ERRORS

As indicated earlier, 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 AISC *Code of Standard Practice* 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 (e.g., see Figure 2-5), 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 Design Drawings so that the quantity, detailing and fabrication requirements for these items can be readily understood." Columns should be designed to eliminate web doubler plates and web stiffeners, when possible.

This text describes only a few of the errors in contract documents encountered by steel detailers in the normal pursuit of their work. The steel detailer should bring any errors discovered to the attention of the design team and be willing to become involved in the resolution of those falling within his or her field of experience. Often a steel detailer's suggested correction of a discrepancy in the contract documents will be helpful to and accepted by the design team.

DETAILING QUALITY

Whether shop drawings are made by hand or with computer-aided drafting (CAD), they must be accurate and complete

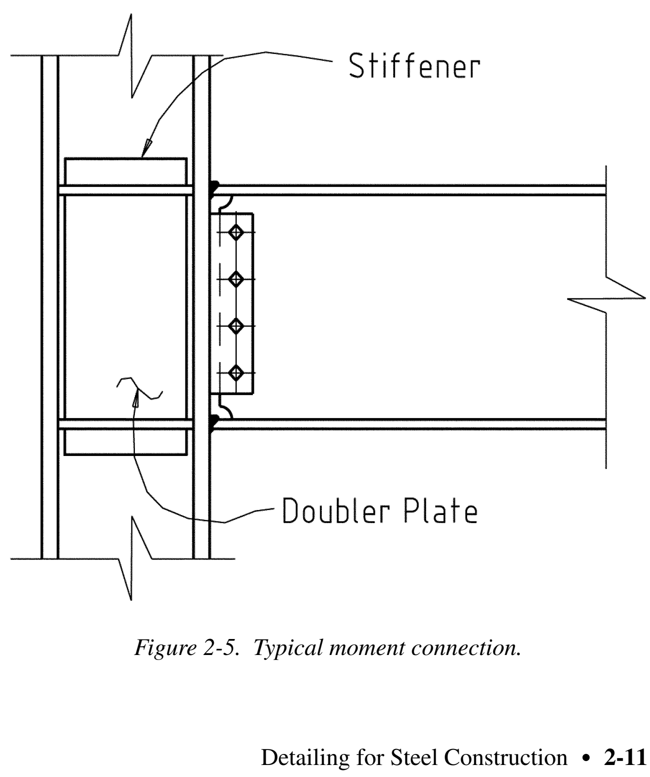


Figure 2-5. Typical moment connection.

and easily readable in the shop environment. Additionally, the steel detailer must remember that the shop drawings are used not only by the fabricating shop, but also by other subcontractors such as plumbers, HVAC contractors, fire-protection applicators, and others.

Drawings must be neat and never appear cluttered. In preparing a shop drawing, the number of views needed is determined by the amount and kind of fabrication required and the attached detail material. The spacing of the views must allow adequate dimensioning and the addition of any notes that may be required. More information covering the preparation of shop drawings will be found in later chapters.

SPECIFICATION AND CODE REQUIREMENTS

The AISC *Specification* covers design, fabrication and erection of structural steel for buildings. The steel detailer is encouraged to review the headings of the many sections of the AISC *Specification* to become familiar with its coverage. Much of the AISC *Specification* is concerned with design criteria, with which the steel detailer will have little, if any, need in performing the customary detailing functions. However, certain sections are of considerable interest to the steel detailer and should be given specific attention:

SECTION	TOPIC
A1	Scope
A3	Material
B3.6	Design of Connections
Chapter J	Design of Connections
J10	Flanges and Webs with Concentrated Forces
Chapter M	Fabrication, Erection and Quality Control
Commentary	Sections related to the foregoing

The AISC *Code of Standard Practice* is a compilation of the trade practices that have developed among those involved in the buying and selling of fabricated structural steel. It has been updated several times since its inception in 1924. As with the AISC *Specification*, the steel detailer is encouraged to review the entire AISC *Code of Standard Practice* to become familiar with the many areas it covers. However, of particular significance to the steel detailer are the following sections:

SECTION	TOPIC
1	General Provisions
2	Classification of Materials
3	Design Drawings and Specifications
4	Shop and Erection Drawings
10	Architecturally Exposed Structural Steel

OSHA SAFETY REGULATIONS FOR STEEL ERECTION

OSHA safety regulations for steel erection are found in 29 CFR 1926, Subpart R (OSHA, 2006), which is a series of articles starting with 1926.750. As much as possible, the relevant article will be referenced, but in the text that follows, 1926 will be omitted, as it is repetitive. This discussion is not intended to list every aspect of the OSHA regulations, as they are far too numerous and detailed. Instead, the discussion will emphasize those aspects of the OSHA regulations that are of particular interest to steel detailers, regarding the fabrication of structural steel. The full text of the safety regulations is available for download from OSHA's website at www.osha.gov. Additional information is given by Barger and West (2001).

Scope of the Standard [.750]

The scope is extremely broad and encompasses virtually all activities of steel erection. It applies to new construction and the alteration or repair of structures where steel erection occurs. Interestingly, other structural materials, such as plastics and composites, are included when they resemble structural steel in their usage.

Definitions [.751]

The following definitions are of particular interest:

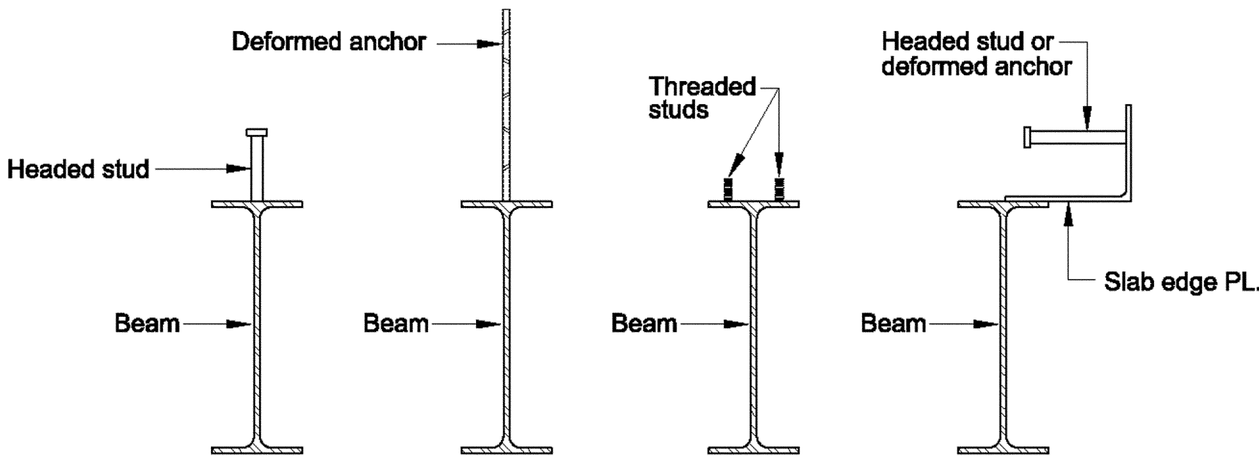
- Column
- Constructability
- Double Connection
- Double Connection Seat
- Final Interior Perimeter
- Opening (in a decked area)
- Post (as opposed to a column)
- Project Structural Engineer of Record
- Shear Connector
- Systems-Engineered Metal Building

Tripping Hazards [.754(c)(1)]

The shop placement of shear connectors, weldable reinforcing bars, deformed anchors, or threaded studs is prohibited where they would obstruct the walking surfaces of beams or joists (Figure 2-6). The shop placement of threaded studs on column cap plates to receive strut joists, deformed bars on column webs, or shear studs on beam or column webs is not prohibited since these are not walking/working surfaces (Figure 2-7).

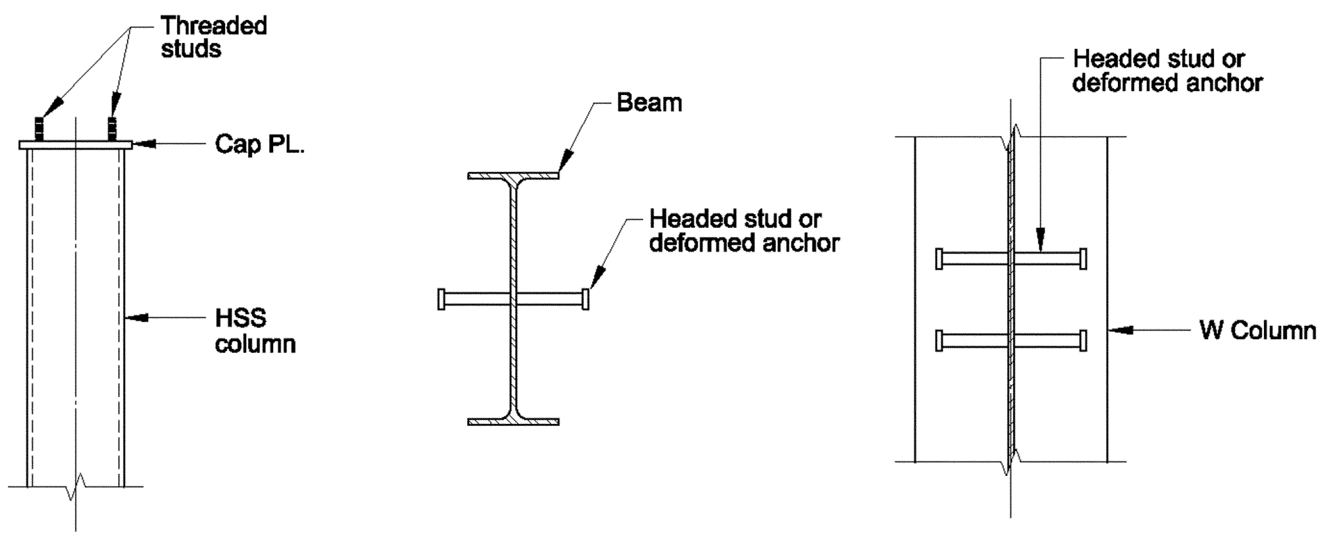
Roof and Floor Holes and Openings [.754(e)(2)]

Where design constraints and constructability allow, the structural supports for deck openings are to be fabricated so that decking runs continuously over the openings (Figure 2-8). This does not apply to major openings such as elevator shafts or stairwells. Other deck openings are not to be cut until the opening is needed.



Note: Headed stud, deformed anchor or threaded studs may not be shop attached because they obstruct the walking/working surface.

Figure 2-6. Examples of prohibited connection element placement that obstructs the walking surface.



Note: Examples of the shop attachment of headed studs, deformed anchors or threated studs that do not obstruct the walking/working surface and may be shop attached.

Figure 2-7. Permissible details.

Column Anchor Rods [.755]

Columns are required to have a minimum of four anchor rods [.755(a)(1)] (Figure 2-9) and those anchor rods as well as the column foundation are to be capable of supporting a 300-lb load (the weight of an erector and his tools) at the column top located at both 18 in. from the face of the column flange and from a plane at the tips of the column flange (Figure 2-10) [.755(a)(2)]. Posts (see OSHA definition) are not required to have four anchor rods (Figure 2-11).

The structural engineer of record must design a column's base plate and supporting foundation to accept the four anchor rods. The clear distance between column flanges (Figure 2-12) may not allow for a significant spread between anchor rods when placed inside the flanges of W8 and W10 columns. It is recommended that they be placed outside the column at the base plate corners. The designer may give consideration to the fact that base plates frequently require slotting in the field to accommodate misplaced anchor rods.

In the erection of all columns, the erector must evaluate the jobsite erection conditions and factors such as wind, when the column will be tied in, etc., and determine the necessity for guying or bracing [.754(d)(1) and .755(a)(4)]. This is consistent with the requirements of the AISC *Code of Standard Practice*, Sections 1.8 and 7.10.

Minimum Erection Bolts [.756(a) and (b)]

The requirements given in regulation are the minimum number of bolts to be used during erection to support a member until the crane's load line is released. Two bolts in each connection are the minimum to connect solid web members, and

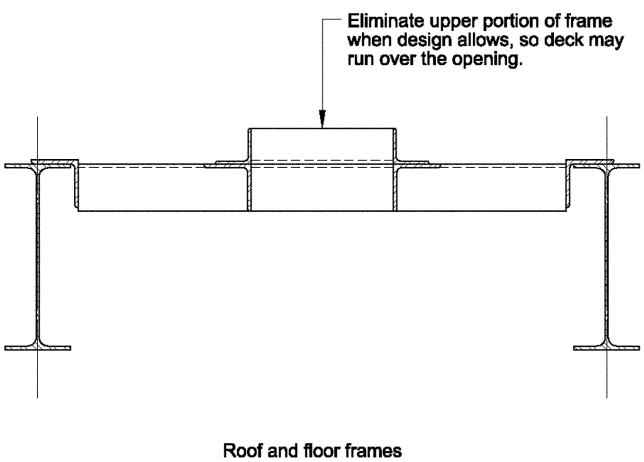


Figure 2-8. Continuous structural supports for deck openings.

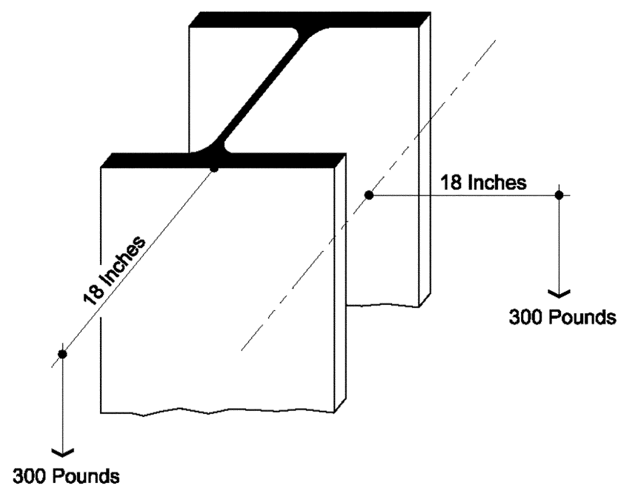


Figure 2-10. Loads required at column tops for erection.

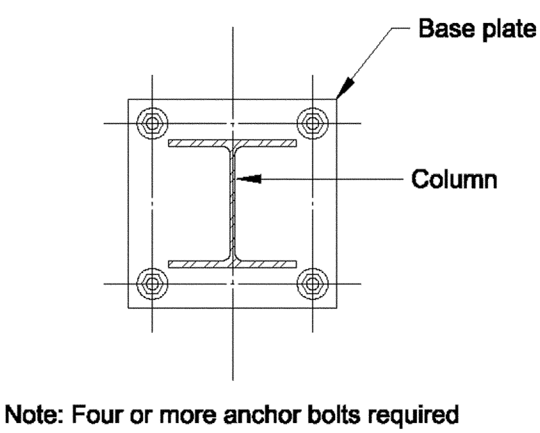


Figure 2-9. Minimum anchor rod requirement.

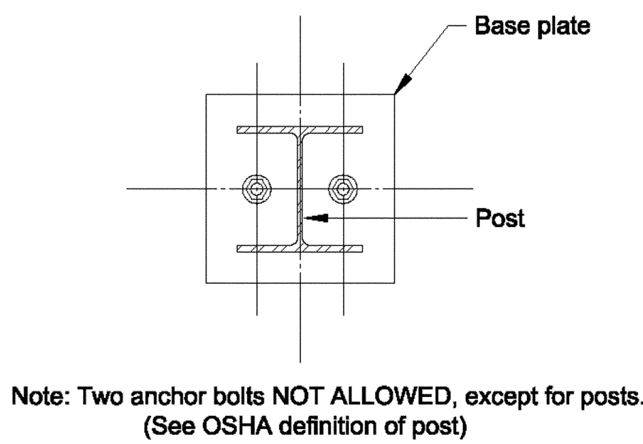


Figure 2-11. Special anchor bolt requirements for posts.

one bolt is the minimum for solid web bracing members or the equivalent as specified by the project structural engineer of record. The initial minimum bolts are to be the same size and strength as shown in the erection drawings. The erector is required to maintain structural stability at all times during the erection process [.754(a)], and the determination of the number of bolts required to temporarily support members is a responsibility of the erector.

Double Connections [.756(c)]

Only double connections of beams to column webs or to the webs of girders over columns in the case of cantilevered construction are regulated—not such connections at locations away from the columns. This boxes the bay with strut beams. The rule is based on the fact that an erector commonly sits on the beam on the first side of the double connection while the beam on the opposite side is connected in these regulated instances. If the connection gets away from the erector, beam and column collapse can occur and the erector may fall. Typical beam-to-beam double connections (other than at a cantilever over a column) require no special consideration since the erector can instead sit on the girder that receives both beams. At column conditions, there are many ways to facilitate safe double connections (Figures 2-13 through 2-17). The staggering of end angles on each side of the column web (single staggered), as shown in Figure 2-13, may not stabilize the beam's top flange unless metal deck is present and the angles may be better staggered on each side of the beam web (double staggered), as shown in Figure 2-14. When seats (Figures 2-15 through 2-17) are used, the beam must have a positive connection to the seat, while the second member is erected. The figure in the OSHA Standard's Appendix H shows clipped plates where end plates are used as shear connections.

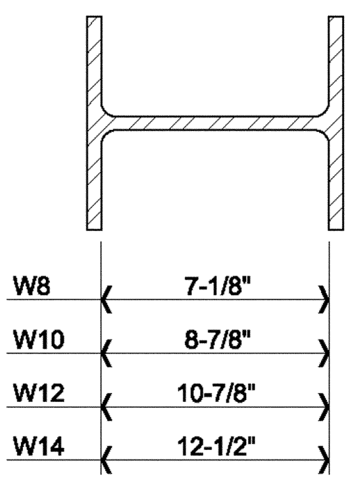


Figure 2-12. Approximate clear distance between column flanges.

Column Splice Strength [.756(d)]

Column splices have the same 300-lb loading requirement at the top of the upper shaft as required for anchor bolts (rods) (Figure 2-10). Again, the erector must consider other factors, such as wind, and guy the column accordingly, if necessary.

Column Splice Locations [Appendix F]

Since connectors are required to tie off when the fall distance exceeds 30 ft, placing column splices every three floors is an inefficient choice for the purposes of erection. The erector will erect two floors, deck the second level, and then erect and deck the third level before starting the process again. It would be better for the project structural engineer of record to place column splices either every two floors or, in some cases, every four floors so as to optimize the erection process.

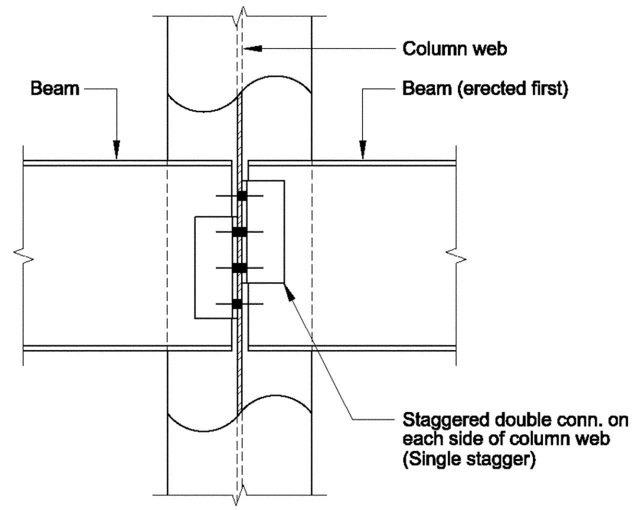


Figure 2-13. Single-staggered stabilization of beam's top flange.

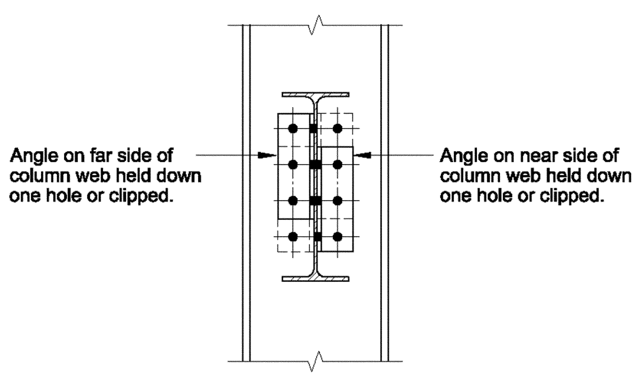


Figure 2-14. Double connection staggered on each side of each beam web.

Column Splice Height at Perimeter Columns/Perimeter Safety Cable Attachments [.756(e)]

Except where constructability does not permit, perimeter columns must extend a minimum of 48 in. above the finished floor to allow the attachment of safety cables. Per [.760(a)(2)], perimeter safety cables are required at the final interior (see definition) and exterior perimeters for the purpose of protecting the erector from falls from decked areas. The columns must be provided to the erector with either holes or attachments to support the top and middle lines of the safety cables at 42 in. and 21 in. above the finished floor. This is not required at openings such as stairwells, elevator shafts, etc.

It is best left to the fabricator to determine the most economical way to support the safety cables. Perimeter safety cables must meet the requirements for guardrail systems in 1926.502 (Appendix G) [.760(d)(3)].

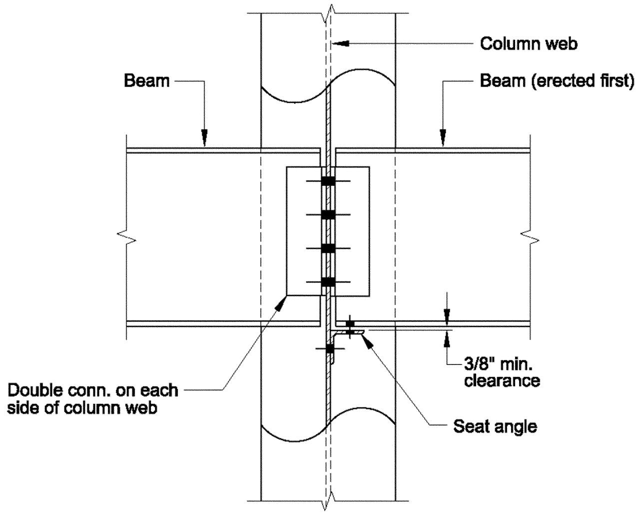


Figure 2-15. Double connection with temporary bolted erection seat.

Joist Stabilizer Plates at Columns [.757(a)(1)]

When the columns are strutted with joists, the column must be provided with a plate to receive and stabilize the joist bottom chord. The plate must be a minimum of 6 in. by 6 in. and extended 3 in. below the joist bottom chord with a $^{13}/_{16}$-in. diameter hole for attaching guying or plumbing cables (Figures 2-18 and 2-19). Figures 2-18 and 2-19 show details at column tops in cantilevered girder construction. Figure 2-18 shows stiffeners in the beam web above the column. In this case, the stiffeners acting with a properly designed column cap will provide the necessary continuity and stability for the column top. Thus, the joist bottom chord extensions need not be welded to the stabilizer plates. In Figure 2-19 there is no stiffener over the column, and stability of the column top is provided by welding the extended bottom chords to the stabilizer plates. These welded connections create continuity in the joists. The resulting moments must be reported to the joist supplier so that the joists can be properly sized. The timing of the welding must be indicated so that it is consistent with the continuity moments reported. For example, the effects of loads applied prior to welding need not be included in the continuity moments.

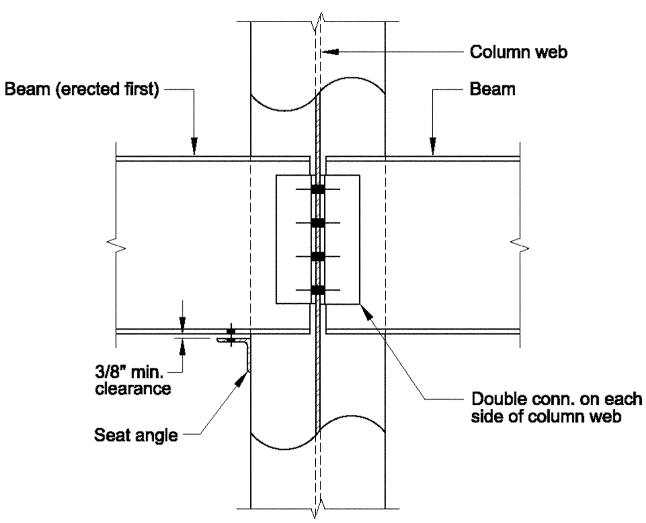


Figure 2-16. Double connection with shop-welded erection seat.

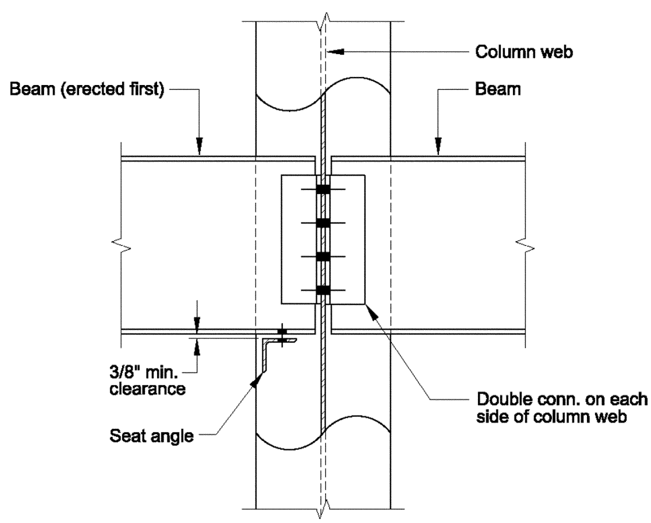


Figure 2-17. Double connection with shop-welded erection seat (alternative location).

Joists [.757]

Regulations regarding joists of interest to the structural steel detailer are:

- Strut joists at or near columns must be field-bolted [.757(a)(1) and (2)].
- Unless panelized, joists of 40 ft or greater span must be field-bolted to their supports unless constructability does not allow [.757(a)(8)].

Steel detailers must take the bolting requirements for joists of 40-ft spans and over into consideration in beam details, particularly in cantilevered construction over the cantilever support. Note that strut joists require bolting and stabilizer plates regardless of span. K-series joists commonly use ½-in.-diameter bolts, while LH-series and DLH-series joists use ¾-in.-diameter bolts. Fabricators must not arbitrarily increase bolt diameters without verifying with the project structural engineer of record that the additional loss of net cross-sectional area from the beam flange will not affect the supporting member's design. Threaded studs may not be used on walking/working surfaces because they constitute a tripping hazard [.754(c)(1)].

Systems-Engineered Metal Buildings [.758]

All requirements of Subpart R apply to systems-engineered metal buildings (see definition) except as noted in that section. Additionally, there are some safety requirements that are unique to this type of construction.

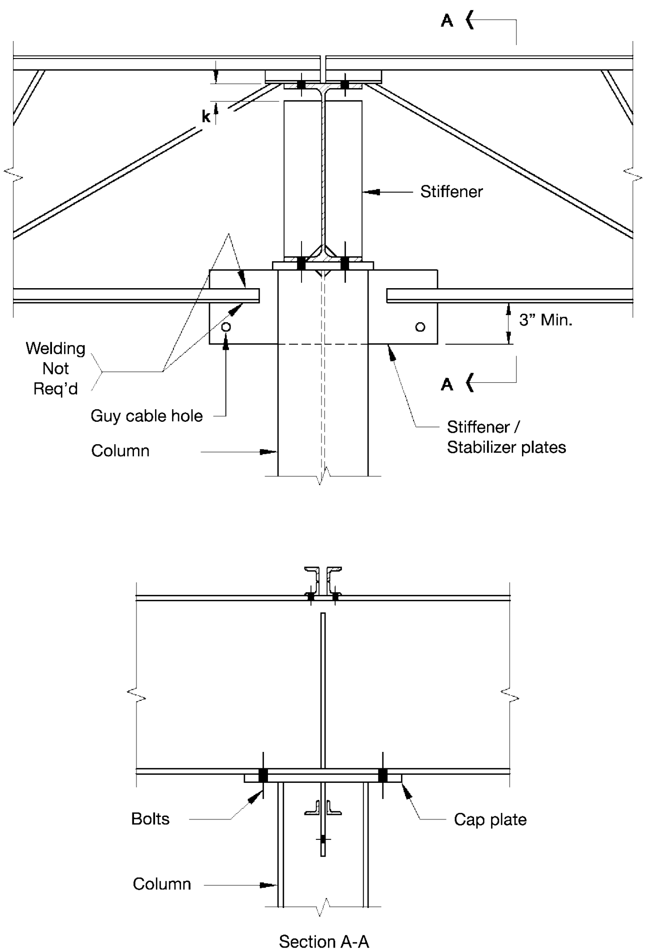


Figure 2-18. Strut joist with stiffener and stabilizer plate.

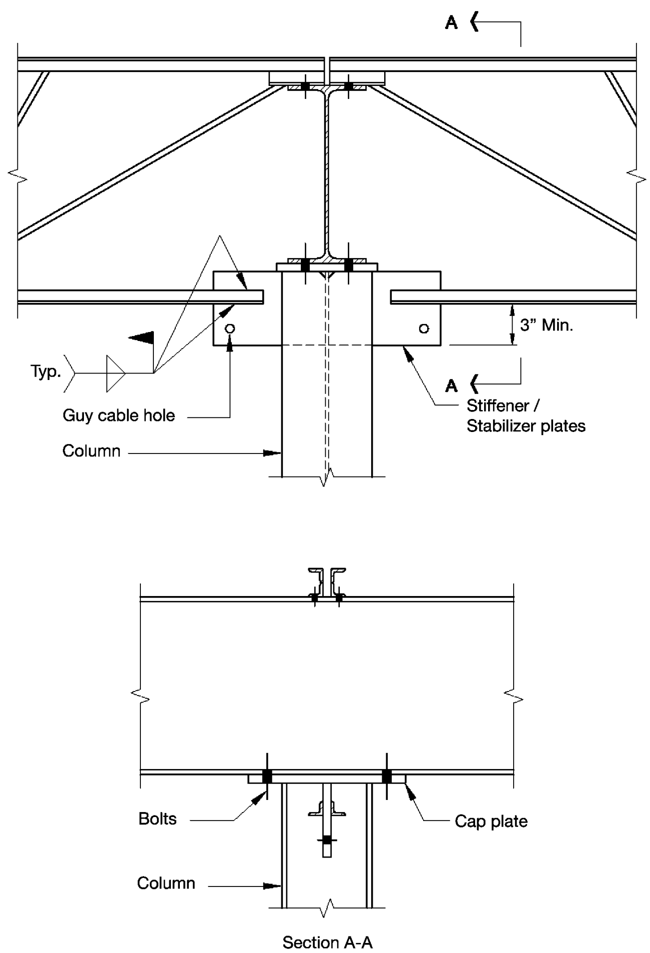


Figure 2-19. Strut joist with welded bottom chord and stabilizer plate.

CHAPTER 3
COMMON CONNECTION DETAILS

Various framing configurations and the force transfer mechanisms that occur in each, with illustrations of the common connection details that are used in each case.

An external force applied to a structural member is called a load. Loads are most commonly expressed numerically in kips, where a kip is 1,000 lb. When loads are applied to a member, internal forces develop within the member as it resists the loads. In a member resisting a load, the internal axial force divided by the area over which it acts is called a stress. Stresses are expressed numerically in kips per square inch (ksi).

The beams, girders, trusses, columns and other members that form a completed structural frame are designed to resist these loads (i.e., self-weight and superimposed loads). Each of these members must transmit its loads through its connections to supporting members. For example, beams transmit their loads to girders, girders transmit their loads to columns, columns transmit their loads to foundations, and foundations transmit their loads to the ground. Each member in this system must be provided with proper connections to transfer its loads to other members safely and economically, usually through a combination of bolts, welds and connecting elements, such as angles, plates or tees.

The AISC *Specification* includes both load and resistance factor design (LRFD) and allowable stress design (ASD) methods. In LRFD, factored loads and load combinations with separate factors for each load and for the resistance are used. In ASD, service loads and load combinations with a safety factor applied to the resistance are used. Because of these differences in load levels used, it is important to determine which method is required in the contract documents, and to indicate which method has been used in any connection design work. This text utilizes both load and resistance factor design and allowable stress design in all design examples. Refer to the *Manual* Part 2 for a more detailed discussion of the fundamentals of LRFD and ASD.

TYPES OF FASTENERS

ASTM A325 and A490 High-Strength Bolts

High-strength structural bolts are furnished in two strength grades: ASTM A325 high-strength carbon steel bolts and ASTM A490 quenched and tempered alloy steel bolts. ASTM A325 and A490 bolts are available from $^1/_2$-in. diameter to $1^1/_2$-in. diameter and in lengths generally up to 8 in. Each strength grade can be ordered as Type 1 (medium carbon steel for ASTM A325; alloy steel for ASTM A490) or Type 3 (atmospheric corrosion resistant steel for both ASTM A325 and A490). In the event a bolt order does not specify a bolt type, either Type 1 or Type 3 may be supplied at the manufacturer's option. Reference to Type 2 bolts was removed from the RCSC *Specification* following its removal from the ASTM A325 and ASTM A490 Specifications.

The RCSC *Specification* establishes the requirements for materials, design, installation and inspection of these fasteners.

ASTM F1852 Twist-Off-Type Tension-Control Bolts and Alternative Design Fasteners

The RCSC *Specification* also covers the use of ASTM F1852 twist-off-type tension-control bolts, which are the "TC" equivalent of ASTM A325 bolts. These bolts are installed with a special tool that twists off the splined end of the bolt when the proper bolt pretension is reached. RCSC *Specification* Section 2.8 also permits the use of other alternative design fasteners subject to the requirements in the RCSC *Specification* for alternative design fasteners. TC bolts equivalent to ASTM A490 bolts can be ordered under these provisions.

ASTM A307 Bolts

ASTM 307 bolts are sometimes called unfinished, machine or common bolts. Once common, these fasteners are rarely used today.

FORCES IN BOLTS

When resisting a load that tends to stretch it in the direction of its length, a bolt is said to be loaded in tension; the load creates a tensile stress in the material. Thus, the bolt labeled as (A) in Figure 3-1 is in tension. The available tensile strength of the bolts is listed in the *Manual*, Table 7-2.

More commonly, structural bolts are used to resist loads that are transverse to the length of the fastener. Shear on material is the algebraic sum of all the external forces acting on one side of a section plane through the material. For any section, the shear force on one side of the plane is numerically equal to the shear force on the other side, but opposite in direction. Figure 3-1 [see label (B)] illustrates the case of a fastener loaded in shear; the plane separating the plies of material is the section plane through the bolt. A shearing stress, determined by dividing the shear force by the area over which it acts, is created in the bolt at the section plane. Bolts have two design shear strengths, depending upon the location of the bolt threads with respect to the shear plane ("N" means threads pass through and are iNcluded in the shear plane; "X" means threads do not pass through and are eXcluded from the shear plane). The common practice is to use the lesser bolt available shear strength that includes the bolt threads in the shear plane, A325-N or A490-N. Many engineers prefer the use of "N bolts" to simplify bolt installation since there is no need to ensure that the threads are excluded from the shear plane. Bolts in connections designed with threads excluded from the shear plane are designated A325-X or A490-X. These bolts are either installed in the snug-tightened condition or pretensioned (see RCSC *Specification* Section 4).

In some cases, slip resistance must also be provided for shear connections, and the resulting connection is called slip-critical. In this connection the bolts are pretensioned and the faying surface(s) are prepared to achieve a defined slip coefficient, creating a clamping force between the connected parts that in turn creates a frictional resistance on the surfaces in contact. The RSSC *Specification* defines the faying surface as "the plane of contact between two plies of a joint." Thus, a double-angle, bolted/bolted connection would have two faying surfaces at the web of the supported member and one faying surface at the face of the supporting member. The need for slip-critical connections in building structures is normally quite limited, as indicated in RCSC *Specification* Sections 4 and 4.3. The only purpose of a slip-critical connection is to eliminate slip at design service loads. The bolt shear, bolt bearing or other such limit states may control the design of slip-critical connections and must be checked in addition to the slip resistance. Slip-critical joints are appreciably more expensive because of the associated costs of faying-surface preparation. When slip resistance is required and the steel is to be painted, the fabricator should be consulted to determine the most economical approach to providing the necessary slip-resistance. Special paint systems that are rated for slip-resistance can be specified. Alternatively, a normal paint system can be used with the faying surfaces masked. Note that the surfaces under the bolt head, washer and/or nut are not faying surfaces, see Figure C-3.1 in the RCSC *Specification*.

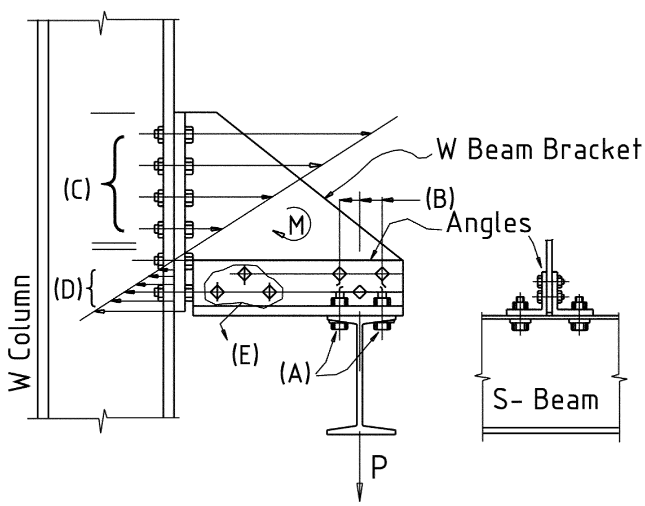


Figure 3-1. Basic functions of fasteners in a connection.

The same forces that cause shearing stress also attempt to push the bolt against the side of the hole and the resulting resistance is called a bearing stress. In Figure 3-1 [see (B)], the bearing loads applied against the opposite faces of the connecting bolts cause the shear stress in the bolts. When a fastener transmits a shear load in a bearing connection, a bearing stress is present in the connected material. In pretensioned and slip-critical shear connections, the fasteners also impose compressive stresses at the contact surface surrounding the bolts. Compressive stresses are caused by axial forces directed towards each other, tending to compress or shorten the material. These stresses induce the friction between the faying surfaces of the connected material.

Figure 3-1 illustrates the basic functions of fasteners in a connection:

An S-beam suspended from a bracket supports a load *P*, which is transmitted to the bracket angles by the bolts marked A. Bolts A resist the downward pull of *P*; each bolt supports a share of the load and is stretched in the direction of its length. These bolts are loaded in tension. The bolt force may be amplified by what is called "prying action."

The load from bolts (A) passes through the two bracket angles and is transferred by bolts (B) into the bracket web. These bolts prevent the angles from moving downward and, in doing so, resist a shearing force between the contact surfaces of the angles and the bracket web. Bolts (B) are loaded in shear.

The bolts attaching the bracket to the flange of the W column are divided into groups (C) and (D), in accordance with the loads they support. The entire group, (C) + (D), is affected by the downward force, *P*, and each bolt is loaded in shear. However, because of the position and direction of *P*, a rotating force or moment, *M*, is initiated, which tends to rotate the bracket in a clockwise direction, pulling the top away from the column and pushing the bottom toward it. The pull at the top of the bracket is resisted by the bolts in group (C). Bolts (C), therefore, are loaded in tension (in varying degrees) as well as in shear and are said to be loaded in combined shear and tension. Bolts (D) in the lower part of the connection, where the bracket presses against the column, are loaded in shear alone. The compressive load is transmitted through metal-to-metal bearing between bracket and column flanges and is not carried by the bolts. The diagonal line represents the assumed distribution or horizontal load intensity from top to bottom of the bracket.

Bolts (E) clamp the angles to the bracket web and, thereby, stiffen its bottom edge against buckling. When used in this way, they are called stitch bolts. Stitch bolts carry no readily calculable stresses. Their primary function is to hold together component parts to ensure action of the member as a unit, rather than an assemblage of loose pieces. Stitch bolts also are employed to seal the edges of contact surfaces against moisture, rusting and paint. The maximum spacing of fasteners for stitching is limited by AISC *Specification* Section J3.5.

The use of fasteners is not limited to transferring loads to and from structural connections. Fasteners also serve to transfer calculated forces between the main material elements comprising built-up members.

Nominal strengths are established by computations using specified material strengths and dimensions, and formulas derived from accepted principles of structural mechanics, or by field tests or laboratory tests of scaled models. In LRFD, design strengths are determined by multiplying nominal strengths by the resistance factor, ϕ. Thus, the design strength equals ϕR_n, where R_n is the nominal strength. In ASD, allowable strengths are determined by dividing the nominal strength by the safety factor, ϕ. Thus, the allowable strength equals R_n/Ω, where R_n is the nominal strength.

Available tension and shear strengths for bolts and available bearing strengths in base material are given in the AISC *Specification* Sections J3.6 and J3.10. The following discussion of strengths in fasteners is based on and limited to provisions in the AISC *Specification*. In detailing practice, the design specifications for a project (AISC and any other applicable specification) must be used.

The nomenclature most commonly used in computations involving fasteners employs the following symbols:

A_b = Nominal body area of a fastener (cross-sectional area based on nominal diameter), in.^{2}
F_{nt} = Nominal tensile stress, ksi
F_{nv} = Nominal shear stress, ksi
F_p = Nominal bearing stress of the type of steel being used, ksi
F_y = Specified minimum yield stress of the type of steel being used, ksi
F_u = Specified minimum tensile strength of the critical connected part, type of steel or fastener being used, ksi
f_t = Computed tensile stress, ksi
f_v = Computed shear stress , ksi
ϕ = Resistance factor for LRFD
Ω = Safety factor for ASD
r_n = Nominal strength of one fastener from the AISC *Specification*, kips/bolt
r_{ut} = Required tensile strength of one fastener, kips/bolt, LRFD
r_{uv} = Required shear strength of one fastener, kips/bolt, LRFD
r_{at} = Required tensile strength of one fastener, kips/bolt, ASD
r_{av} = Required shear strength of one fastener, kips/bolt, ASD
R_n = Nominal resistance or strength of joint, kips/bolt
ϕR_n = Design resistance or strength of joint, kips/bolt
R_n/ϕ = Allowable resistance or strength of joint, kips/bolt

Shear

If two plates are connected by a bolt as in Figure 3-2(a) and equal opposing forces, P, act on them, the tendency of these plates to slide past one another along their contact or faying surfaces is resisted by the bolt. The bolt is stressed on one transverse section, X-X, identified as a shear plane. The bolt is said to be loaded in single shear.

In Figure 3-2(c) the bolt shown is required to transmit the forces, P, from each of two outside plates into the middle plate. In this case the bolt is stressed on two shear planes, Y-Y and Z-Z. Each transverse section transmits a load $^1/_2P$ to equalize the force, P, in the middle plate. This condition of loading is called double shear.

Although single- and double-shear connections account for most bolted joints, multiple plate arrangements can involve triple and quadruple or more shears. For example, in Figures 3-2e and 3-2f the plates are placed alternately to utilize three and four shear planes, with proportionate increases in the total force one bolt is capable of transmitting. Thus, for a given load, additional plates lessened the force across each shear plane.

Every fastener has a specific available strength. If the applied load exceeds this strength, the fastener is undersized and it may "fail." For example, if the fasteners in Figures 3-2a and 3-2c did not have sufficient strength to resist the loads, P, the plates would shear the fastener shanks, as illustrated in Figures 3-2b and 3-2d.

The ability of a fastener to resist shear at any transverse plane is dependent on its nominal body area, A_b, its nominal shear strength, F_{nv}, and the resistance factor, ϕ. The product of these quantities, $\phi F_{nv} A_b$, is the design single-shear strength of the fastener. Twice this product, $2(\phi F_{nv} A_b)$ is the design double-shear strength; three times the product, the design triple-shear strength, etc. For convenience, these design shear strengths are labeled ϕR_n, which is the total shear load that one fastener is permitted to support in a specific condition of shear. These shear relationships can be expressed by the equations:

single shear

$$\phi r_n = \phi F_{nv} A_b \quad \text{(LRFD)}$$

and

$$r_n/\Omega = (F_{nv} A_b)/\Omega \quad \text{(ASD)}$$

double shear

$$\phi r_n = 2(\phi F_{nv} A_b) \quad \text{(LRFD)}$$

and

$$r_n/\Omega = 2(F_{nv} A_b/\Omega) \quad \text{(ASD)}$$

Numerical values of F_{nv} are established by the AISC *Specification* to provide an adequate margin of safety for each fastener type. Table J3.2 of the AISC *Specification* lists the nominal shear strengths for bolts. For connections where the

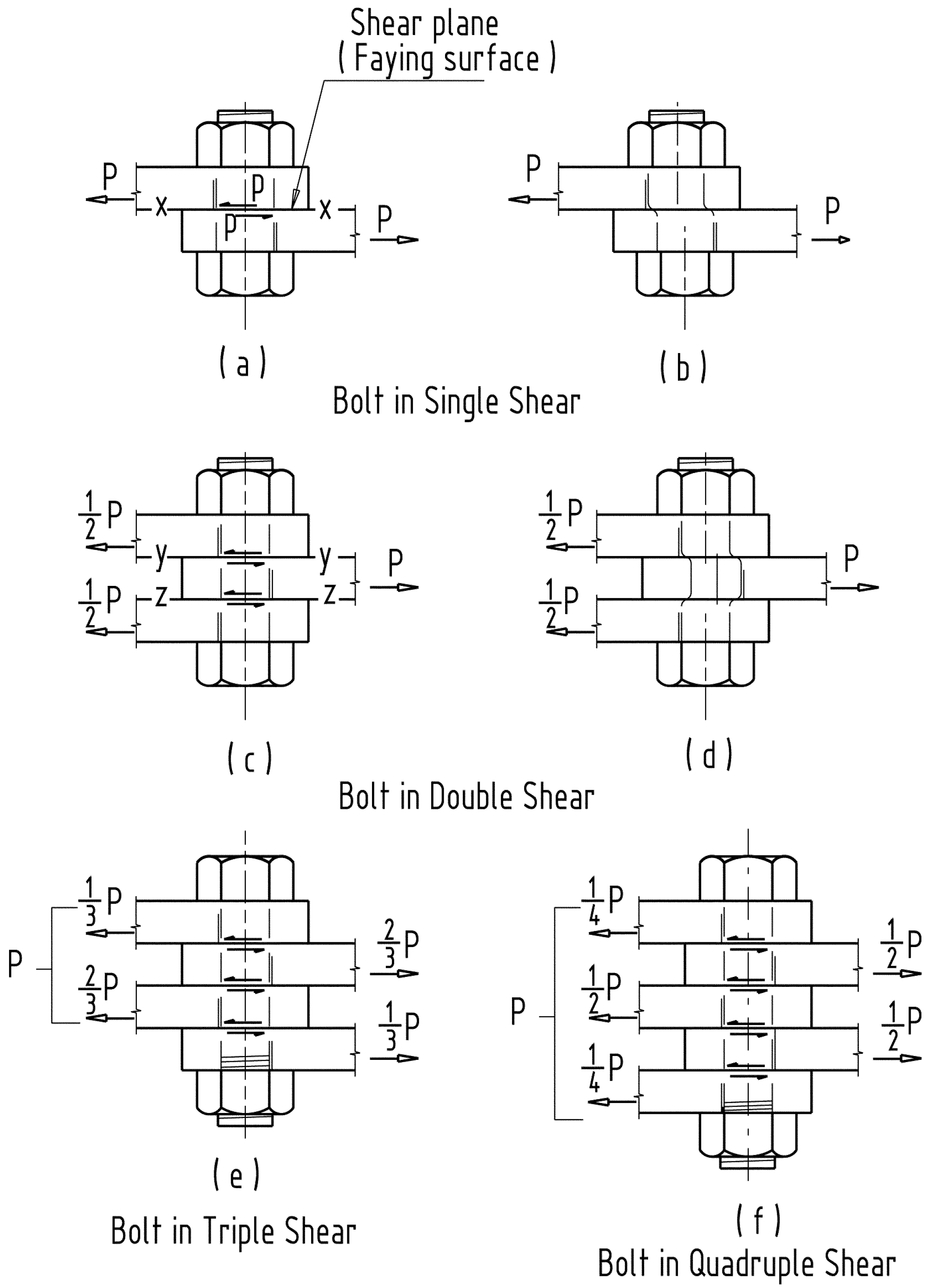


Figure 3-2. Bolt bearing.

bolt threads pass through the shear plane, F_{nv} includes an allowance for the reduction in bolt cross-section caused by threading.

Substituting a $^7/_8$-in. ASTM A325 bolt in a bearing connection with threads excluded from the shear plane and ϕ = 0.75, F_{nv} = 60 ksi, A_b = 0.6013 in.^{2} for LRFD and Ω = 2.00, F_{nv} = 60 ksi, A_b = 0.6013 in.^{2} for ASD in the preceding equations, provides an available single-shear strength of:

$$\phi r_n = 0.75(60)(0.6013) = 27.1 \text{ kips} \qquad \text{(LRFD)}$$

$$r_n/\Omega = [(60)(0.6013)]/2.00 = 18.0 \text{ kips} \qquad \text{(ASD)}$$

The same bolt has an available double-shear strength of:

$$\phi r_n = 2(0.75)(60)(0.6013) = 54.1 \text{ kips} \qquad \text{(LRFD)}$$

$$r_n/\Omega = 2\{[(60)(0.6013)]/2.00\} = 36.1 \text{ kips} \qquad \text{(ASD)}$$

Triple- and quadruple-shear strengths of this bolt would be three and four times the single-shear strength, respectively (Figures 3-2e and 3-2f).

In the *Manual* Part 7, Table 7-1 provides available single- and double-shear strengths for structural fasteners ranging in size from $^5/_8$- to $1^1/_2$-in. diameter. Thus, Table 7-1 eliminates the need to calculate ϕr_n and r_n/Ω. The table lists values of available shear strengths according to the identifying letter for the type of connection, i.e., N = bearing with threads included in the shear plane and X = bearing with threads excluded from the shear plane. This is usually written as A325-N or A325-X (or A490-N or A490-X). Bolts in slip-critical connections are noted A325-SC (or A490-SC). This coding method is for the user's convenience; the same actual bolt as manufactured is used in all these applications.

In most, if not all connections, more than one fastener is required. The designer must (1) determine how many fasteners of a certain type and size are needed to support an applied load or (2) analyze a connection to determine its strength. These computations are illustrated in the *Manual*.

Bearing in Bolted Shear Connections

Although the fasteners in a structural joint may be strong enough internally to transmit the applied forces, the joint may fail if the material joined is not capable of transmitting these forces into the fasteners. Figure 3-3 illustrates how a thin connecting plate in a shear bearing connection might fail by a bearing or a tension tear out failure before the full shear strength of the fastener can become effective.

The ability of a joint to resist this type of failure depends on the amount and bearing strength of the connected material. Figure 3-4a illustrates single-shear bearing. The opposing forces, P, act on the body of the fastener through the sides of the holes. The surfaces over which these forces are assumed to bear uniformly are the projected areas abcd and efgh, shown in the cutaway view, Figure 3-4b. These projected areas are calculated by multiplying the plate thickness by the nominal fastener diameter.

Figure 3-4c illustrates double-shear bearing. Principles and assumptions here are the same as for single-shear bearing. Note, however, that the amount of bearing pressure in the outer plates is only half of that in the center plate when all of the plates are of the same thickness. Bearing in triple- and quadruple-shear joints is similar; the individual-ply bearing stresses being proportional to the total forces applied to the individual plies of material.

The pressure that may be applied safely in bearing is not dependent on the stronger fastener material, but rather on the material being fastened. Available bearing strengths are established by AISC *Specification* Section J3.10 and are based on the specified minimum tensile strengths of the types of material connected. If a joint is made up of material having differing F_u values (i.e., A36 steel connected to A992 steel), the bearing strength for each type of material must be considered.

The *Manual* is of considerable assistance in solving problems involving shear and bearing relationships. Values of fastener available bearing strengths for steels of various F_u values, tabulated by fastener diameter, are given in Table 7-5. Thus, shear and bearing strengths may be compared quickly and the available strength can be determined without computation.

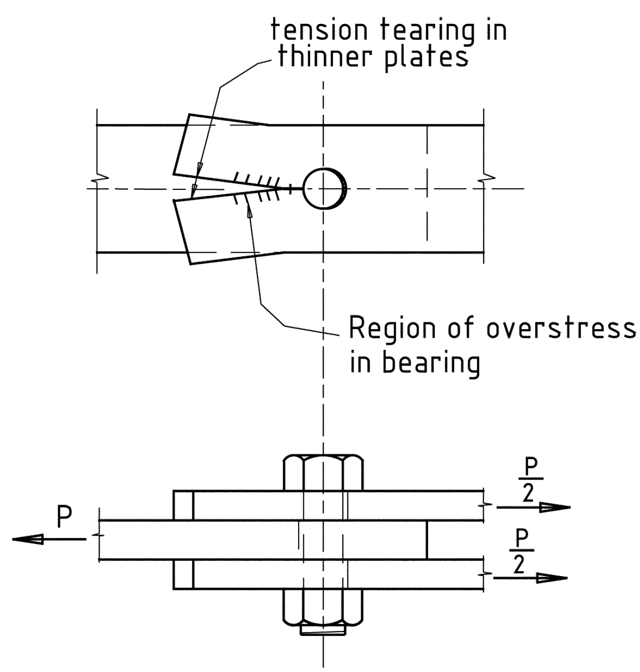


Figure 3-3. Tension tear-out failure.

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 AISC *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 J3.5 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

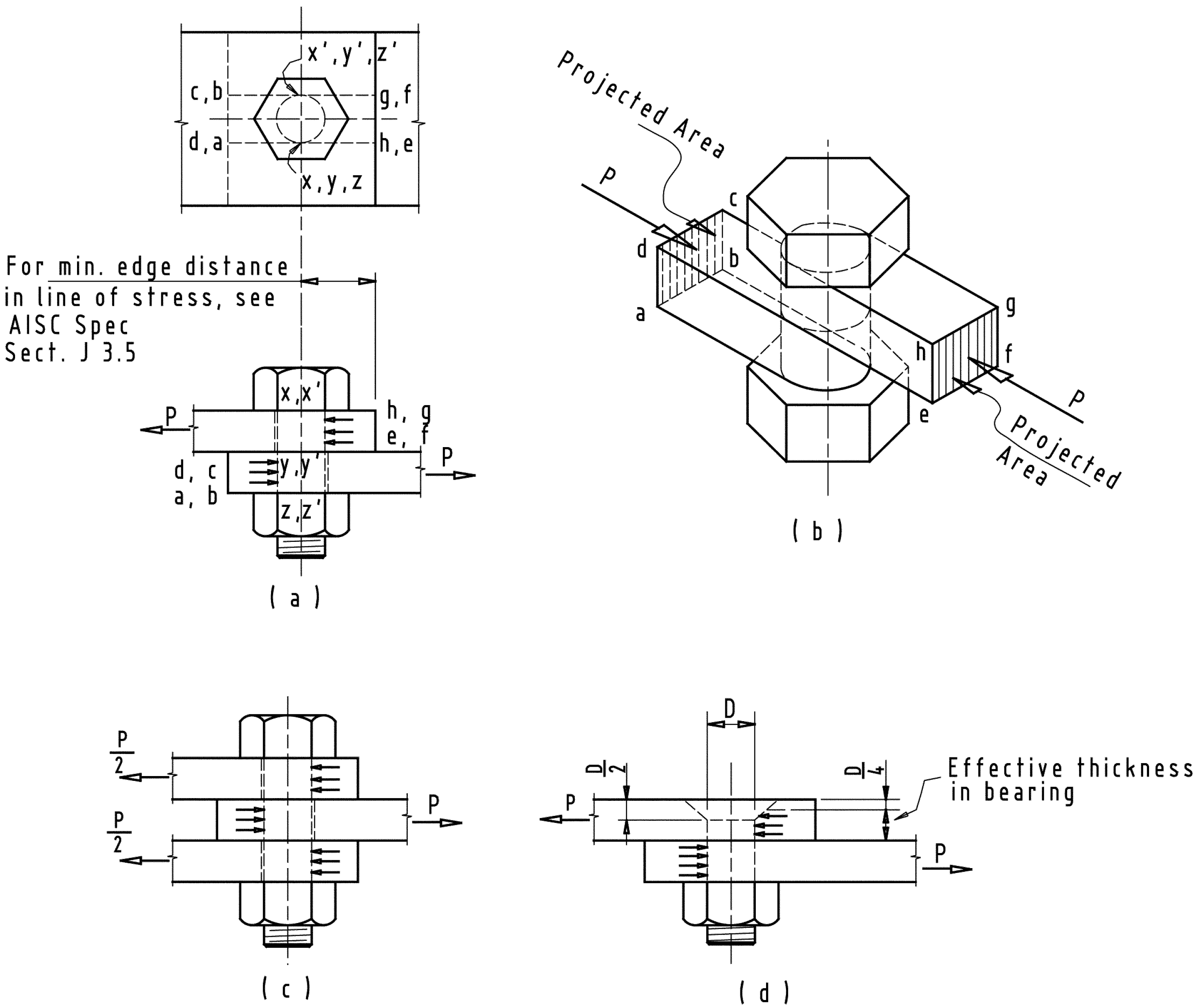


Figure 3-4. Bolt bearing.

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 strength, F_{nv}, 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_{nv} 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 ensuring 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 Table C-2.1 of the Commentary in 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.

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_{nv} 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 8.2).

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 4.3.

The frictional resistance required to transfer the shear loads depends on the high clamping forces exerted by pretensioned bolts and the condition of the surfaces in contact. Consequently, the RCSC *Specification* requires that all contact surfaces, including those under bolt heads, nuts and washers, shall be in solid contact, free of loose mill scale, dirt and other foreign material. Contact surfaces within the joint shall be free of any applied coating that would tend to lubricate the surfaces and so reduce their frictional resistance. Alternately, the joints may be specified to have painted faying surfaces, which are blast cleaned and coated with a special, qualified paint. Faying surfaces specified to be galvanized shall be hot-dipped galvanized and manually roughened using a hard wire brush.

The amount of frictional resistance caused by the clamping action of a high-strength bolt is related directly to the cross-sectional (nominal body) area of the bolt, the pretension of the bolt, and the nature of the faying surfaces and type of hole. Discussions on coated surfaces are provided in Chapter 4.

Section J3.8 of the AISC *Specification* specifies methods for determining the available slip resistance for three categories of hole conditions for slip-critical connections. The values are based on Class A (slip coefficient 0.35), clean mill scale, and blast-cleaned surfaces with Class A coatings. In the *Manual* Part 7, Table 7-3 lists the available shear strengths for bolts in sizes from $^5/_8$ to $1^1/_2$ in., when slip is a serviceability limit state. Similarly, Table 7-4 lists the available shear strengths when slip is a strength limit state.

As the fastener in a slip-critical connection could slip into bearing at ultimate load, the AISC *Specification* requires that the base material be checked for bearing strength as for a bearing member.

In slip-critical connections, fasteners shall be installed in properly aligned holes and tightened to the minimum required pretension as required in RCSC *Specification* Section 8.2.

Tension Joints

Nominal tensile strength in bolts is measured by the product of the gross (nominal) bolt area, A_b, and the nominal tensile strength per unit area, F_{nt}. Thus, the design tensile strength (LRFD), ϕr_n, of a bolt equals $\phi F_{nt} A_b$ and the allowable tensile strength (ASD), r_n/Ω, of a bolt equals $(F_{nt} A_b)/\Omega$. Nominal stress, F_{nt}, for these fasteners is listed in the AISC *Specification* Table J3.2.

Gross bolt areas, in square inches, are based on the gross bolt diameters of structural bolts, with no deduction for thread depth. Table 7-4 in the *Manual* Part 7 lists bolt areas for a wide range of diameters.

For example, determine the available tension strength, ϕr_n or r_n/Ω, of a $^7/_8$-in.-diameter ASTM A325 bolt:

Using Table J3.2 for values of ϕ and F_{nt} and *Manual* Table 7-2 for values of A_b:

ϕ = 0.75 and Ω = 2.00

F_{nt} = 90 ksi

A_b = 0.601 in.^{2}

$$\phi r_n = \phi F_{nt} A_b \quad \text{(LRFD)}$$

$$\phi r_n = 0.75(90)(0.601) = 40.6 \text{ kips} \quad \text{(LRFD)}$$

$$r_n/\Omega = (F_{nt} A_b)/\Omega \quad \text{(ASD)}$$

$$r_n/\Omega = [(90)(0.601)]/2.00 = 27.0 \text{ kips} \quad \text{(ASD)}$$

Available tension strengths for ASTM A325, A490 and A307 bolts may be read directly from Table 7-2 in the *Manual* Part 7.

High-strength bolts having countersunk heads should be avoided in applications where they will be stressed in tension. Nominal stresses, F_{nt}, in AISC *Specification* Table J3.2 are based on fasteners with full heads and nuts.

Joints with Fasteners in Combined Shear and Tension

Fasteners subjected simultaneously to shear and tension loads require special analysis to ensure conformance to the AISC *Specification* provisions for F_{nv} and F_{nt}. Joints with this type of combined loading occur frequently in end connections of diagonal bracing members (see Figure 3-5).

AISC *Specification* Section J3.7 provides interaction equations establishing design stresses for fasteners subject to combined shear and tension. In analyzing this type of connection, stresses for shear and tension are considered separately and are not combined as a single resultant force on the fastener.

Bearing Connections in Combined Tension and Shear

Available tensile strength of bolts in bearing-type connections subjected to tension are determined using the values of F_{nt} found in Table J3.2, but are modified per the equations found in AISC *Specification* Section J3.7. Further exposition on the topic of combining the effects of tension and shear on bolts is found in Section J3.7 of the Commentary to the AISC *Specification*.

Slip-Critical Connections in Combined Tension and Shear

Interaction equations for high-strength bolts in slip-critical connections subject to combined shear and tension loading differ from bearing connections in that the available slip resistance is reduced in value by factors given in AISC *Specification* Section J3.9.

BEAM REACTIONS

When detailing a beam, the steel detailer first must select the end connections to transmit the beam factored load to its supporting members. In order to achieve the most economical connections, the end reaction of each beam should be shown on the design drawings. In the *Manual* Part 2 the discussion on “Simple Shear Connections” states that, although not recommended, if beam reactions are not shown on the design drawings, beam end reactions can be approximated as a percentage of the tabulated uniform load from the maximum total uniform load tables for beams in the *Manual* Part 3 for the purpose of connection design.

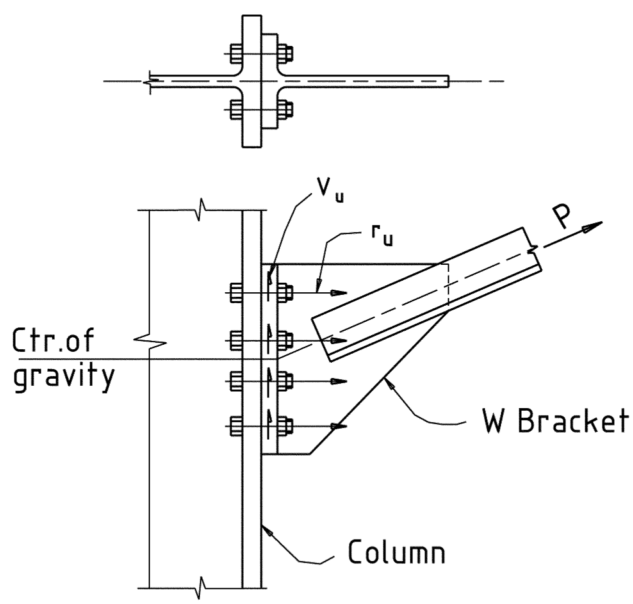


Figure 3-5. Typical end connection of a diagonal bracing member.

Care must be observed in determining end reactions if the beam uniform load constants are used. There are several situations in which this approach is not appropriate.

For example:

- When beams are selected for serviceability considerations or for shape repetition, the uniform load tables will often result in heavier connections than would be required by the actual design loads.
- When beams have relatively short spans, the uniform load tables will often result in heavier connections than would be required by the actual design loads.
- When beams support other framing beams or other concentrated loads occur on girders supporting beams, the end reactions can be higher than 50% of the total uniform load.
- For composite beams, the end reactions can be higher than 50% of the total uniform load. The percentage requirement can be increased for this condition, but the resulting approach is still subject to the preceding considerations.

COMMON BOLTED SHEAR CONNECTIONS

Section B3.6 of the AISC *Specification* defines two types of connections: simple connections and moment connections. Moment connections are divided into two types: Type FR (fully restrained) and Type PR (partially restrained). Type FR, commonly designated "rigid-frame" (continuous frame), assumes that connections have sufficient rigidity to maintain the angles between intersecting members. Type PR assumes that connections do not have sufficient rigidity to maintain the angles between intersecting members. The type of connections specified in the design shall be indicated on the contract documents. The steel detailer is encouraged to become familiar with AISC *Specification* Section B6. When the design does not require connection restraint, commonly designated "simple framing," the steel detailer can assume that, for the transmission of gravity loads, the ends of the beams and girders are connected for shear only and the connection can accommodate the beam end rotation. This type of connection is called a "simple shear connection."

The *Manual* Part 10 describes the seven types of simple connections in which high-strength bolts are used: double-angle, shear end-plate, unstiffened seats, stiffened seats, single-plate, single-angle, and tee connections. All of these connection types have been tested extensively and have excellent performance histories in actual use under a wide range of conditions. Shear end-plate, stiffened seats, and single-plate connections require welding in addition to bolts to share in transferring factored loads from beams to supporting members. Double-angle, unstiffened seat, single-angle and tee connections may utilize high-strength bolts (shop or field) and weld (field or shop) in combination to complete the connection.

The steel detailer is referred to the *Manual* Part 10 for examples and tabulations of these connections. For most of the connections that the steel detailer will use, the tables provide the design data and the extent of applicability. When checking available bearing strengths in these connections, the steel detailer should refer to Tables 7-5 and 7-6 in the *Manual* Part 7. Bearing values are given for 1-in.-thick material. To determine the available bearing strength, multiply the tabulated value by the material thickness. Where lines of bolts are used (as in double-angle connections), the preferred bolt pitch is 3 in. The exception to this preference is found in the bolted connections of unstiffened seats.

Double-Angle Connections

The double-angle connection is very versatile in that the angle thickness, leg widths, fastener and weld sizes, gages, length and material type can vary in order to accomplish the needed performance. The primary function of this connection is shear resistance resulting from gravity loading. It also has good resistance to torsion, which occurs when the member is twisted. If there is axial loading in the beam, the double angle connection offers excellent resistance to axial compression loads in the beam, usually at the expense of requiring angles thicker than those generally used in framing connections. However, unless the angles are properly stiffened or thickened, the connection has moderate resistance to axial tension loads. The relative rigidity of this type of connection varies greatly depending on the length of the angles, the fastener size, and location. The use of double angles, even with the minimum number of rows, frequently results in a connection with excessive design strength compared to design reaction. The AISC *Manual* recommends that the minimum length of angle be at least one-half the *T*-dimension of the beam web to provide stability during erection.

The thickness and length of connection angles can be determined from Table 10-1 in the *Manual* Part 10, but the width of angle legs is not given. The width depends upon the gages used, which in turn depend upon the diameter and type of fasteners and the required clearances for tightening field bolts in the outstanding legs. A discussion of gages and the width of angle legs is covered in Chapter 7 of this publication. In the *Manual* Part 10, examples of the size of the angle leg and the transverse spacing between holes in the outstanding legs are given. This information is usually determined at the beginning of a detailing job by an experienced steel detailer. Many fabricators have "shop standards," which have the connection material sizes and fastening methods predetermined. Figure 3-6 illustrates the connection of a W18×60 to a W24×76. It shows the *T* and *k* found in the tables of dimensions in the *Manual* Part 1. Note that the *Manual* tabulates both a *k* for design and a *k* for detailing. These values are based on surveys of industry practices and are described as follows: *k*-design is a lesser value that maximizes web heights and *k*-detailing is a greater value that can be used in detailing to check connection material encroachment onto the fillet at the web-to-flange junction.

When similarly detailed beams are framing between the webs of several columns in a line, care must be exercised to not shorten the structure by the cumulative amount each beam is shortened. The erector may resolve this condition by inserting shims between the framing angles and webs of some of the columns. Fabrication tolerances for this condition are provided in paragraph 6.4.1 of the AISC *Code of Standard Practice*.

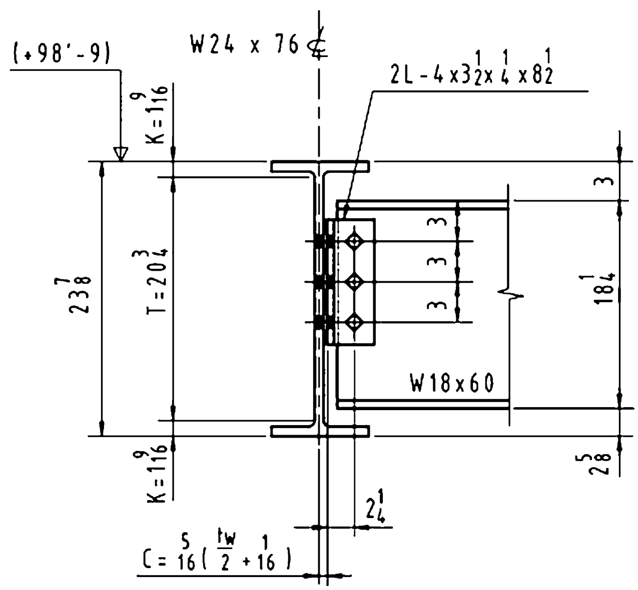


Figure 3-6. Bolted double-angle connection.

When using double-angle connections and the diameter and/or type of fastener in the beam web leg of the angle differs from that used in the outstanding leg, the tables may indicate a different thickness would be required for each leg. In such cases, investigate the bearing strength of the thinner size to determine if it can resist the design load.

The usual practice is to furnish the same number and diameter of fasteners in each leg. While this rule is not 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 shear end-plate is to resist gravity load. To ensure the rotational flexibility required in the design of these connections, plates in the 1/4- to 3/8-in. thickness range are used. An end-plate has good resistance to axial compression in the beam. Usually, however, in the thickness range noted earlier 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 can offer 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

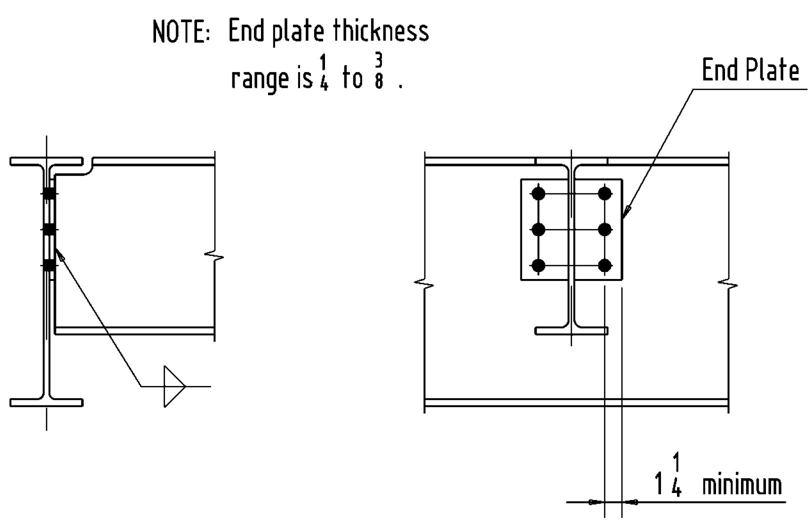


Figure 3-7. Typical end-plate connection to the web of a beam.

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.

The steel detailer is referred to the *Manual* Part 10 for further information, an example, and Table 10-4, which are to be used in selecting an end-plate connection.

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, i.e., no common bolts through the column web. 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 (e.g., 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 7.
- In the case of larger size beams, seated connections reduce the number of field bolts to be installed, resulting in overall economy.

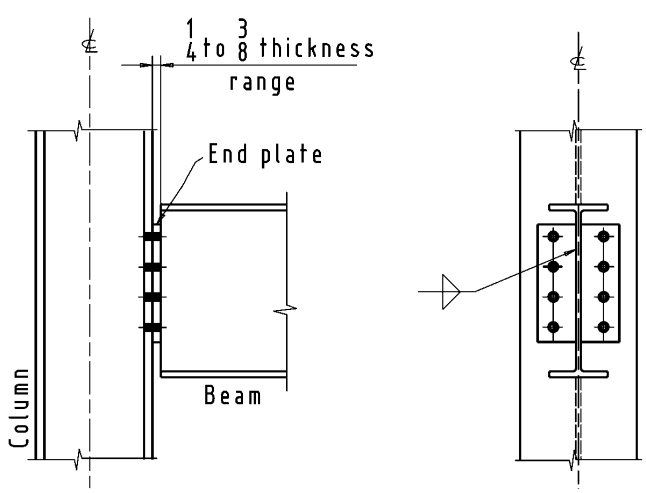


Figure 3-8. Typical end-plate connection to a column flange.

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, but before the imposition of superimposed loads.

The strength of the supported beam web must be checked for web crippling and local web yielding. Thus, the thickness of the supported beam web becomes an important factor in determining the strength of a seated connection.

Unstiffened Seated Connections

A common situation when unstiffened seat angles are used to support beams is illustrated in Figure 3-10, a beam connecting to the web of a column. Ordinarily, the end of a seated beam is stopped approximately $\frac{1}{2}$-in. short of the face of the supporting member to which the seat is attached. The outstanding leg of an unstiffened seat angle is usually 4 in. wide, thus providing a nominal $3\frac{1}{2}$ in. of bearing, measured longitudinally along the web of the supported beam. Factors for checking beam web crippling and local web yielding are given for each size of beam in the tables of "Maximum Total Uniform Loads" in the *Manual* Part 3.

When beam end reactions require more than a nominal $3\frac{1}{2}$ in. of bearing or exceed the strength of unstiffened seats the steel detailer must defer to the use of stiffened seats (see "Stiffened Seated Connections" later in this chapter). The procedure for designing unstiffened seats for longer bearing lengths or higher end reactions is beyond the scope of this text. If design drawings require such seats, they should be shown on the design drawings.

Table 10-5 in the *Manual* Part 10 provides data for selecting several types of unstiffened bolted seated beam connections. The table values shown depend on:

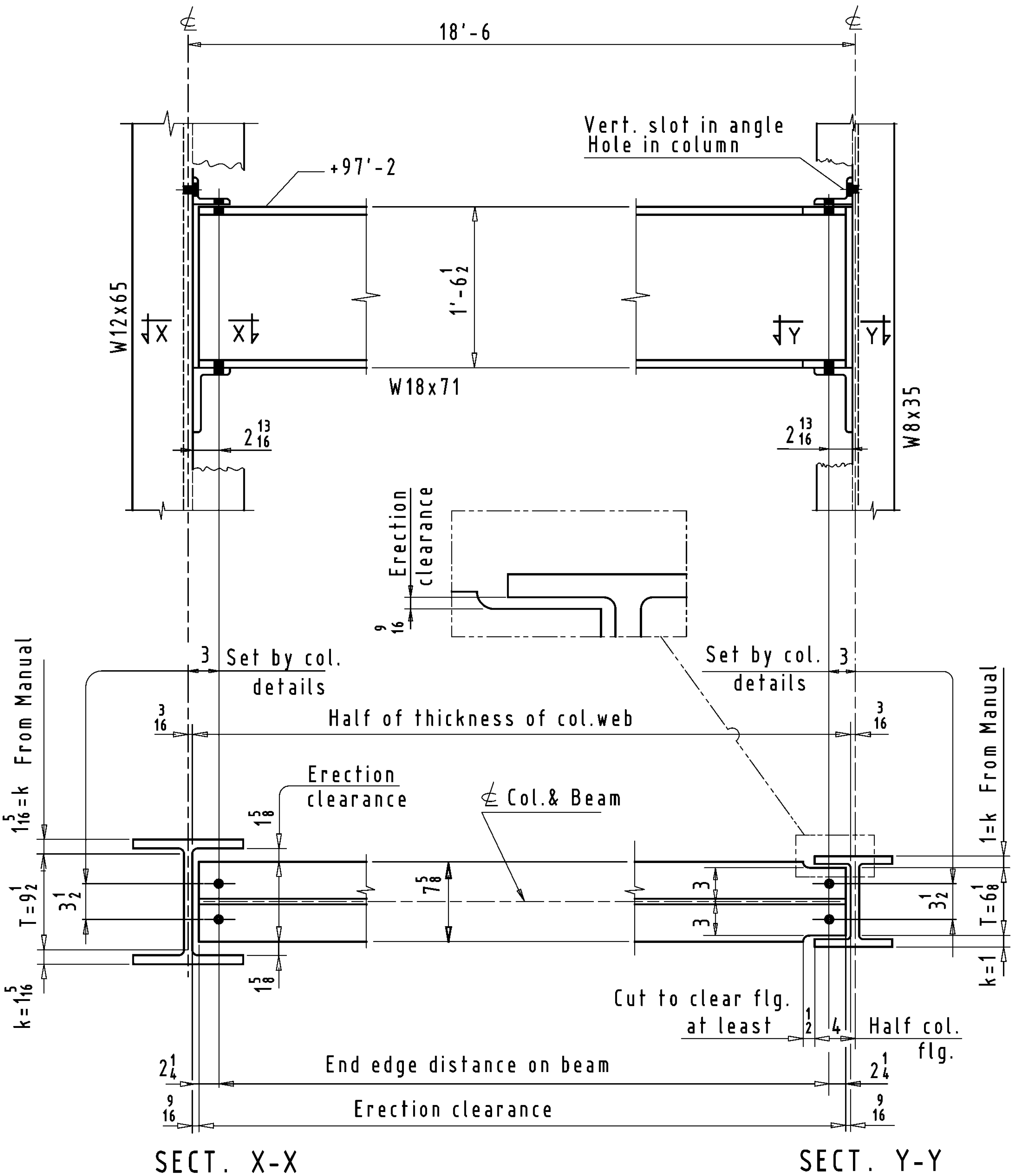


Figure 3 -9. Erection clearances.

- Length of the seat angle
- Thickness of the seat angle
- The required bearing length

Two lengths of seat angle (6 in. and 8 in.) are tabulated in Table 10-5. An increase in the length of these seat angles would add some stiffness to the outstanding leg but would not materially increase the strength of the connection, as this strength is limited by local web yielding of the supported beam.

Table 10-5 gives the total bolt available strength for Connection Types A through F, and the angle sizes for Connection Types A through F. These Connection Types are illustrated in the *Manual* Part 10 in the discussion of Table 10-5. The available shear strengths of these connections are dependent on the size, type and number of fasteners in the connection. Bolts in these seated connections are subject to shear and bearing, just as in the case of framed beam connections. The small eccentricity due to the location of the supported beam reaction is neglected because it has no appreciable effect on the fastener strength in shear and tension.

Table 10-5 lists the available angle sizes and range of thickness available for the seats.

The following procedural steps are recommended when using Table 10-5:

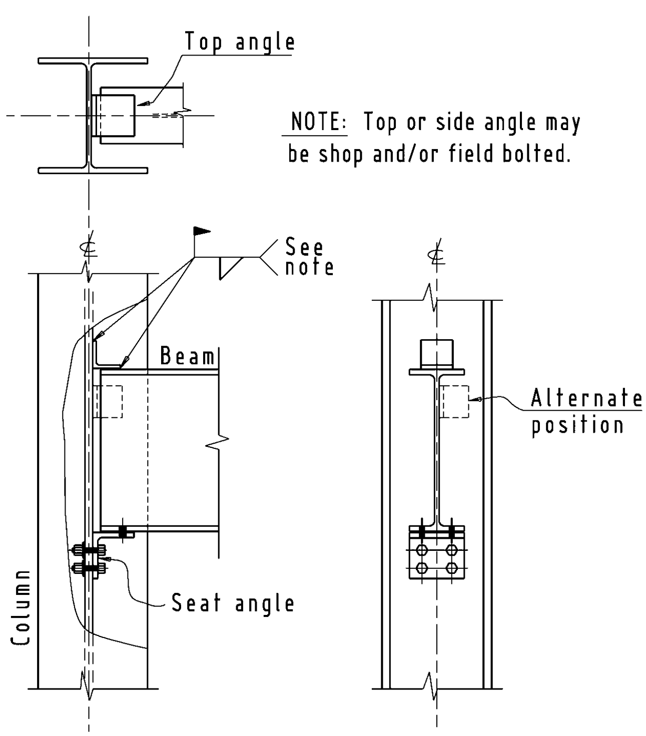


Figure 3-10. Unstiffened seated connection.

1. For the given fastener and type, select the Connection Type and the bolt available strength that will most economically satisfy the end reaction.
2. Investigate the available bearing strength of the fasteners with respect to the thickness and F_u of the supporting material, using Tables 7-5 and 7-6, “Available Bearing Strength at Bolt Holes,” in the *Manual* Part 7.
3. Depending upon the F_y of the steel in the supported beam (ASTM A992 has F_y = 50 ksi) and using the table for angle length of 6 in. or 8 in. (depending on the gages and space available on the supporting member), select the thickness required for the given beam web thickness.
4. Select the most economical angle size available for the required thickness and type.

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, as noted previously in “Shear in High-Strength Bolted Connections,” the available 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

When a beam reaction exceeds the outstanding leg strength of an unstiffened seated beam connection as listed in Table 10-5, a stiffened seated beam connection may be used. In such conditions, stiffeners are fitted to bear on the underside of a seat plate in all bolted stiffened seats (see Figure 3-11).

Table 10-7 in the *Manual* Part 10, All-Bolted Stiffened Seated Connections, gives available strengths of the outstanding legs of two stiffener angles having 3½-in., 4-in. and 5-in. outstanding legs with a thickness range of $^5/_{16}$ to $^3/_4$ in. inclusive. Values are given for steel with F_y = 36 ksi and F_y = 50 ksi.

To allow for a nominal beam setback of ½-in. and a ¼-in. underrun in beam length, 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 available bearing strength of a $3^1/_2$-in-wide and $^3/_8$-in.-thick stiffener angle of A36 steel is computed as follows (see AISC *Specification* Section J7). One stiffener:

$$R_n = 1.8 \times F_y \times A_{pb} \qquad \text{(AISC \textit{Spec.} Eqn. J7-1)}$$

$$\phi R_n = 0.75 \times 1.8 \times 36 \times {}^3/_8(3^1/_2 - {}^3/_4) = 50.1 \text{ kips} \qquad \text{(LRFD)}$$

$$R_n/\Omega = [1.8 \times 36 \times {}^3/_8(3^1/_2 - {}^3/_4)]/2.00 = 33.4 \text{ kips} \qquad \text{(ASD)}$$

Two stiffeners:

$$\phi R_n = 50.1 \times 2 = 100 \text{ kips} \qquad \text{(LRFD)}$$

$$R_n/\Omega = 33.4 \times 2 = 66.8 \text{ kips} \qquad \text{(ASD)}$$

Thus, 66.8 kips and 100 kips are the allowable and design strengths (ASD and LRFD, respectively) 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 accommodate the outstanding leg of the stiffener angle used. The number of bolts required plus the end distances determines the length of the stiffener angle.

The available strengths of connections with various fastener types and diameters for a varying number of fasteners in one vertical row of each of two stiffener angles are listed in Table 10-7. These strengths are based on the single available shear strength of the fasteners. As previously discussed, the available bearing strength of the connected material with respect to the fasteners also should be considered.

Another factor to consider when designing a stiffened beam seat is the compressive stress in the web of the beam directly above the seat plate and stiffeners. The width of the outstanding leg of stiffeners and the resultant length of bearing must be such that the available compressive strength in the beam web at the toe of the fillet does not exceed the local web yielding or nominal web crippling strengths called for in AISC *Specification* Sections J10.2 and J10.3. The steel detailer is referred to the *Manual* Part 10 for examples.

The steps in selecting a stiffened seated connection are:

1. Determine the length of bearing to prevent web crippling and local web yielding of the supported beam.
2. Determine the number of fasteners required.
   (a) Using Table 10-7 determine the number of fasteners of the specified type and diameter that are required for the beam reaction.
   (b) Investigate bearing strength of connected material with respect to the fasteners.
3. Determine size and thickness of stiffener angles. From Table 10-7 find the required thickness, using the width

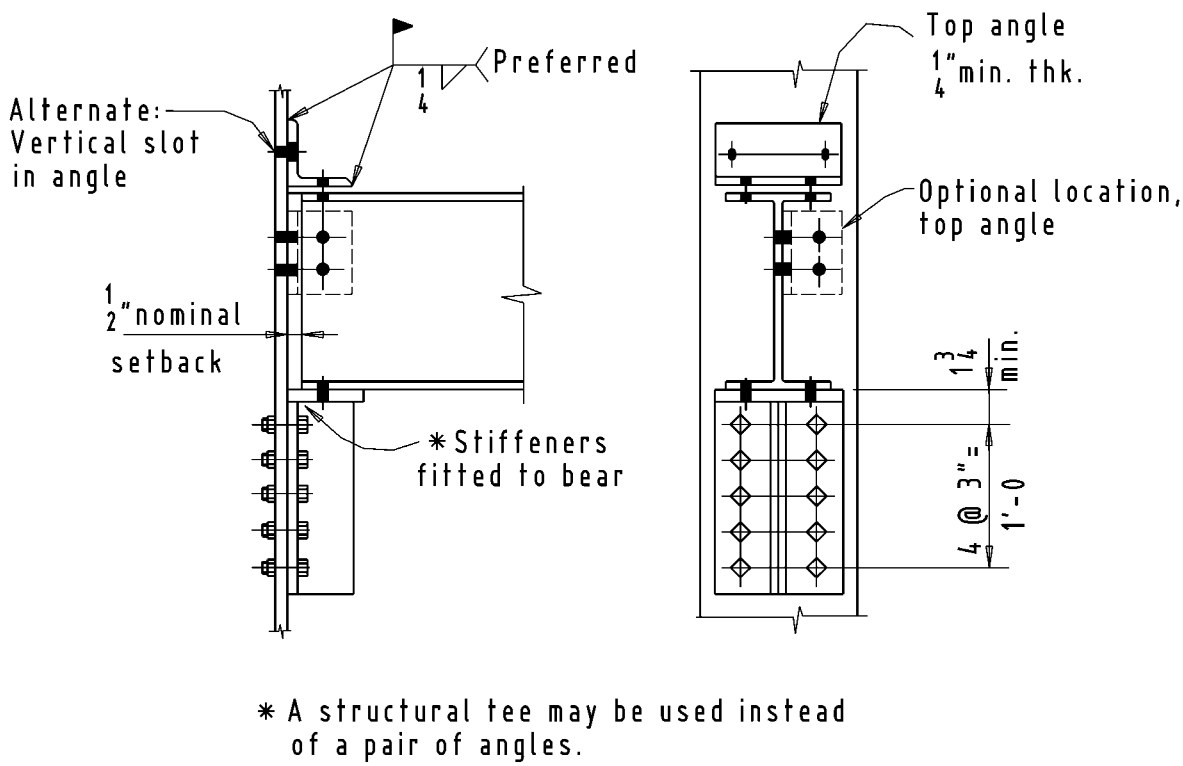


Figure 3-11. Stiffened seated connection.

of stiffener leg that furnishes the required length of bearing obtained from step 1. Check the angle sizes in the "Dimensions and Properties" tables in the *Manual* Part 1 to make certain that the selected combinations of stiffener angle sizes and thicknesses are available.
4. Determine size of seat plate.
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 bolts, reaming (if required), and pretensioning the bolt (if required). 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.

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 Tables 10-9a and b 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

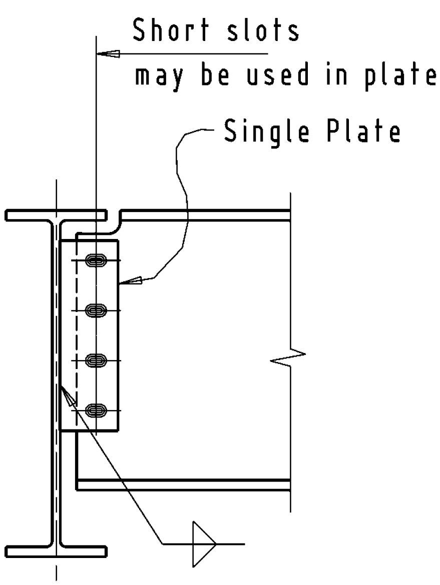


Figure 3-12. Single-plate connection to the web of a beam.

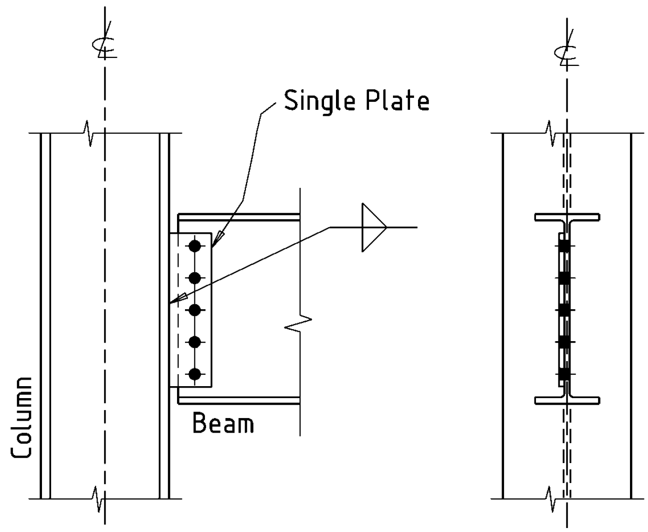


Figure 3-13. Single-plate connection to a column flange.

not be used in cases where it would be subject to torsion loads. Because of its flexibility, it is unsuitable for resisting axial tension loads in the beam.

Figure 3-14 illustrates a single-angle connection. Recommended practice is to connect the angle to the supporting member in the shop, either by bolting through standard holes or by welding. Connecting the single angle to the supported member in the shop causes this connection to be far more difficult to erect. This type of connection often can be used where interferences prohibit the use of a double-angle connection. The field connection is made by bolting through the angle leg into the web of the filler beam. Horizontal short slots can be used in the field-connected leg to accommodate fabrication and erection tolerances. The slots also accommodate the beam end rotation associated with cambering.

In the *Manual* Part 10, the steel detailer will find additional information, an example and Table 10-10 for designing single-angle connections.

Tee Connections

A tee connection is primarily a shear connection. The tee can either be cut from a W or HP shape or fabricated by welding two plates together (useful for skewed connections). Only W and HP shapes that fit criteria for web and flange thicknesses, flange width and other dimensions are suitable for use as tee connections. To provide for stability during erection, the recommended practice is to furnish a tee having a minimum length equal to one-half the *T*-dimension of the supported beam. The maximum length must be compatible with the *T*-dimension of an uncoped beam and the remaining web depth, exclusive of fillets, of a coped beam.

The flange thickness of tees should be held to a minimum to permit the flexure necessary to accommodate the end rotation of the beam, unless the tee stem is designed to meet the requirements for single-plate connections.

The tee connection has good resistance to axial compression load in the beam, but relatively poor resistance to axial tension. Resistance to torsion is fair even when using the minimum length of tee allowed by the beam web.

The tee is usually shop attached to the supporting member (Figure 3-15). However, on rare occasions it can be shop attached to the supported beam, but this negates some of the advantages of the tee connection. Bolts or weld can be used to attach to the supporting member. Standard round holes or horizontal short slots can be used in both the flange and web of the tee.

This connection is both safe and fast to erect. However, it is not the most economical connection because of the extra shop work involved in preparing the tee.

In the *Manual* Part 10, the steel detailer will find additional information for use in selecting tee connections.

FORCES IN WELDS

Welds may be loaded in shear, tension, compression or a combination of these, depending on the direction and point of application of external forces and the arrangement of joint components. Available strengths for welds in buildings are given in AISC *Specification* Section J2.4.

The nomenclature commonly used in design computations involving strengths of welds employs the following symbols:

ϕR_n = Design strength of weld or base metal using LRFD, kips

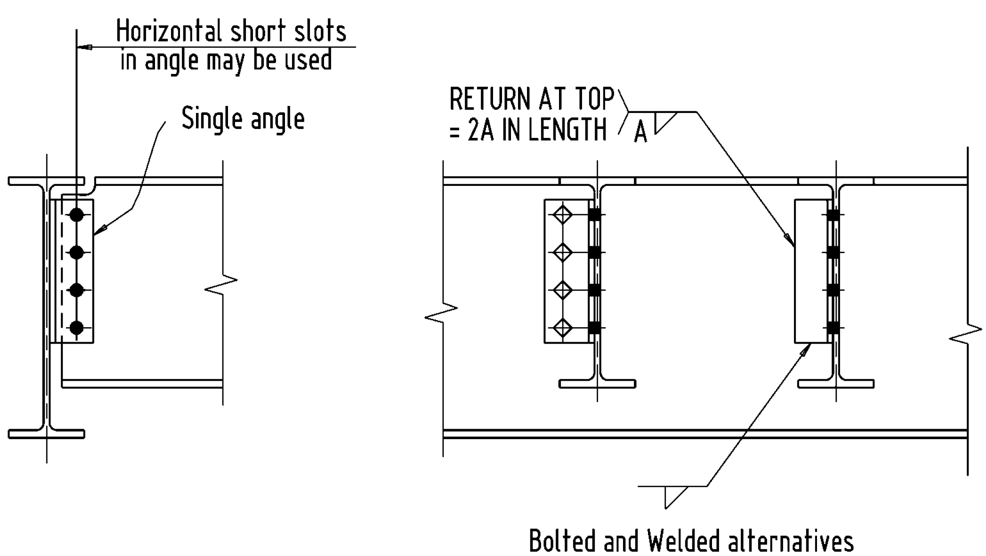


Figure 3-14. Single-angle connection.

R_n/Ω = Allowable strength of weld or base metal using ASD, kips
D = Number of sixteenths of an inch in fillet weld size
F_{BM} = Nominal strength of the base metal to be welded per unit area, ksi
F_w = Nominal strength of the weld metal per unit area, ksi
ϕr_n = Design strength of a 1-in. length of weld of a given size using LRFD, kips
r_n/Ω = Allowable strength of a 1-in. length of weld of a given size using ASD, kips
l = Effective length of weld, in.
t = Material thickness, in.
w = Weld size (leg size for fillet welds and depth of filling for plug and slot welds), in.
A_{BM} = Cross-sectional area of the base material, in.^{2}
A_w = Effective cross-sectional area of the weld, in.^{2}
R_u = Required strength (load, force) using LRFD, kips
R_a = Required strength (load, force) using ASD, kips
ϕ = Resistance factor
Ω = Safety factor

The following discussion deals with forces in concentrically loaded welded joints in which the welds are assumed to be loaded uniformly throughout their lengths. Eccentrically loaded welded joints, in which the welds are loaded in varying amounts along their lengths, are not discussed in this text. Refer to the *Manual* Part 8 for coverage of eccentric loading and its effects on weld joints. See Figure 3-16 for nomenclature of the common terms related to fillet and groove welds.

Forces in Concentrically Loaded Fillet Welds

The critical force in a fillet weld is always considered to be a shear force and may occur in one of two directions: (1) parallel to the axis of the weld or (2) transverse to the axis of the weld. Therefore, tension, compression and moment forces acting upon a fillet welded joint are always resolved on the basis of shear in the weld throat.

Figure 3-17a shows a fillet welded lap joint with welds A loaded in parallel shear and weld B loaded in transverse shear. If loads R_u or R_a are increased enough to exceed the total strength of these welds, rupture will occur in the planes of least resistance. As shown in Figure 3-17b, this is assumed to be in the weld throats, where the least cross-sectional area is present.

For design purposes, the effective area of a fillet weld is calculated as the product of its effective throat and its effective length. The effective throat area[1] equals 0.707wl. The deep penetration of fillet welds made by the submerged arc process is recognized in AISC *Specification* Section J2.2.

The nominal strength of a fillet weld is equal to the product of its effective throat area and the nominal strength of the

[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.

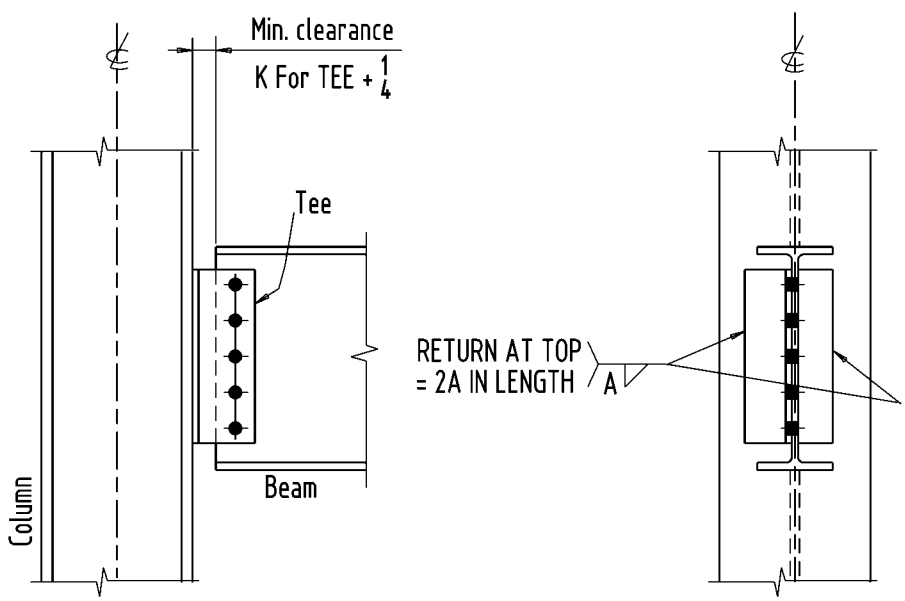


Figure 3-15. Tee connection.

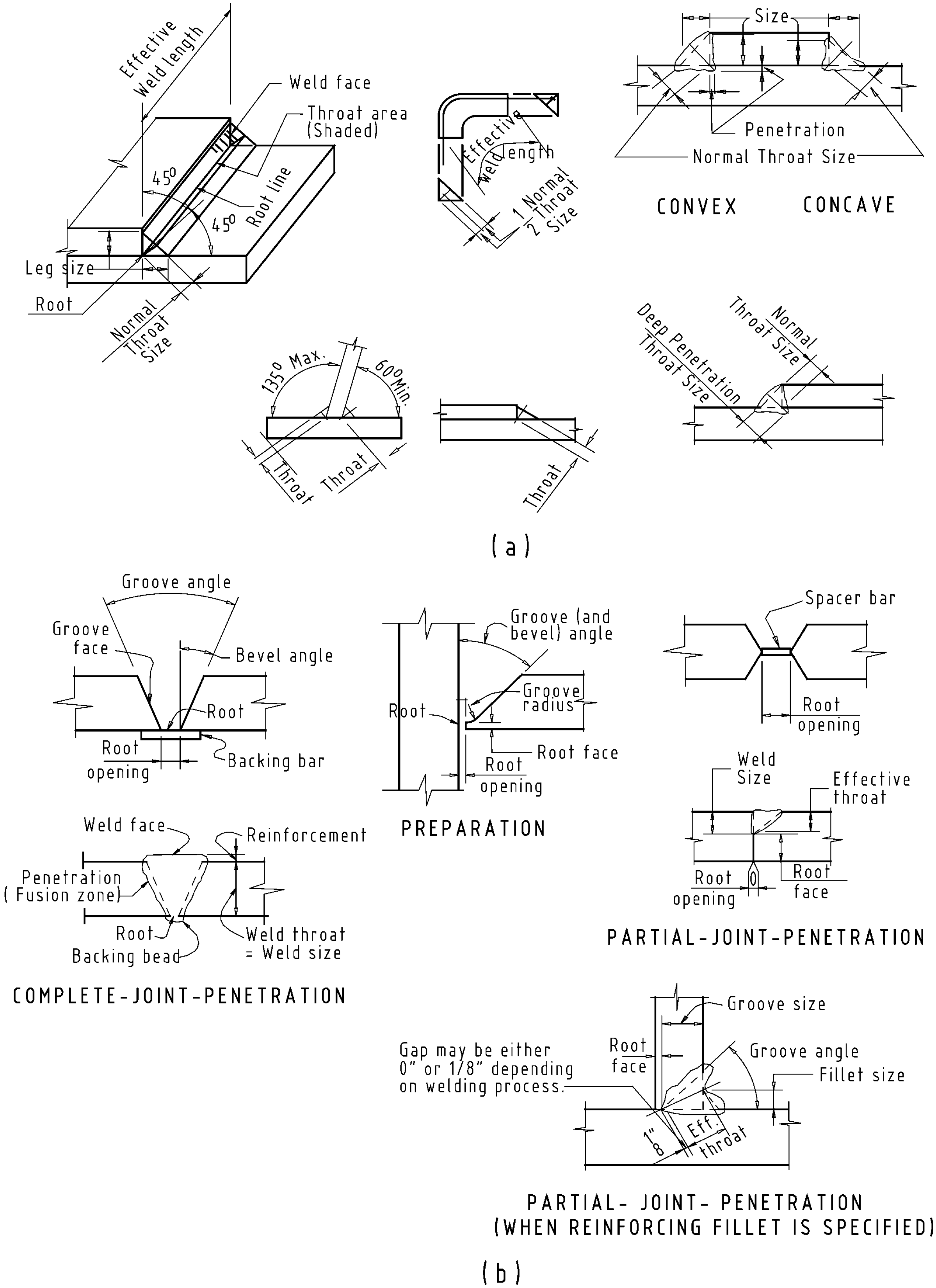


Figure 3-16. Common fillet and groove weld nomenclature.

weld metal, F_w, permitted by the specifications. Similarly, the strength of a concentrically loaded fillet weld group is the sum of the strengths of each fillet weld in the group. Nominal strengths for the weld metal, F_w, for fillet welds permitted by AISC *Specification* Section J2.2 are shown in Table J2.5 of that specification.

Assuming that the F_w value is appropriate to the steel being welded and normal throat areas are applicable, the strength of a fillet weld in shear may be expressed as:

$$\phi R_n = \phi \times F_w \times A_w \quad \text{(LRFD)}$$
$$\phi R_n = 0.75 \times (0.60\, F_{EXX}) \times 0.707wl \quad \text{(LRFD)}$$

$$R_n/\Omega = (F_w \times A_w)/\Omega \quad \text{(ASD)}$$
$$R_n/\Omega = [(0.60\, F_{EXX}) \times 0.707wl]/\Omega \quad \text{(ASD)}$$

$R_n \quad = 0.424F_{EXX}wl$, where F_{EXX} is the electrode classification number (Grade) of the electrode

Table 3-1 lists values of ϕr_n and r_n/Ω for fillet welds of various sizes, based on nominal throat areas, for F_{EXX} values of 60, 70 and 80 ksi weld electrodes. Also included in this table are ϕr_n values based on deep penetration throat areas for welds made by the submerged arc process. Table 3.1 in the American Welding Society *Structural Welding Code-Steel* (AWS D1.1) (AWS, 2008), lists the base and filler metals that are permitted to be used with ("match") each of the electrodes stated earlier. AWS D1.1 is discussed in Chapter 4. AISC *Specification* Table J2.5 states that filler metal one strength level less than matching filler metal is permitted to be used for groove welds between the webs and flanges of built-up sections transferring shear loads or in applications where high restraint is a concern. The steel detailer is not responsible for ensuring that this requirement is met, but should be aware of it.

When the available shear strength of fillet welds is based on normal throat areas, as is usually the case, determining fillet weld strengths may be simplified by memorizing the available shear strength value of a $^1/_{16}$-in. fillet weld, per in. of length, for the most frequently used electrode strength levels.

For example, the available shear strength value for a $^1/_{16}$-in. fillet weld, 1-in. long, with $F_{EXX} = 70$ ksi, is:

$$\phi r_n = 0.75 \times (0.60 \times 70) \times (0.707 \times {}^1/_{16} \times 1) = 1.392 \text{ kips/in.} \quad \text{(LRFD)}$$

$$r_n/\Omega = [(0.60 \times 70) \times (0.707 \times {}^1/_{16} \times 1)]/2.00 = 0.928 \text{ kips/in.} \quad \text{(ASD)}$$

Using the letter D to denote the number of sixteenths of an inch in the leg size, the value per inch of any size of fillet weld (where $F_{EXX} = 70$ ksi) can be expressed as $\phi r_n = 1.392D$ (LRFD) and $r_n/\Omega = 0.928D$ (ASD).

Limitations on Length and Size of Fillet Welds

The minimum effective length of a fillet weld, when used alone and not as part of a continuing joint boundary, is limited to four times the nominal size. Thus, the shortest $^5/_{16}$-in. weld that can be considered fully effective to transmit loads is $4 \times {}^5/_{16} = 1^1/_4$ in. Conversely, its maximum effective size can be no greater than $^1/_4$ of the weld length (see AISC *Specification* Section J2.2b).

Intermittent fillet welds likewise are subject to the preceding rules, with the added requirement that the increment length must be not less than $1^1/_2$ in. (see AISC *Specification* Section J2.2b).

AISC *Specification* Section J2.2b places a limitation on the maximum size of fillet welds against the edges of connected parts of a joint. For material less than $^1/_4$-in. thick, the weld size may equal the thickness of the material. For material $^1/_4$-in. or more in thickness, the maximum size weld against an edge shall be $^1/_{16}$-in. less than the thickness of material, unless the drawing is noted specifically to build up the weld to achieve full throat size. This limitation recognizes that the

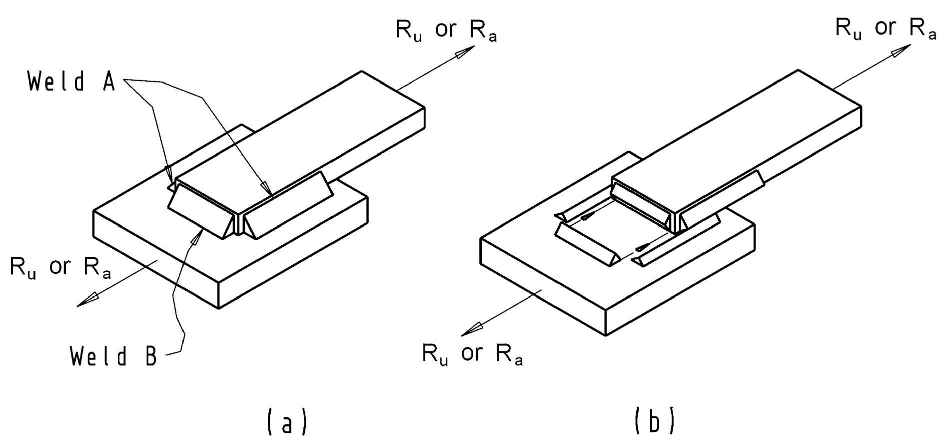


Figure 3-17. Fillet weld shear failure.

Table 3-1. Design Strength of Fillet Welds (ϕr_n or r_n/Ω), kips/in.

Weld Size (in.)	LRFD[1] ϕr_n		ASD[1] r_n/Ω	
	Grade 70 F_{EXX} 70 ksi	Grade 80 F_{EXX} 80 ksi	Grade 70 F_{EXX} 70 ksi	Grade 80 F_{EXX} 80 ksi
1/8	2.78	3.18	1.86	2.12
3/16	4.18	4.77	2.78	3.18
1/4	5.57	6.36	3.71	4.24
5/16	6.96	7.95	4.64	5.30
3/8	8.35	9.55	5.57	6.36
7/16	9.74	11.1	6.50	7.42
1/2	11.1	12.7	7.42	8.48
9/16	12.5	14.3	8.35	9.54
5/8	13.9	15.9	9.28	10.6
3/4	16.7	19.1	11.1	12.7
1	22.3	25.4	14.8	17.0

[1]Values are for static loading only. See AISC *Specification* Appendix 3 for connections subject to high cycle loading.

upper corner of the welded edge tends to melt down into the weld, as shown in Figure 3-18(a), reducing the leg dimension and weld throat. Also, the toes of most rolled shapes are rounded as shown in Figure 3-18(b) and the actual thickness of the weld is less than the nominal thickness, *t*, of the member.

The AISC *Specification* also specifies the minimum size of fillet weld to be used in a joint. Section J2.2b provides Table J2.4 showing minimum fillet weld sizes. The minimum weld size is specified according to the thickness of the thinner of the two parts joined. For example, if two 7/8-in. plates are joined, the minimum size of fillet weld permitted is 5/16 in., even though a 1/4-in. weld might suffice to transfer the force across the joint. However, where dissimilar thicknesses are encountered, the weld size need not exceed the thickness of the thinner part, unless a larger size is required by stress calculations. These size limitations have been established to avoid cracks that might occur in the welds because of internal forces created by the rapid and often uneven cooling after welding. For quick reference, the minimum fillet weld size required for a given material thickness of the thinner part connected in a joint, based on AISC *Specification* Section J2.2b, is given in Table 3-2.

AISC *Specification* Section J2.2b describes limitations on fillet welds, which deal with lengths of fillet welds, proportions

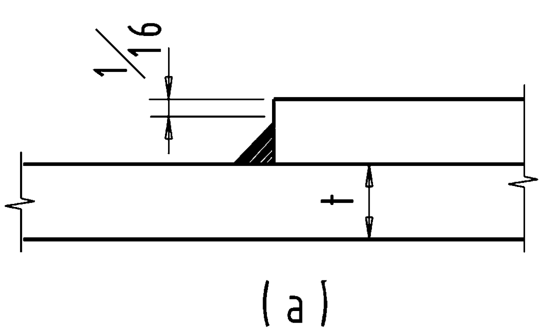


Figure 3-18. Resultant fillet weld dimensions.

Table 3-2. Minimum Permissible Fillet Weld Sizes

Material Thickness of Thinner Part Joined, in. (mm)	Minimum Size of Fillet Weld[1], in. (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19)	5/16 (8)

[1]Leg dimension of fillet welds. Single pass wleds must be used.
Note: See Section J2.2b for maximum size of fillet welds.

of lap joints, end returns of fillet welds, and fillet welds in holes and slots. The steel detailer must become familiar with all of these limitations in the AISC *Specification*.

Strength of Connected Material

The transfer of factored load from one element to another is dependent on both the design strength of the weld and the design strength of the materials to resist factored loads.

A simple example is illustrated in Figure 3-19. The plate N is checked for available tension strength. Note that the weld is transversely loaded. Using AISC *Specification* Equation J2-4 with E70XX electrodes, the weld available strengths per 1-in. length are:

$$\phi R_n = \phi \times (0.60 \times F_{EXX})(1.0 + 0.50\sin^{1.5}\theta)(A_w) \quad \text{(LRFD)}$$

$$\phi R_n = 0.75 \times (0.60 \times 70)(1.0 + 0.50\sin^{1.5}90°) \times (0.707wl) \quad \text{(LRFD)}$$

With $l = 1$ in.,

$$\phi R_n = 33.4w \text{ kips/in.} \quad \text{(LRFD)}$$

$$R_n/\Omega = [(0.60 \times F_{EXX})(1.0 + 0.50\sin^{1.5}\theta)(A_w)]/\Omega \quad \text{(ASD)}$$

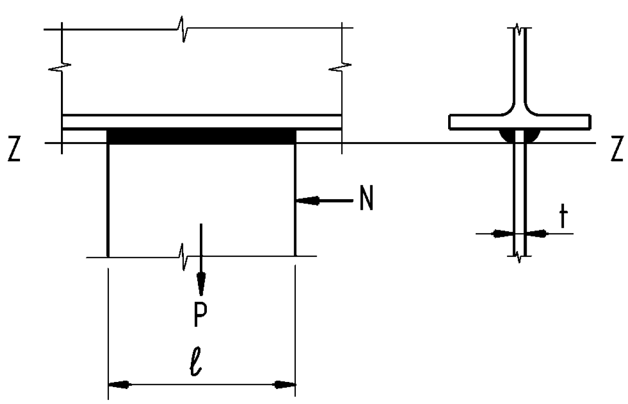


Figure 3-19. Plate loaded in tension.

$$R_n/\Omega = [(0.60 \times 70)(1.0 + 0.50\sin^{1.5}90°) \times (0.707wl)]/2.00 \quad \text{(ASD)}$$

With $l = 1$ in.,

$$R_n/\Omega = 22.3w \text{ kips/in.} \quad \text{(ASD)}$$

The available tensile rupture strengths of the plate per inch are:

$$\phi R_n = \phi_t F_u t, \text{ kips/in.} \quad \text{(LRFD)}$$

$$R_n/\Omega_t = F_u t/\Omega_t, \text{ kips/in.} \quad \text{(ASD)}$$

The available tensile yield strengths of the plate per inch are:

$$\phi R_n = \phi_t F_y t, \text{ kips/in.} \quad \text{(LRFD)}$$

$$R_n/\Omega_t = F_y t/\Omega_t, \text{ kips/in.} \quad \text{(ASD)}$$

The weld size to develop the full tensile strength of the plate can be determined by equating these two values.

For $F_y = 36$ ksi steel:

$$\phi R_n = 0.75 \times 58 \times t = 33.4w \quad \text{(LRFD)}$$

$$R_n/\Omega_t = 58 \times t/2.00 = 22.3w \quad \text{(ASD)}$$

$$w = 1.30t$$

$$\phi R_n = 0.90 \times 36 \times t = 33.4w \quad \text{(LRFD)}$$

$$R_n/\Omega_t = 36 \times t/1.67 = 22.3w \quad \text{(ASD)}$$

$$w = 0.97t$$

The smallest of these two values will control the design; therefore, $w = 0.97t$. For two lines of fillet weld as shown in Figure 3-19:

$$w = 1/2 \times 0.97t = 0.48t \text{ for each line of weld}$$

A different case is shown in Figure 3-20. Here, an angle is welded to a plate or the stem of a tee. Given that the available shear strength, ϕR_n or R_n/Ω, of the weld exceeds the required axial strength, P_u or P_a, the plate (or tee stem) must be checked for tension yielding on the Whitmore section. This section is identified by the width of the plate, L_w, considered to be resisting the load. The value of the width, L_w, must be equal to or less than the actual width of the plate, L, available for resisting the load. Thus, the available tensile strength from AISC *Specification* Section J4.1 is:

$$\phi R_n = \phi \times F_y \times A_{g\,eff} \geq P_u, \text{ with } \phi = 0.90 \quad \text{(LRFD)}$$

$$R_n/\Omega = (F_y \times A_{g\,eff})/\Omega \geq P_a, \text{ with } \Omega = 1.67 \quad \text{(ASD)}$$

F_y = specified minimum yield stress of the type of steel being used, ksi

where

$$A_{g\,eff} = (t \times L_w) \leq (t \times L), \text{ in.}^2$$

L_w = [2 × l × tan (30°)] + d, in.

Secondly, shear yielding of the base metal along the toe and heel of each fillet weld line must be checked. Thus, the available shear yield strength of the plate from AISC *Specification* Section J4.2 is:

$\phi R_n = \phi \times (0.60 \times F_y) \times A_g \geq P_u$, with $\phi = 1.00$ (LRFD)

$R_n/\Omega = [(0.60 \times F_y) \times A_g]/\Omega \geq P_a$, with $\Omega = 1.50$ (ASD)

where

A_g = 4 × l × t, in.^{2} (The factor 4 refers to the number of shear planes)

Thirdly, the plate (or tee stem) must be checked for shear rupture of the base metal along the toe and heel of each weld line. Thus, the available shear rupture strength from AISC *Specification* Section J4.2 is:

$\phi R_n = \phi \times 0.60 \times F_u) \times A_n \geq P_u$, with $\phi = 0.75$ (LRFD)

$R_n/\Omega = [(0.60 \times F_u) \times A_n]/\Omega \geq P_a$, with $\Omega = 2.00$ (ASD)

where

F_u = specified minimum tensile strength of the type of steel being used, ksi

A_n = 4 × l × t, in.^{2} (The factor 4 refers to the number of shear planes)

Figure 3-21 illustrates failure of the plate (or tee stem) by block shear rupture. Any potential way for a connection to fail (block shear rupture, bearing, tearout, etc.) is referred to as a limit state. A limit state is a condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state). The block shear strength is determined by the sum of the shear strength on a failure path(s) and the tensile strength on a perpendicular segment. AISC *Specification* Section J4.3 provides an equation for determining nominal block shear rupture strength, R_n. The basis for the equation is the existence of two possible nominal shear strengths:

Figure 3-21a — Rupture strength on the tensile section ($F_u \times A_{nt}$) along with shear rupture on the shear planes ($0.60 \times F_u \times A_{nv}$).

Figure 3-21b — Rupture strength on the tensile section ($F_u \times A_{nt}$) along with shear yielding on the shear areas ($0.60 \times F_y \times A_{gv}$).

The steel detailer is referred to the CD attached to the *Manual* for an example of the calculations of the available strengths illustrated in Figures 3-20 and 3-21. Note that the rupture strength on the tensile section is compared to the rupture and yield strength on the shear areas—the smaller sum of these values determines the block shear rupture strength of the plate (or tee stem). For an additional explanation of the rupture phenomenon, the steel detailer should read AISC *Specification* Commentary Section J4.

For reasons of shear lag, AISC *Specification* Section D3.3 limits the l/d ratio (Figure 3-21) to obtain the maximum effective tension area. Shear lag occurs when less than the total cross-sectional area of a member is connected, resulting in a decrease in efficiency of the section in the region of the connection.

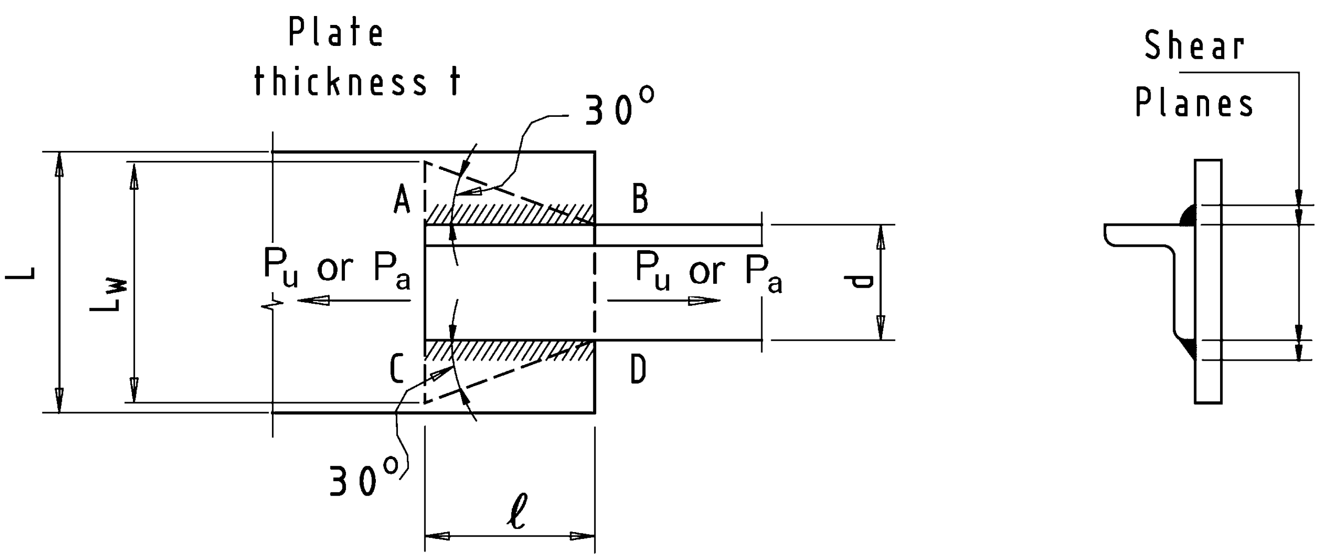


Figure 3-20. Whitmore section yielding of an angle welded to a plate or tee stem.

The foregoing discussion does not apply, necessarily, to the case of double angles welded to the end of the web of a girder. In this case the load transferred into the web is more complex and depends on several factors, such as the distance from the connection to the girder reaction and the relative length of the connection angle fillet weld to the depth of the girder. The usual practice is to assume only one shear plane. Using nominal strengths,

Weld fracture Plate fracture

$$\frac{0.60 \times F_{EXX} \times D \times 2}{16 \times 2^{1/2}} \le 0.60 \times F_u \times t_w$$

for each weld length. The minimum girder web thickness, t_w, is calculated as follows:

Longitudinal shear in the weld is compared to the shear in the girder web plate with θ = 0 (angle of loading measured from the weld longitudinal axis, degrees)

Where

$$t_w \ge \frac{0.0884 \times F_{EXX} \times D}{F_u}$$

with

$$F_{EXX} = 70;$$
$$t_w \ge \frac{6.19 \times D}{F_u}$$

The following table lists the values of t_w for ASTM A36 (F_u = 58 ksi) and A992 (F_u= 65 ksi) steels for varying values of D:

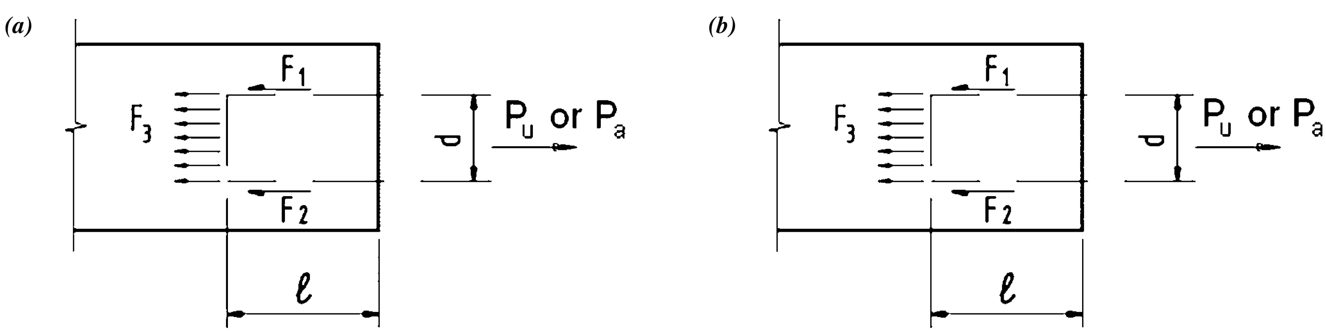
D	t_w for A36	t_w for A992
3	0.32	0.29
4	0.43	0.38
5	0.53	0.48
6	0.64	0.57
7	0.75	0.67
8	0.85	0.76

F_{EXX} = classification strength of the weld metal, ksi
D = number of sixteenths of an inch of weld size
F_u = specified minimum tensile strength of the type of steel being used, ksi
t_w = thickness of web plate, in.

A similar comparison between transverse shear in the weld and tension in the web plate (θ = 90°) provides a plate only slightly thinner. Therefore, to check the required minimum web plate thickness, use the value for t_w given previously. If the weld is not loaded fully, t_w may be reduced proportionately.

Forces in Complete-Joint-Penetration Groove Welds

AISC *Specification* Section J2.4 establishes the available tensile and compressive strength of complete-joint-penetration (CJP) groove welds as being equal to the available strength of the base metal, providing a "match" of base and



(a)

$F_1 = 0.6 \times F_u \times l \times t$ (rupture)
$F_2 = 0.6 \times F_u \times l \times t$ (rupture)
$F_3 = F_u \times d \times t$ (rupture)
$R_n = F_1 + F_2 + F_3$

(b)

$F_1 = 0.6 \times F_y \times l \times t$ (yielding)
$F_2 = 0.6 \times F_y \times l \times t$ (yielding)
$F_3 = F_u \times d \times t$ (rupture)
$R_n = F_1 + F_2 + F_3$

Figure 3-21. Block shear rupture of a plate or a tee stem.

weld metal is made as outlined in Table 3.1 of AWS D1.1. Exceptions are as follows:

- When the load is compressive and normal to the weld axis, filler metal with a strength level equal to or one strength level less than matching filler metal is permitted to be used.
- For any condition of load, weld metal one strength level stronger than "matching" weld metal will be permitted.
- In joints involving base metal of two different yield stress values, use either a filler metal that matches the higher strength base metal or a filler metal that matches the lower strength and produces a low hydrogen deposit. See user note in AISC *Specification* J2.6.

Assuming compliance with these requirements, one can say that complete-joint-penetration groove welds are equal to the base metal in all respects. No allowance for the presence of such welds need be made in proportioning the connections of structural members for any type of static loading. However, where members of unequal cross-section or differing nominal strengths are joined, the splice cannot be considered stronger than the weaker member.

Most building construction is designed on the basis of static loading. However, some parts, such as crane runways, machinery supports, etc., are frequently subject to dynamic loading or cyclic loading. When subject to this cyclic or dynamic loading, consideration of joint details and requirements normally reserved for bridge work should be recognized and applied when appropriate. Contract documents should outline specific requirements to avoid misunderstanding.

Prequalified CJP welded joint profiles are shown in AWS D1.1, Subsection 3.13 and also are reproduced in the *Manual* Part 8, courtesy of AWS.

Forces in Partial-Joint-Penetration Groove Welds

Based on filler metal with a strength level equal to or less than matching filler metal, the nominal strength for partial-joint-penetration groove welds permitted by AISC *Specification* Section J2.4 is equal to 0.60 × nominal strength of the weld metal for compression connections of members designed to bear, other than columns, as described in Section J1.4(b). For compression connections not finished to bear, the nominal strength of PJP welds is given as 0.90 × nominal strength of the weld metal. Nominal shearing forces parallel to the axis of the weld are allowed at 0.60 × nominal strength of the weld metal. Nominal tensile forces normal to the weld axis are allowed at 0.60 × nominal strength of the weld metal.

The last two exceptions listed under the heading "Forces in Complete-Joint-Penetration Groove Welds" also apply to this type of weld. Refer to AISC *Specification* Table J2.5 for additional limitations.

Figure 3-16b shows nomenclature and sketches of a butt and tee partial-joint-penetration groove weld in which the effective throat may be less than the dimensioned groove weld size. AWS D1.1, Subsection 3.12, details the prequalified partial-joint-penetration groove welds (also reproduced in the *Manual* Part 8, courtesy of AWS) and establishes for each weld joint an effective throat, *(E)*, as a function of the material thickness, the weld preparation depth, *(S)*, the groove preparation, and the permitted welding process. This information is summarized in AISC *Specification* Table J2.1.

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

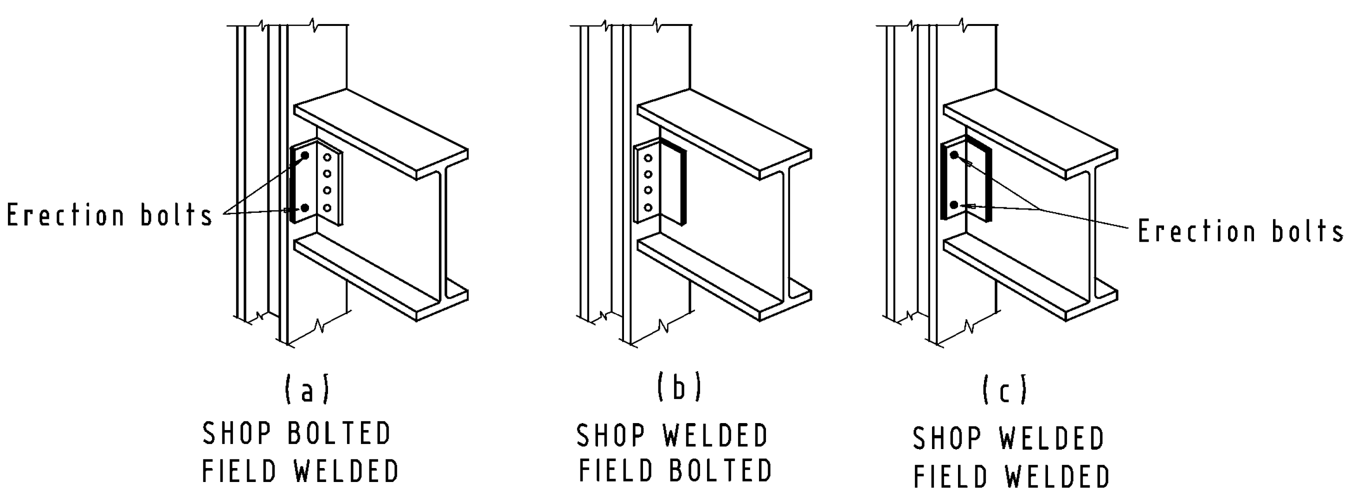


Figure 3-22. Typical welded shear connections.

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.

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

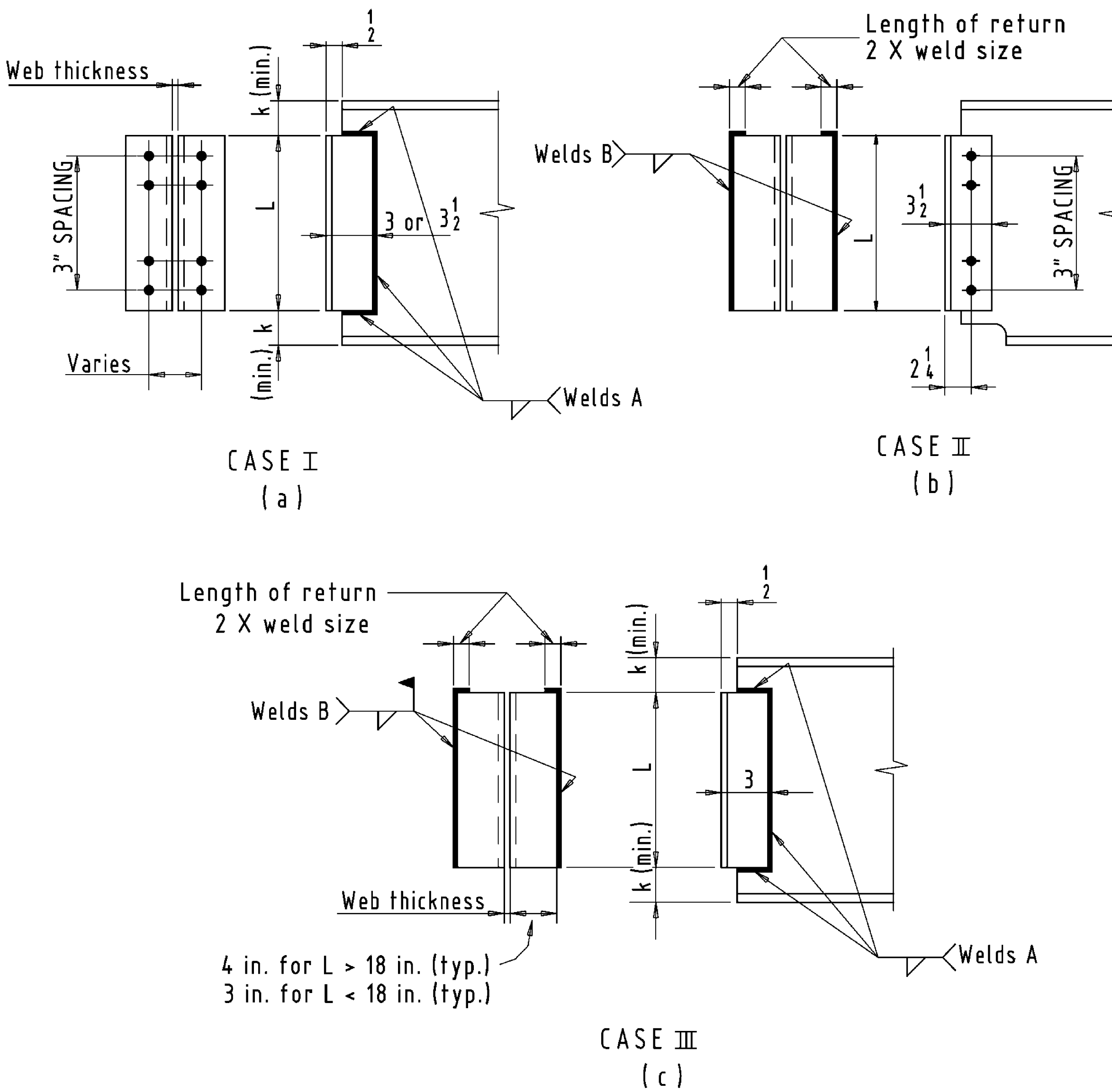


Figure 3-23. Typical welded framed beam connections.

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 available weld 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 available weld strengths for several lengths of connection angles and weld sizes suitable for combination with Table 10-1, of the *Manual* Part 10. Table 10-3 has been developed for a third case, referred to here as Case III, in which the connections are welded completely, in the shop and field. In all of these tables, the available 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 available strength of the number of fasteners selected can be investigated by referring to Table 10-1 or by direct calculation. The available strength of the fastener group must be equal to or greater than the required load to be carried.

The available 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, 2L4×3 1/2 will accommodate usual gages with the 4-in. leg attached to the supporting member. Available strengths are based on beams 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. Available strengths for these connections are based on the use of F_y = 50 ksi steel beams.

In both Case I and III connections, weld A, connecting the angles to the beam web, is loaded eccentrically. The available 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 10-2 covering combinations of welded or bolted connections, the available 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 3 1/2-in.-wide angle web leg permits the use of a standard 2 1/4-in. gage if fasteners are used. However, to provide a more economical connection, the width of the web leg can be reduced from 3 1/2 in. to 3 in. when welds are used and the value determined from Table 10-3.

In Table 10-3, the angle leg against the beam web has been limited to 3 in. Allowing for a 1/2-in. setback of the beam end, the available strength of weld A is based on 2 1/2-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.

The available strengths of weld B in Table 10-3 are based on 4-in.-wide outstanding legs. This width will accommodate the standard 5 1/2-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 available strength 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 available strength of weld A must be reduced by multiplying it by the ratio of actual thickness to tabulated minimum thickness. Thus, using LRFD and Table 10-3, if 1/4-in. weld A, with a design strength of 139 kips and an 8-in.-long angle, is considered for a W14×43 beam (web thickness = 0.305 in.) of F_y = 50 ksi steel, the design strength must be multiplied by 0.305/0.381, giving 111 kips.

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

$$t_{min} = \frac{0.6F_{EXX} \times \sqrt{2}/2 \times D/16 \times 2}{0.6 \times F_u}$$

$$= \frac{6.19 \times D}{F_u}$$

where

t_{min} = minimum web thickness, in. and

D = number of sixteenths of an inch of weld size

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 available weld 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 AISC *Specification* Section J2.2b.

The available 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 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 in step 1, select the weld size required.
3. Check if the angle thickness obtained from step 1 can accommodate the weld size; increase the thickness, if necessary.
4. For weld A, note the minimum web thickness required and reduce the tabulated available strength of the weld if the thickness of beam web is less than the minimum. For weld B, investigate the available strength of the supporting material to receive the weld force.

Size of Connection Angles:

1. In general, for Cases I and II use $4 \times 3^1/_2$ angles with $3^1/_2$-in. web legs and 4-in. outstanding legs.
2. The width of web leg in Case I may be reduced optionally from $3^1/_2$ in. to 3 in.

Case III:

1. Enter Table 10-3 under welds A and B; select the available strength that 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 7, "Clearance for Welding.") The recommended minimum length is half the *T*-dimension. If several selections can be made from Table 10-3 that satisfy the requirements for available weld strength and angle length, the following criteria should be considered: 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 these two criteria are satisfied, using the shorter length of connection angles and lesser length of weld generally is advisable, although, if the same connection will be duplicated extensively, making a cost study and evaluating all factors is suggested.
2. Weld A:
   Compare available strength with the required reaction. Note the minimum thickness of beam web required and reduce the available strength of the weld as previously explained if the thickness of the beam web is less than the minimum. If the available weld strength is inadequate, select a longer length angle and recheck the available strength of the longer weld.
3. Weld B:
   Compare available strength with the required reaction. Investigate the available 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 available weld strength, select a longer length angle and recheck the available strength of the longer weld.

   Use 3×3 angles for lengths up to, but excluding, 18 in. and 4×3 angles (4-in. leg outstanding) 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 design loads up to the limits of the available strengths, these require a minimum of shop and field welding. Available strengths for various thicknesses of welded unstiffened seated angles are given in Table 10-6 of the *Manual* Part 10 for beams with steel of $F_y = 50$ 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 either $3^1/_2$ in. or 4 in. wide.

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. Available strengths are based on the use of E70XX electrodes. Other required checks are described in Parts 9 and 10 of the *Manual*.

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 (AISC *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 from the supporting member are resisted along four shear planes (refer to Figure 10-9 in the *Manual* Part 10). Reduction of the tabulated available weld 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 effect on the web. This is demonstrated in the text and supporting calculations provided in the *Manual* Part 10, *All-Welded Unstiffened Seated Connections.* Likewise, connection strength is rarely limited by the thickness of the supporting member. This is also demonstrated by text and calculations in Part 10 of the *Manual*.

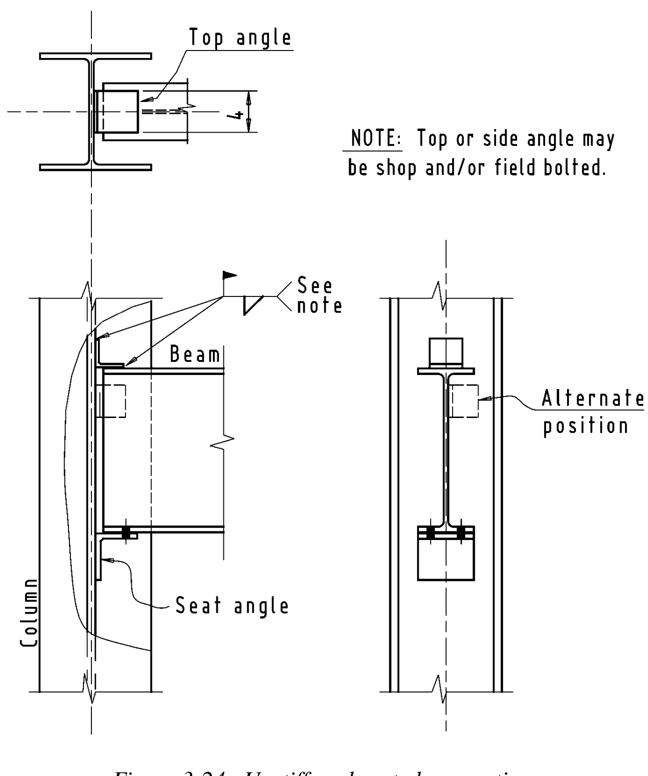


Figure 3-24. Unstiffened seated connection.

Essentially, the length of the seat angle is determined by the required strength and the placement of bolts connecting the beam to it. However, other factors should be considered. The length of the seat angle should be such that the bottom flange of the beam will clear the weld returns on top of the seat angle in case the beam overruns in length. Otherwise, the steel detailer would have to chamfer the end of the bottom flange to clear the fouling weld. Assuming a 5/16-in. weld, each return will be 5/8 in. long. Allowing 3/8 in. for clearance at each side of the flange, the preferred length of seat angle is $2 \times (5/8 + 3/8)$ or 2 in. longer than the width of beam flange. The beam flange must be wide enough to provide holes for bolts. Clipping the corners of the bottom flange at 45° to clear the return welds may be suitable provided sufficient flange remains to accommodate the field bolts.

For any given beam end reaction, several angle and weld size combinations are presented in Table 10-6. Welds larger than 5/16 in. should be avoided where possible, as they require more than one pass by the welder. The range of available seat angle thicknesses shown at the bottom of Table 10-6 will determine the maximum size of weld that can be used.

Examination of Table 10-6 shows that the available strengths for an 8-in.-long seat angle are only slightly greater than for one 6 in. long. This is particularly true for seats supporting beams with relatively thin webs. Similarly, available strength values for seats slightly longer than 8 in. or shorter than 6 in. will not differ appreciably from the values in Table 10-6.

Ordinarily, calculating the available strengths for odd length seats would not materially increase the strength of the connection, as this strength is limited by the web thickness of the supported beam. Where angles must be made longer than 8 in., limiting the available strength to that of a corresponding seat 8 in. long will be conservative. Where a seat shorter than 6 in. is required, the available strength may be estimated conservatively by dividing the tabular value for a 6-in. seat by 6 and multiplying the result by the desired length in inches.

The size of the outstanding leg of an unstiffened seat supporting a channel usually is made 6 in. instead of 4 in. to permit placing two field bolts in the channel flange (which has only one gage line). All other features of the connection remain unchanged.

A 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. Generally, a $4 \times 4 \times 1/4$ angle field-welded along the toe of each leg is used. This permits the end of the beam to rotate slightly as it deflects under applied loading without appreciable restraint from the top angle. Additional welding should not be done unless called for by the designer because it may stiffen the beam and alter the behavior of the structure by introducing unintentional moment restraint.

The thickness of the top angle is sometimes controlled by the weld size required by AISC *Specification* Section J2.2b. If the thickness of the supporting member or seated beam flange requires a weld size greater than $^3/_{16}$ in., the top angle must be made thicker than $^1/_4$ in. Any combination of the weld size and material thickness for the seat angle, top angle and connected members must meet the requirements of Section J2.2b of the AISC *Specification*.

If welds are placed on the back side of an angle connecting to a column flange, weld returns should be omitted on top of the seat angle, as shown in Figure 3-25. While the AISC *Specification* makes specific reference to end returns of fillet welds in Section J2.2b, it states also that fillet welds on opposite sides of the same plane shall be interrupted at the corner common to both welds. Figure 3-25 indicates examples of this exception where the welds, while in the same plane, lie on opposite sides of the same contact surfaces. Any attempt to weld around the corner will melt the corner material to create a reduced thickness (notch) and under no circumstances can such a weld be made with a full effective throat. In this situation, the angle will be extended beyond the column flange only enough to provide space for welding. The space (or "shelf") for welding should be based on the values given in Figure 7-84 of this manual.

Figure 3-26 illustrates two additional situations where wrapping or returning the weld around the end of material creates a reduction in thickness and a notch that can create large stress concentrations. These conditions should be avoided from a structural design consideration.

For field bolting beams to shop-welded seat angles, the type of field bolt required is governed by AISC *Specification* Section J1.10. Field bolting beams to seats complies with the Occupational Safety and Health Administration (OSHA) mandate requiring two bolts to secure the beam to the seat. Please see the section on OSHA requirements in Chapter 2 of this text for further detail on these requirements.

Stiffened Seated Connections

Stiffened seated beam connections are used when a beam end reaction exceeds the design strengths given in the *Manual*, Table 10-6. These connections are used, frequently, for beams framed to the webs of columns. Also, they are used for beams framed to column flanges if the stiffener will not interfere with fireproofing or architectural finishes. They are made, normally, by welding two plates to form a tee, but also can be made by cutting a tee-shaped fitting from a suitable rolled shape.

Available strengths of stiffened seats are given in Table 10-8 in the *Manual* Part 10. Note that the design strengths of these seats decreases (for any given length of stiffener and weld size) as the width of seat increases. This is due to the increase in eccentricity of loading on the welds, which results as the seat and stiffener are widened. In the interest of economy, the narrowest listed seat width providing the required bearing should be used.

The available strengths in Table 10-8 are calculated for welds deposited along the full length of the stiffener plate and returned a minimum distance of 0.2 times the stiffener length under each side of the seat plate against the column web (refer to Figure 3-27). Where the width of the seat plate is insufficient to attain the $0.2 \times L$ length of welds required, it is satisfactory to weld the full length of the seat plate. When two plates are used to form a seated connection, the stiffener must be finished to bear on the seat plate and the strength of

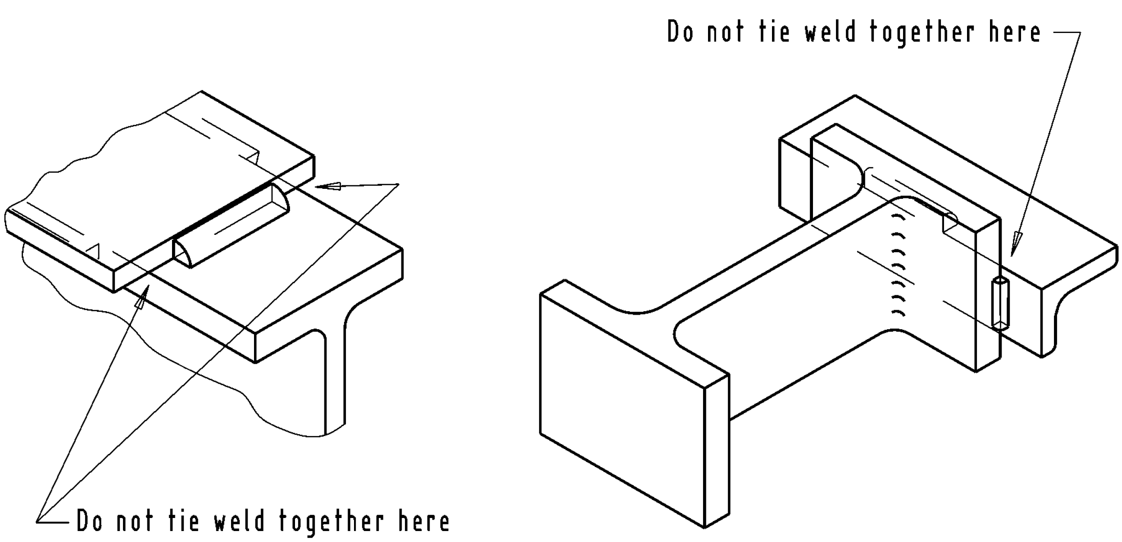


Figure 3-25. Seat angle weld arrangement.

the welds joining the stiffener and seat must be equal to or greater than the horizontal weld returns.

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

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

but not less than $2w$ for stiffeners with $F_y = 36$ ksi nor $1.5w$ for stiffeners with $F_y = 50$ ksi. In the preceding equation, t_w 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, or 1.4 times the beam web thickness. Additionally, the minimum stiffener thickness *(t)* should be at least $2w$ for stiffener material with $F_y = 36$ ksi, 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 of these cutting lines will save material provided that the plates are nested or multipled (see Chapter 1).

Shear End-Plate Connections

Shear 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 available 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 $^3/_4 \times t_p$, where t_p is the thickness of the plate. This ensures 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).

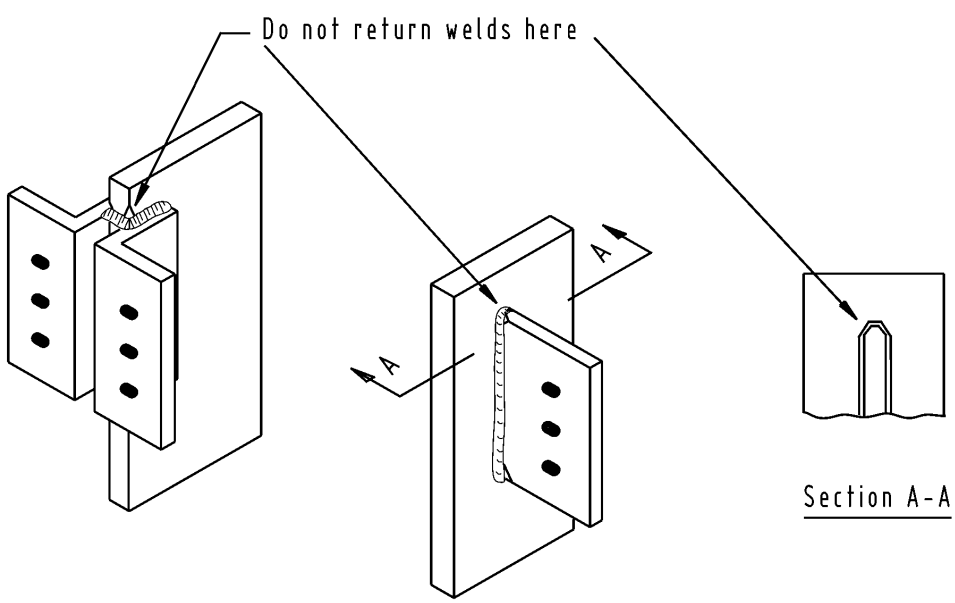


Figure 3-26. Reduction in thickness from wrapping or returning a weld around the end of a material.

Single-Angle Connections

Table 10-11 in the *Manual* Part 10 presents available 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 AISC *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 AISC *Specification* Section 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 J1.8 and J1.10. 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 3 to determine the reactions and to Part 10 to select the connection. Some designers may

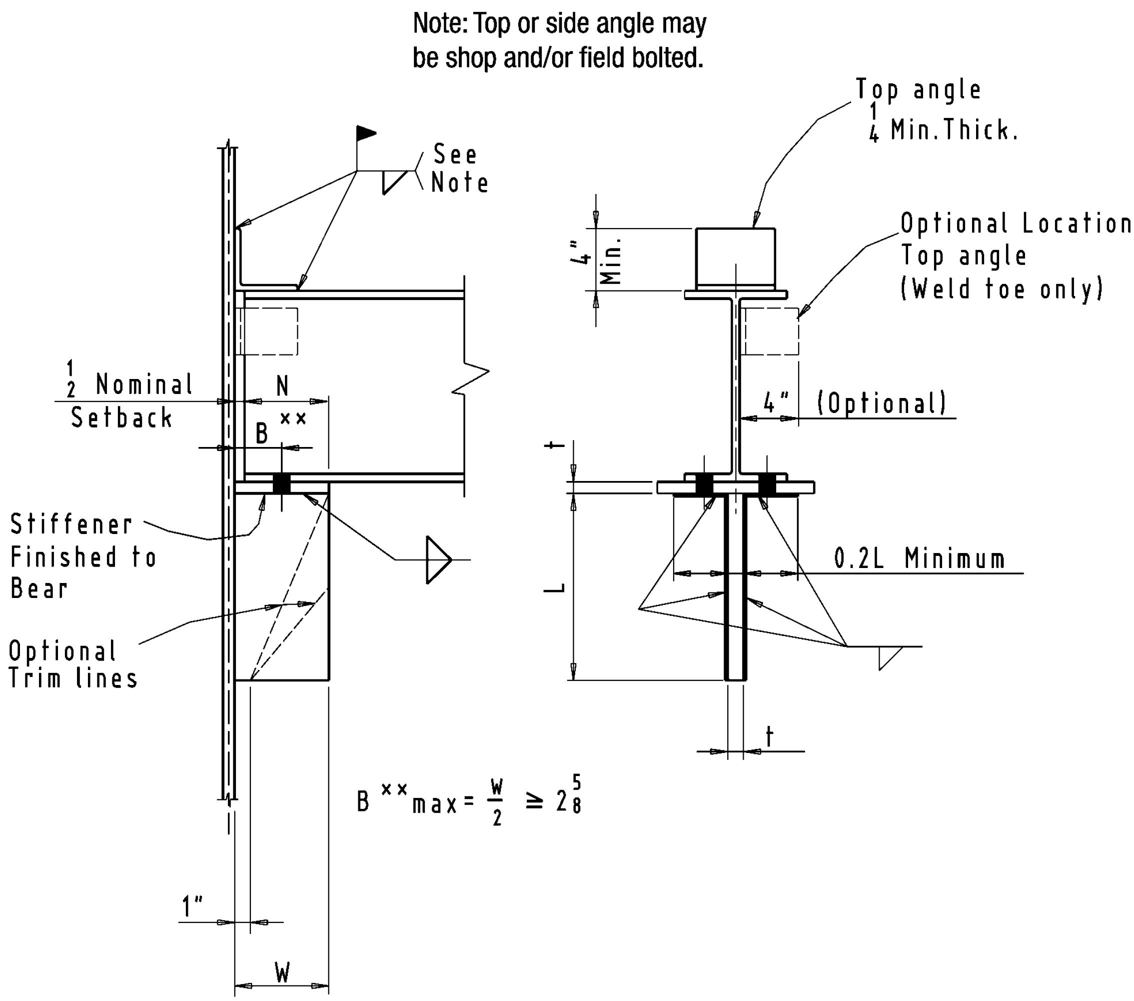


Figure 3-27. Stiffened seated connection.

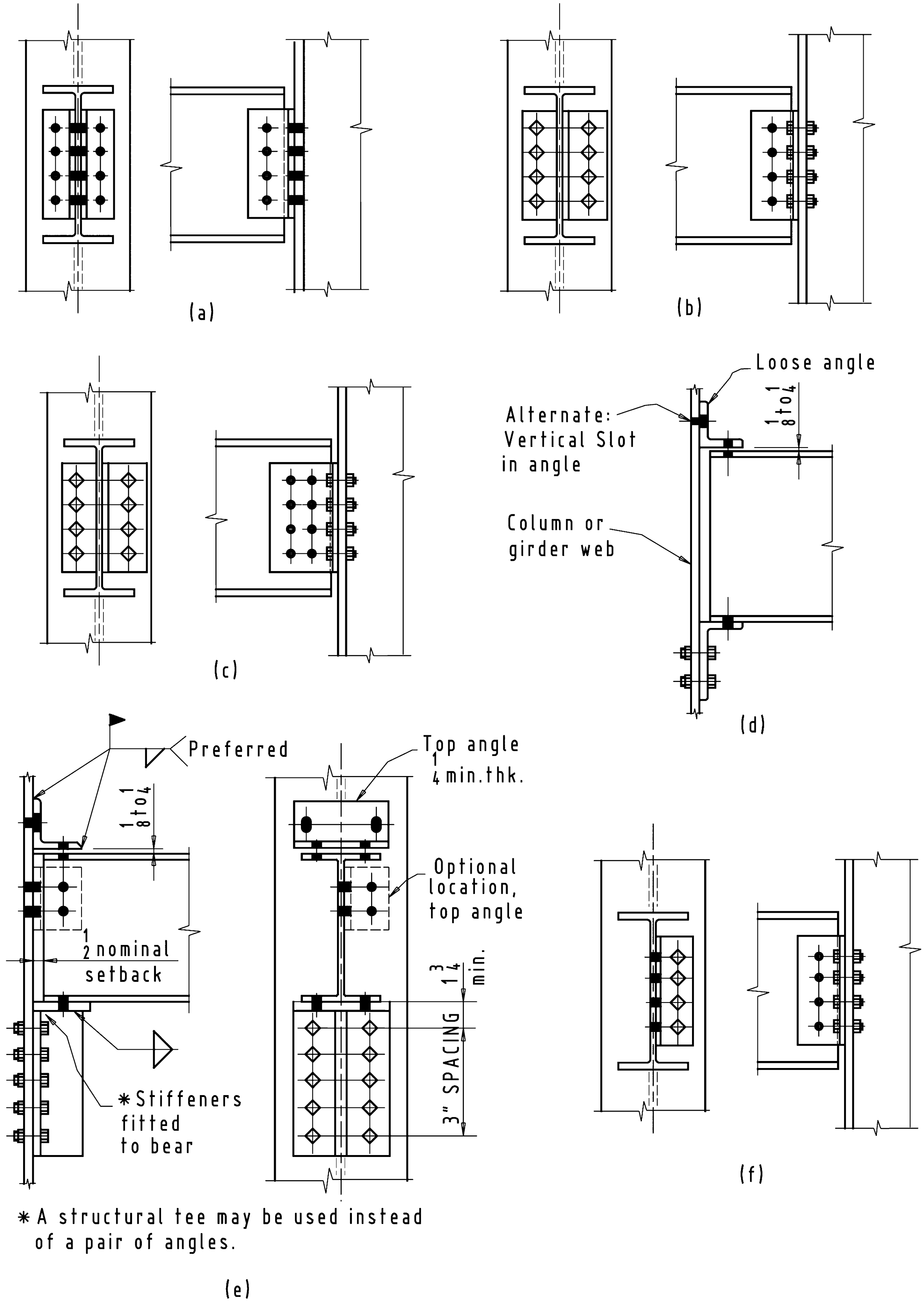


Figure 3-28. Framed and seated connections for shop-bolted construction.

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 connections can be used on column flanges, although the projection of stiffeners through architectural finish or column fireproofing may limit the choice to unstiffened seats. Note in both cases the preferred connection of the top angle is field welded to the column and beam. If field bolting is required, vertical slots may be furnished in the top angle to provide adjustment.

Framed Connections

Shop-welded, field-bolted beam connections suitable for shop welded-field bolted construction are shown in Figure 3-29. Paired angles, shop welded to the column flange, are shown in Figure 3-29a. Note that this is a knife connection, which requires a beam web erection clearance and a bottom flange cope. Erection clearances vary from $^1/_8$ in. to $^1/_{16}$ in., depending on the beam size. The angle legs are drawn together during the bolt tightening process. Figure 3-29b shows an end-plate welded to the beam. Its advantage of simplicity is offset, somewhat, by the close tolerance needed to achieve accurate beam length and square ends. The use of this connection for several beams in a continuous run may require a small reduction in beam length and provision of shims to compensate for overrun or underrun in column dimensions.

Figure 3-29c shows another very simple connection that utilizes a plate shop welded to the column flange. It is somewhat limited in shear strength, but may be used when the connections are designed in accordance with the procedure outlined in the *Manual* Part 10. Figure 3-29d illustrates the use of a tee shop welded to the column with holes for field bolts. The flange thickness of tees used for this purpose should be held to a minimum (as noted earlier in this chapter) to permit the flexure due to the end rotation of the beam. Figure 3-29e shows the connection angles shop welded to the beam web and field bolted to the column flange. This type of connection has the same disadvantage of required close tolerances as the end-plate connection in Figure 3-29b. Figure 3-29f shows a single angle shop welded to the column flange and field bolted to the beam web.

SEATED CONNECTIONS

Shop Welded, Field Bolted

Shop-welded, field-bolted seated connections are similar to the all-welded seated connections in Figures 3-30b and 3-30c, except that the top angles may be bolted as in Figures 3-28d and 3-28e. Refer to earlier sections of this chapter for additional information.

Framed and Seated Connections

All-welded framed connections are adapted readily from the four types shown in Figure 3-29. Figure 3-30a shows a tee connection comparable to the one in Figure 3-29d; the other connections of Figure 3-29 can be converted in a similar manner. The seated connections in Figures 3-30b and 3-30c can be used as connections to either the column flange or web, their applications being subject to the same restrictions noted for the bolted versions. The stiffened seat, shown as a structural tee, may be replaced by one built from plates.

Framed and Seated Connections—Field Clearances

Figure 3-28 shows clearances between the top connection material and the top of beam. The recommended $^1/_8$- to $^1/_4$-in. clearance applies to connections in Figures 3-28d and 3-28e. This clearance is necessary to allow for possible mill variations from the published beam depths and out of square flanges. These details are generally used on connections to column webs.

Three typical situations and the clearances usually provided are shown in Figure 3-31. The range of clearances given reflects the shop practice of some fabricators.

Figure 3-31a shows a seated connection where the top angle is field bolted. When field bolts are used to fasten beams to seated connections, the need for clearances and shims can be eliminated by providing vertical slots in the top angle.

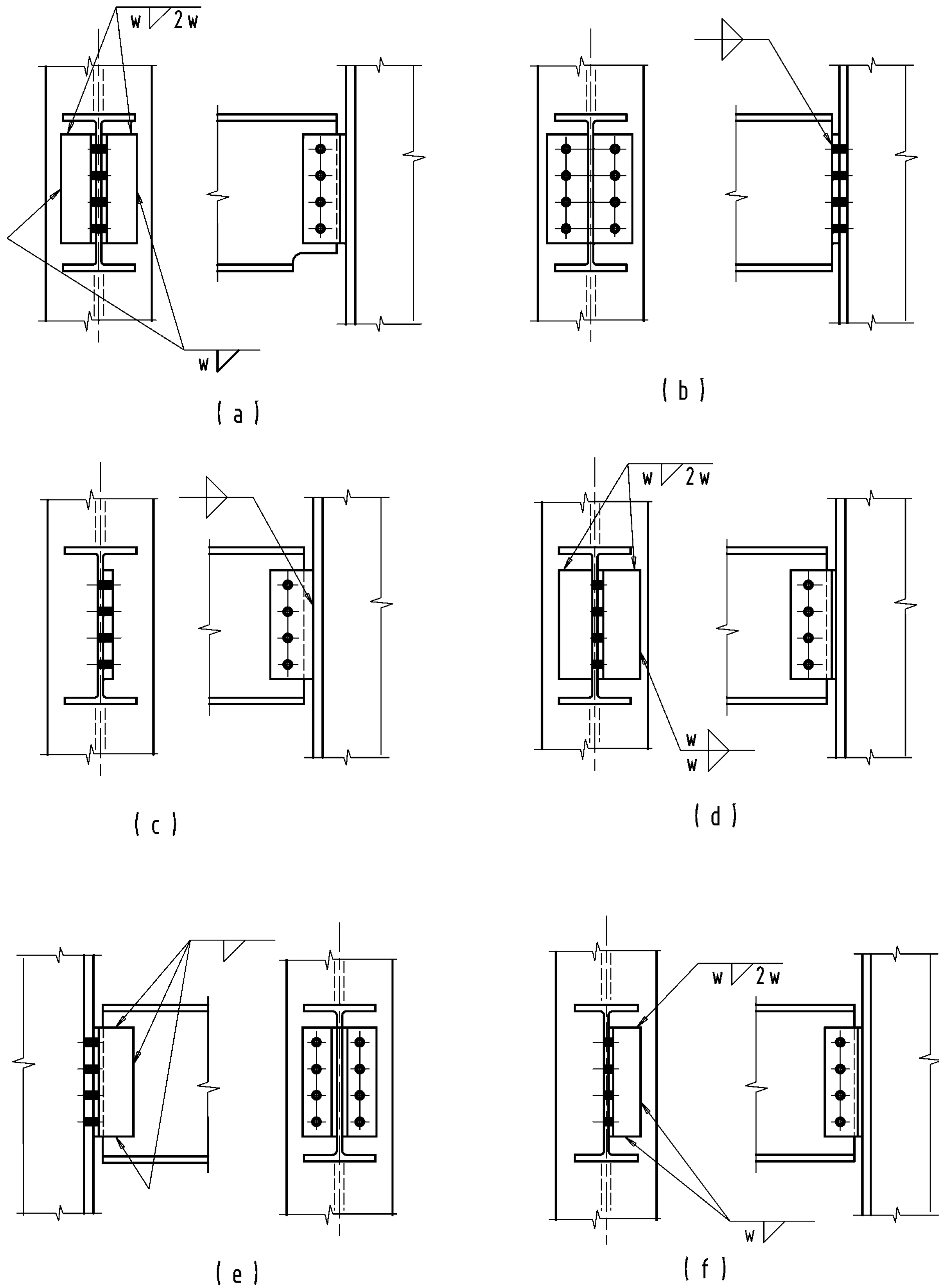


Figure 3-29. Shop-welded, field-bolted beam connections suitable for shop-welded, field-bolted construction.

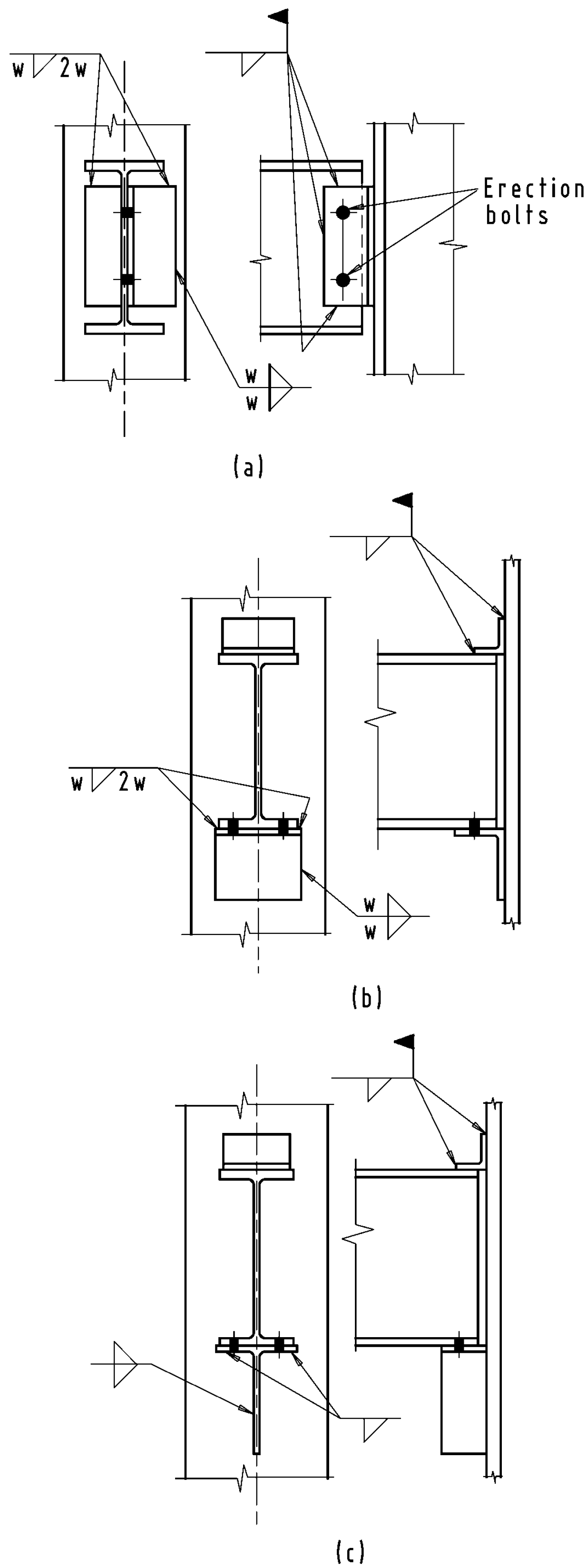


Figure 3-30. All-welded framed connections.

Such slots are located with their centers matching the holes in the column. This will accommodate both overrun and underrun in the beam depth and is the preferred method of many fabricators.

The 1/4- to 3/8-in. clearance shown in Figure 3-31b is suggested when the top connection is either shop bolted or shop welded. This detail may be used on column flanges.

Providing clearance at the bottom as well as at the top, as shown in Figure 3-31c, is optional when a beam web connection is included with shop attached top and seat material.

In each of these cases, because beam depth may underrun as well as overrun, some fabricators supply shims for about twice the opening expected under the top angle. Others supply shims for openings as detailed and furnish additional shims only as required.

When beam web connections are placed on column flanges with one angle bolted, the spread between outstanding legs should equal the decimal beam web thickness plus a clearance that will produce an opening to the next higher 1/8 in. This rule is illustrated in Figure 3-32a. Note that the angle gages will occur in minimum increments of 1/16 in.

In the event that both angles are shop attached to the column, forming a knife connection, provisions should be made for an erection clearance of about 1/16 in. The application of this rule is shown in Figure 3-32b. When similar fittings for a knife connection are welded to a column and angle gages are not a factor, providing an opening equal to the fractional beam web thickness plus 1/16 in. is satisfactory. When these connections are to be made with high-strength bolts, shims must be furnished wherever measured clearances exceed 1/8 in.

Example 1

Several cambered filler beams are to be connected to supporting girders in a series of bays. The beams are horizontal and perpendicular to the girders. Five common simple types of connections are available from which to select—double angle, single angle, single plate, tee and end plate. The reaction given on the design drawing is pure gravity load and well within the strength limits of all five types. First, consider the double-angle connection (Figure 3-32). The use of double angles, even with the minimum number of rows, frequently results in a connection having a strength limit much in excess of the design reaction. Also, the erection clearance employed with this type of connection (a minimum of 1/16-in. setback at one end in each bay) requires that for every fourth or fifth bay, shims may be installed to maintain the overall building geometry. For this reason, the double-angle connection was not selected.

Next, the end-plate connection was considered (Figure 3-8). This connection has the same detriments as the double-angle connection regarding the shimming requirements. Nevertheless, the main objection to end-plate connections is the zero tolerance on the cut length of the beam and the fit-up.

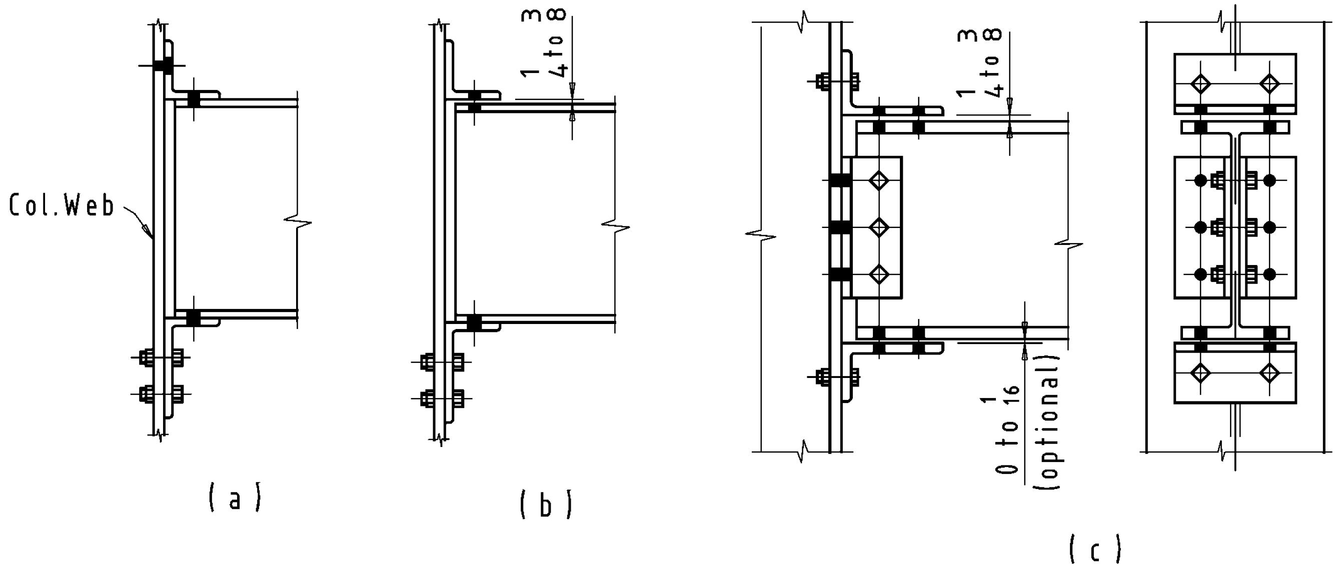


Figure 3-31. Typical seated connections and the clearances provided.

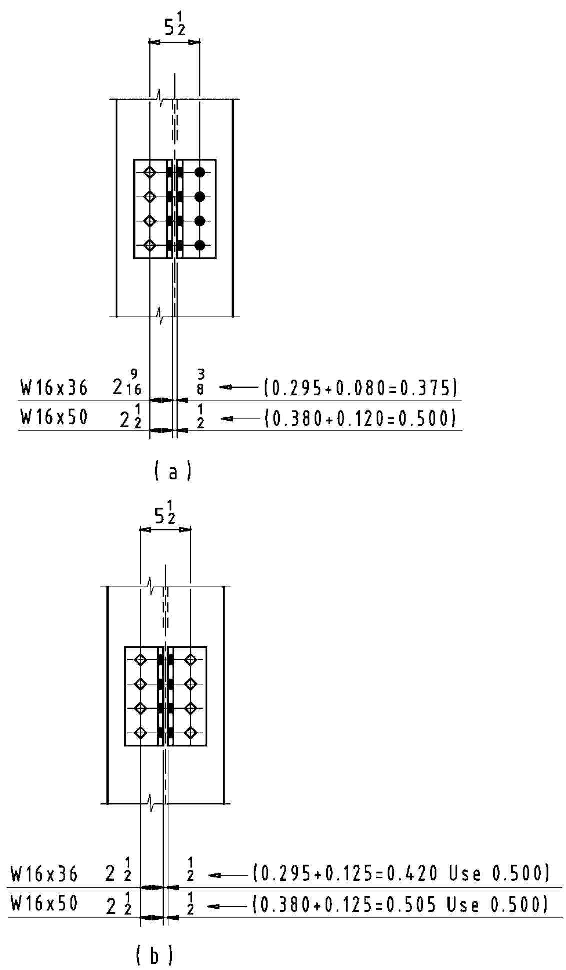


Figure 3-32. Beam web connections placed on column flanges.

The beams must be saw cut on both ends to exact length. In most shops, this is an additional operation not required by other types of connections. Mainly for economic reasons, this connection is rejected.

Next, consider a single-angle connection, with the angle shop bolted to the supporting girder (Figure 3-14). The field connection is made by bolting the web of the filler beam directly to the outstanding leg of the angle. The angle is punched with horizontal short slots, thus negating the need for the objectionable shims associated with the previously considered types of connections. In addition, the filler beams are cambered and the resulting beam end rotation is handled easily by the slots, thus enabling the fabricator to punch or drill the end holes in the filler beams prior to cambering, which is a desirable job procedure. This connection has distinct advantages over the previous considerations. It is safe to erect. The angle was shop bolted to the supporting girders because other bolting was required on the girders. Otherwise, the angle could have been welded.

Next to be considered is a tee connection shop bolted to the connecting girder (see Figure 3-28c). The stem of the tee is punched with horizontal short slots in order to provide fabrication and erection tolerance. The cost of erection is about the same when using the tee as it is for the single angle, but the cost of shop work on the tee is higher. Therefore, for economic reasons, the tee connection will be deleted from the list of choices.

Finally, consider the single-plate connection (Figure 3-12). Many shops are equipped to handle either bolted or welded work. The fabricator equipped primarily to handle bolted fabrication, yet having the capacity to perform some welding, may want to avoid this connection, especially if this type of connection is to be fabricated in large quantities. Otherwise, it is a valid contender. The plate is punched with horizontal short slots for the same reasons as previously mentioned. Shop costs for this connection are a bit higher than for the single-angle connection because of fitting and the need to keep the plate perpendicular to the girder web. (For skewed and/or sloped members the welded single-plate has distinct advantages over the other types). Of the five candidate connections the choice narrows to two—the single angle and single plate. If the fabricator prefers bolting, the single angle is the choice. If welding is preferred, the single plate is the choice.

Example 2

An interior girder beam is to be connected to a column web. The gravity load is moderate and a small axial load, which is shown on the design plan and acts in a direction parallel to the longitudinal axis of the beam, is induced in the beam to brace the column. Two choices are available—a double-angle connection and a stiffened seat connection. The double-angle connection is popular with some fabricators and erectors (Figure 3-32). However, beams framing opposite each other and sharing common holes through the column create a more difficult erection situation. To avoid this, either double stagger the connection angles if the beam depth permits (angles high on one side of the column web and low on the other side) or provide erection seats below the first beam to be erected (or both beams) to support the beam safely while the second beam is being lowered into place. The owner's designated representative for design should determine the size of the connection angles required to resist the axial load. A shim(s) between the angle leg and column web may be necessary to make up the proper bay length.

The stiffened seat connection connecting girder beams to column webs is illustrated in Figures 3-11 and 3-27. It provides a shelf on which to land a beam during erection. It can handle moderate loads with ease. Here, too, the owner's designated representative for design should show on the design drawings the type of connection required to resist the axial load. The top angle can be either field welded or bolted. The choice in this instance is a stiffened seated connection. As in Example 1, the decision to use a bolted or welded connection to the column web depends on the preference of the fabricator.

Example 3

A horizontal floor beam must be connected perpendicular to a column flange. The beam supports a modest gravity load but no axial load. The floor beam is cambered. Six choices of connections are available: seated, double angle, single angle, single plate, tee and end plate. The double-angle connection can be shop attached to either the beam or the column using either welding or bolting. A double-angle connection that is shop attached to the column is referred to as a "knife" connection because the beam web, as it is being erected, is knifed between the angles (Figure 3-33a). Even though this method requires the beam to be coped at its bottom flange, the benefits far outweigh this extra work. (Note that for a fabricator utilizing a beam line to fabricate beams, an all-bolted double-angle connection probably would be the most economical connection.) Assuming the angles will be welded to the face of the flange, punching or drilling holes in the flange is not required. Erection is safe and quick. The connection uses half as many field bolts (compared to field bolting double angles to a column flange) and no bolt heads interfere with other members in the throat of the column. Also, by using short slots, the beam end rotation caused by cambering can be accommodated. In this way the bay length is established without subsequent measuring and shimming and remeasuring. The alternate to the knife connection is to bolt or weld the connection angles to the beam web in the shop (Figure 3-33b). Shimming at the column is required occasionally because the beam may need to be shortened slightly to accommodate anticipated column overrun. The foreshortening prevents the building from "swelling," a problem on multi-bay structures. The beam camber and resulting beam end rotation may complicate erection. Despite potential erection problems, some fabricators and erectors prefer this version of the double-angle connection.

Consider next the single-angle connection shop attached to the column (Figure 3-34). Basically, this will be the same single angle as discussed in Example 1. Horizontal short slots in the outstanding leg of the angle will accommodate the beam end rotation due to camber. It has all of the other advantages of the knife connection just discussed. If the connection requires upgrading in the future, this can be done readily by adding a second angle to make it a double-angle connection. The angle should be welded or bolted to the column flange in the shop. Generally, bolting is suitable only for light columns with a small k so that the gage on the angle can be kept to a minimum ($2^1/_4$ in. or less preferred) in order to reduce eccentricity on the bolt group. Welding eliminates drilling the column flange, which is an important cost consideration for columns with thick flanges.

The end-plate connection (see Figure 3-8) has all the disadvantages previously discussed in Example 1 and can be eliminated from consideration immediately.

The seated connection is not a common choice for connecting a beam to a column flange, but it can be a wise solution given certain parameters. This type of connection would be a good choice if torsion forces were present. Both the stiffened (Figures 3-11 and 3-27) and unstiffened (Figures 3-10 and 3-24) versions are valid choices. However, if the stiff-

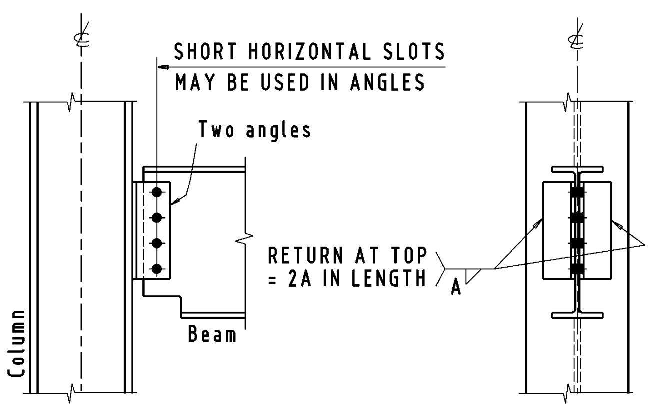


Figure 3-33a. Knife connection.

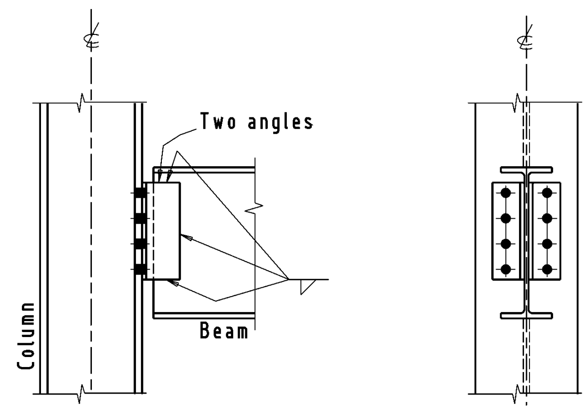


Figure 3-33b. Shop-bolted or shop-welded connection angles to the beam web.

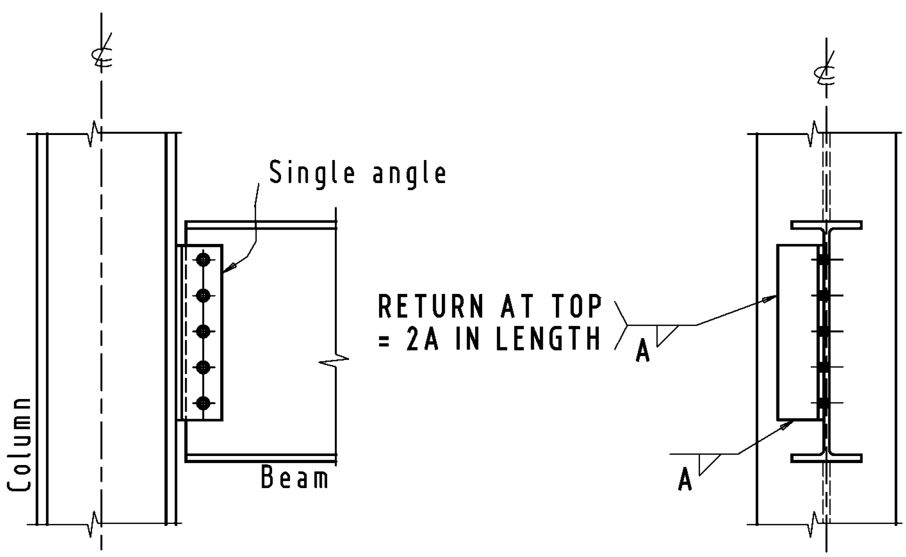


Figure 3-34. Single-angle connection shop attached to the column.

ened version is selected, care must be exercised to ensure that the seat stiffener does not protrude through the fireproofing or other column covering below the ceiling line. In the case at hand, an unstiffened seat angle would suffice. Seated connections can tolerate some axial load, although that is not a requirement here. As no compelling reason can be stated to use seated connections at the column flange, this type of connection will be removed from consideration.

The tee connection (Figure 3-15) is a viable candidate, and its merits and drawbacks have been discussed in Example 1. Nevertheless, for economic reasons it is eliminated from further consideration.

The single plate is a strong contender for connecting a beam to a column flange (Figure 3-13), especially in a fabricating shop that prefers welding as opposed to punching and bolting. Often, single plates (and single angles) are used to resist the gravity load of moment connected beams. The single plate is welded to the column flange and has horizontal short slots. Here, too, the slots will accommodate the camber rotation previously discussed.

The choice of connections has been narrowed from 6 to 4 types: knife-type double angle, double angles shop attached to the beam web, single angle, and single plate. If the gravity load is beyond the design strength of the single plate or single angle, either of the types of connections using double angles is suitable, the choice of which type depends upon the preferences of the fabricator and erector. Otherwise, either the single-angle or the single-plate connection can be used, welded to the column flange.

Offset and Skewed Connections

Special care is required when developing connections for framing that is not centered on column web or flange faces or is skewed with respect to the column. Design drawings show such offset or skewed beams by dimensions referred to column centers, but seldom show details of connecting such framing.

Small offsets in bolted construction can be handled readily by shifting holes in seat angles, by altering gages in framing angles, or by otherwise modifying regular connection material. Larger offsets require special treatment, using such details as one-sided connections, gusset plates or built-up brackets. Consideration should be given to the eccentric loading of fastener groups resulting from these offset connections. Small eccentricities in double-angle or seated connections may be neglected. Where appreciable eccentricities occur and for all one-sided framed connections, fastener strengths must be investigated. This can be done by using the coefficients given for single-plate and single-angle connections in the *Manual* Part 10. Skewed, sloped and canted connections that present similar problems can be found in the *Manual* Part 10, which contains an extended discussion on eccentric conditions, including examples.

Skewed structural framing can connect to a column in a variety of ways.[2] For instance, in addition to the several types of connections already presented, beam webs can be connected directly to the inner or outer faces of column flanges, using intervening fillers if required. No general rule can be cited for all cases. The best connection is usually the one requiring the least number of fasteners or welds and the least amount of connection material. Figure 3-35 displays a few of the many ways nonconcentric and nonrectangular framing may be supported by columns. Such connections are generally worked out on preliminary drawings by an experienced steel detailer in advance of the detailing and checking.

Moment Connections

In tier building construction, many beams and girders connecting to columns are designed to resist bending moments resulting from lateral forces due to wind or earthquake loadings. As these loads are in addition to normal gravity floor loads, the beam connections are proportioned for moment as well as shear forces. These connections are usually indicated on the design drawings by a system of numbers or symbols, keyed to schedules and design drawings, which give the designer's requirements. Field-bolted moment connections are made with angles, plates or tees, which transmit the moment from the beams to the columns. The vertical shear is transmitted by angle or tee connections on the beam web or by a stiffened or unstiffened seat. Field-welded moment connections may employ plates to transmit both shear and moment forces or, where a rigid or continuous frame design is called for, the beams may be welded directly to the column. Where plates are used, they are shop welded to the column and field bolted to the top and bottom flanges and to the web of the beam (see Figures 3-36a and 3-36b).

The owner's designated representative for design is expected to show complete data and detail drawings of desired connections where Type PR (partially restrained moment) or FR (fully restrained moment) framing is to be used in the design. The design of moment connections is not a part of the steel detailer's work. For examples and illustrations of moment connection designs, the steel detailer is referred to the *Manual* Part 11 for Type PR and to Part 12 for Type FR.

Shop-welded, field-bolted moment connections are shown in Figure 3-36a connecting to a column flange and in Figure 3-36b connecting to a column web. As the moment is resisted by the top and bottom plates, deflection of the beam at the connection is minimized, so that provision for end rotation

[2] 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.

of the beam is not necessary. The web connection in Figure 3-36b is so arranged that all fasteners are accessible for impact wrench tightening. The locations of shop attached fittings on the column must allow for rolling mill tolerances, such as beam depth overrun and out-of-square flanges. Also, the space provided between the shop attached plates must be sufficient to permit swinging the beam into its final position.

Figure 3-36c shows an end-plate capable of transmitting both shear and moment. Fabricaton and erection advantages and limitations of this connection are the same as those cited for the end-plate connection in Figure 3-29b. Design procedures for extended end-plates are discussed in Part 12 of the *Manual* and in AISC Steel Design Guide 4, *Extended End-Plate Moment Connections* (Murray and Sumner, 2004). Information on flush and extended multiple-row moment end-plate connections can be found in AISC Steel Design Guide 16, *Flush and Extended Multiple-Row Moment End-Pate Connections* (Murray and Shoemaker, 2002). As the eight bolts required for the connection (four at the top flange and four at the bottom flange) are adequate to resist all applied forces, the distance between the bolts at the top flange and those at the bottom flange should be considered. In this illustration additional bolts were necessary, consistent with AISC *Specification* Section J3.5.

All-welded moment connections generally require fabrication that is more exacting than fabrication of shop-welded, field-bolted moment connections. Figures 3-37a and 3-37b show two ways of making an all-welded moment connection to a column flange. The method in Figure 3-37a requires a close joint between beam web and column flange, which may dictate finishing the beam to close tolerances. In Figure 3-37b this problem is eliminated, but at the cost of more welding and an additional plate for the shear connection. The holes in the web connection are for erection

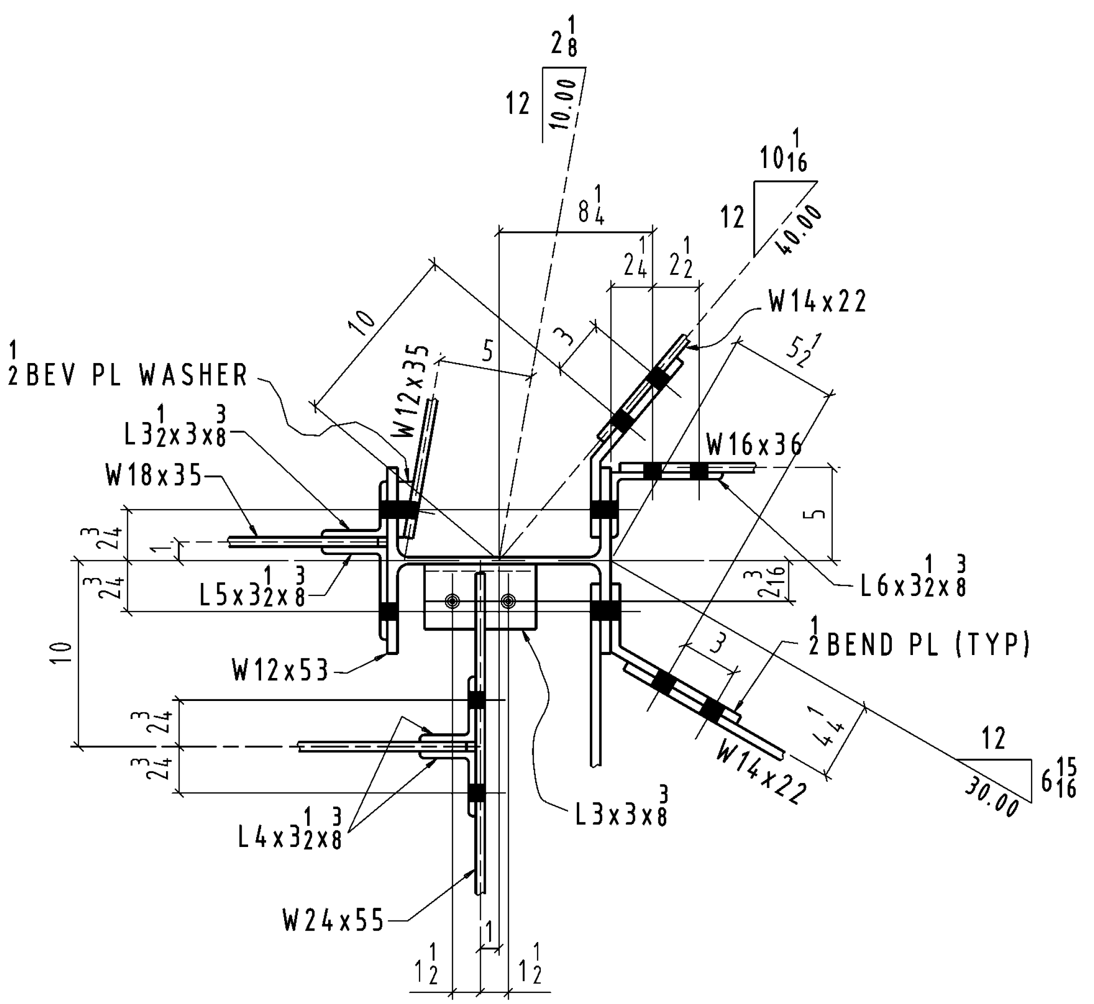


Figure 3-35. Ways in which nonconcentric and nonrectangular framing may be supported by columns.

bolts to hold the beam in position until field welding of the connection is completed. Figure 3-37d is a variation of Figure 3-37b in that the beam web is connected to the single plate with field bolts instead of field welds. On lighter column sections, any of these connections may require stiffener plates between the inside faces of the column flanges. These plates distribute the moment forces from the connections to the column web or to a similar connection on the opposite face of the column.

Although the details in Figures 3-37a and 3-37b appear suitable for a column web connection, the limited space between column flanges may make welding difficult. The all-welded moment connection illustrated in Figure 3-37c may be used for column web connections. Note that this is, in effect, a beam stub, extending just beyond the column flange so that field welding can be performed easily. Groove welds and backing bars are used to obtain complete-joint-penetration groove welds in the beam flange joints. The shear joints are made with fillet welds. A common variation of this joint employs the use of high-strength bolts instead of field welding to connect the beam web to the stub plate.

The steel detailer is cautioned that connections to W-shape column flanges, as shown in Figures 3-37a, 3-37b and 3-37d, may not perform well on cover-plated columns and, if so shown on a design, should be questioned. The force exerted by whichever of the moment plates is in tension will tend to separate the cover plate from the column flange. If cover plates are necessary, the designs must show the manner in which the cover plates are to be attached to the W-shape in the area of the moment connection, so that the beam-column continuity force can be developed.

In detailing beams as shown in Figure 3-37b, compensation for field weld shrinkage may be necessary. Experience has shown that this is about $^1/_{16}$ in., perpendicular to the weld throat, for the typical groove weld. Shrinkage of this

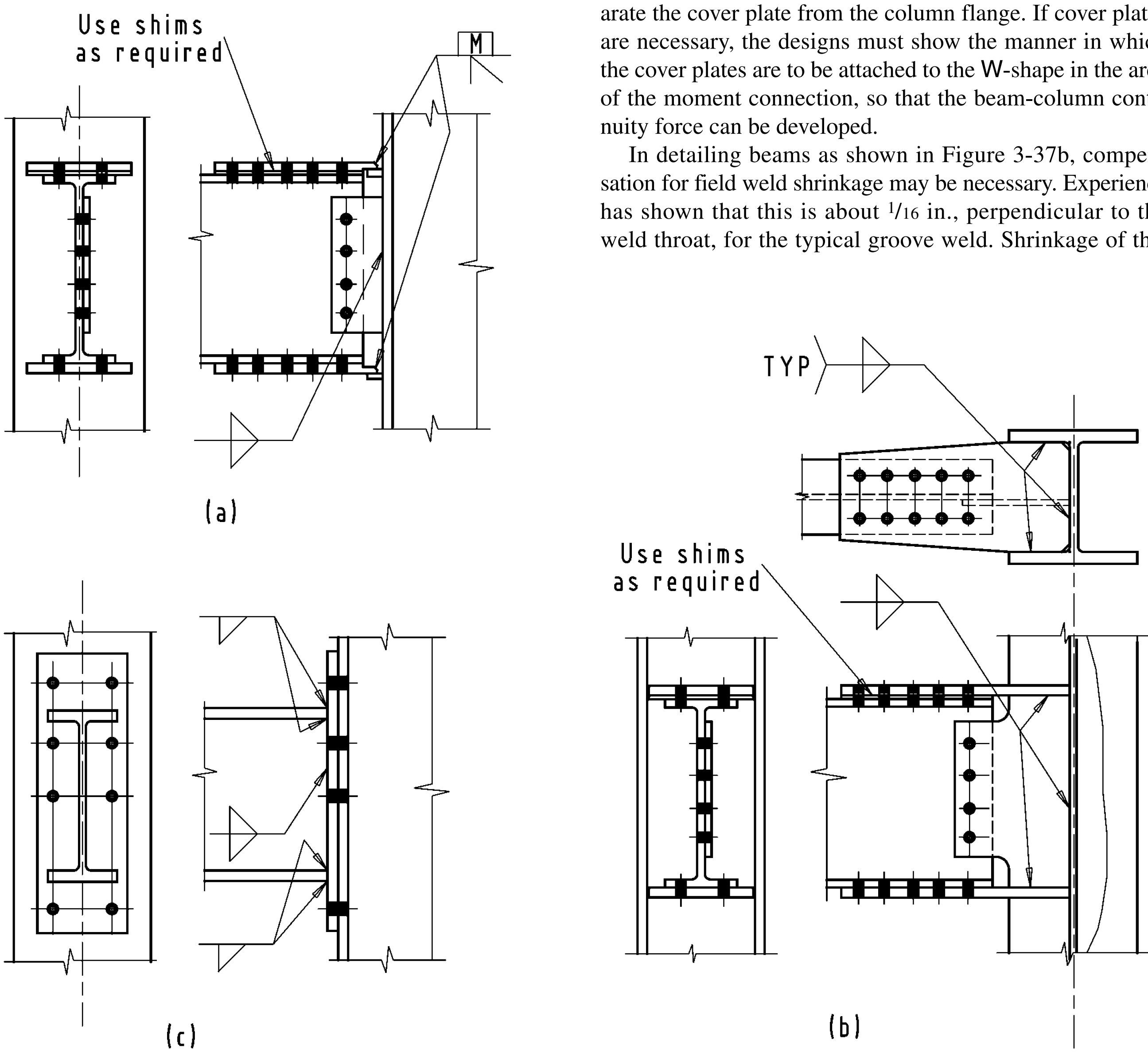


Figure 3-36. Shop-welded, field-bolted moment connections.

kind can cause problems in erection, particularly where several beams and columns occur in a continuous run. Corrections can be made by detailing beams with $^{1}/_{8}$ in. added to the center-to-center dimension of end holes or by providing adjustment slots for erection bolts in the column fittings (see Figure 3-37c).

These all-welded connections approach full rigidity and are generally limited to Type FR construction. Their use in structures designed as Type PR framing should not be undertaken without express approval from the owner's designated representative for design.

COLUMN SPLICES

As the height of a building increases, so does the need for splicing the column sections because of available lengths or, more often, for economy because of the change in loading at the different floor levels.

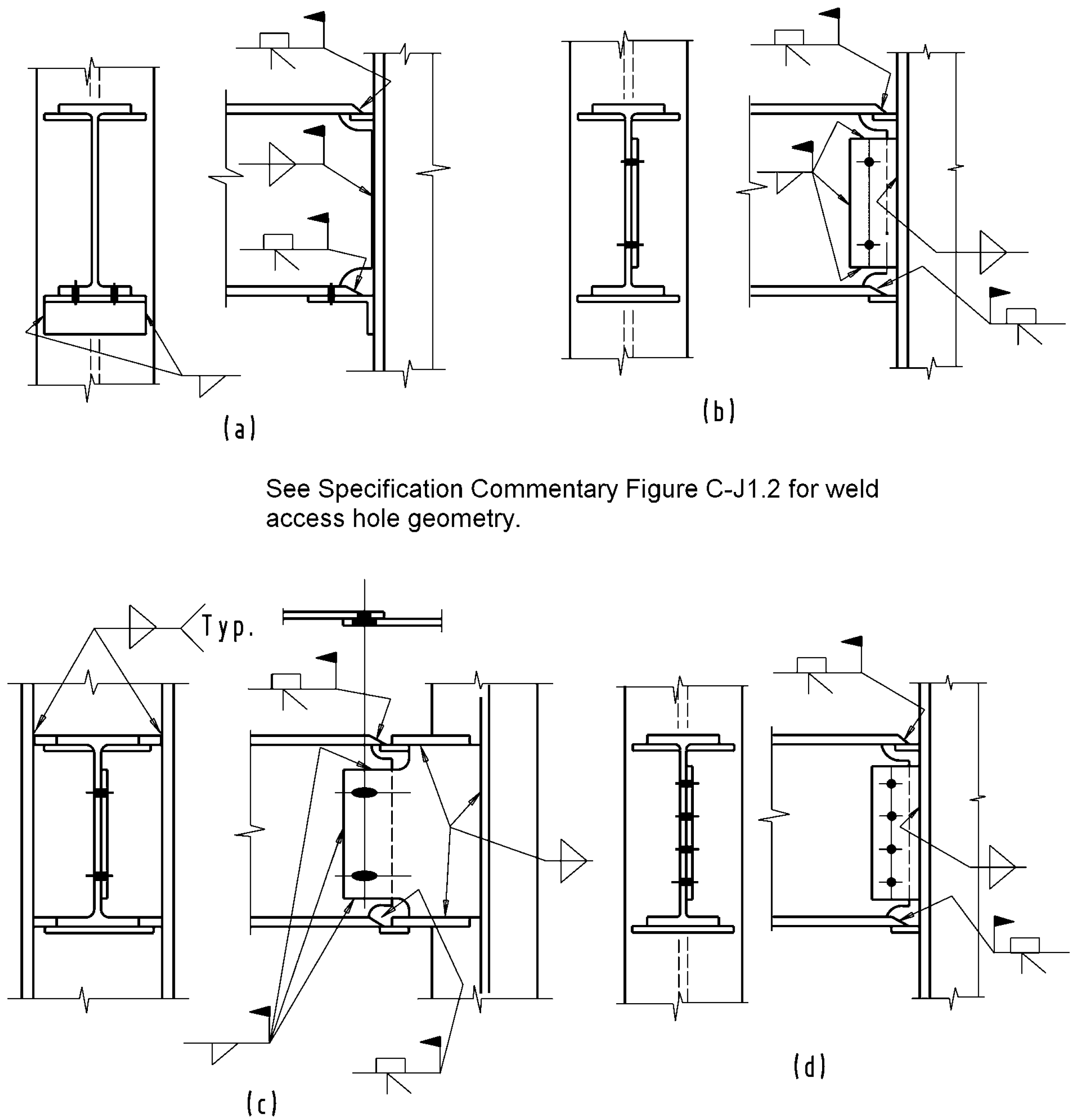


Figure 3-37. All-welded moment connections to column flanges.

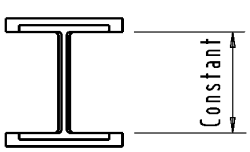
The structural design drawings should furnish sufficient information and detail to indicate the size, number and arrangement of fasteners or welds and the size and type of splice material required by the loading or by the AISC *Specification*. In addition, the erector must furnish any forces resulting from special erection operations. AISC *Specification* Sections J1.4, J1.10, M2.6 and M4.4 form the basis for discussion in this section. The steel detailer should become familiar with these *Specification* Sections before proceeding.

The cross-sections of the W-shapes most frequently used as columns are such that for any given nominal size, the distance between the inner faces of the flanges is constant. As the weight per foot increases in each of the nominal size groups, the column depths and web thicknesses increase. This results in full bearing of the lighter sections when they are centered over the heavier sections. Figure 3-38 shows the relationship, which prevails within the listed weight groups.

Where upper and lower column sections are not centered, or where different nominal sections must bear on each other, some areas of the smaller sections will not be in contact with the larger section. Shaded areas in Figure 3-39 illustrate these conditions. Load transfer may be accomplished by providing fillers under the flange plates (finished to bear on the larger section) or by interposing a butt plate in the joint on which the upper and lower sections of the columns will bear.

The general requirements for the flange-plate-type splice, when upper and lower shafts are not centered, are (1) to provide sufficient area of fillers in bearing to equal substantially that part of the lighter section which is not in contact and (2) to furnish the fillers with enough fasteners or welds to transmit the bearing load into the column shaft. The splices shown in the *Manual* Part 14 have proved satisfactory for column sections differing by up to 2 in. in nominal depth. Joints involving greater differences are special and details should be shown on the design drawings.

Groove welded butt splices are used frequently in welded construction. Edge preparation is made in the shop, usually for partial-joint-penetration bevel or J-welds (see Figure 3-40). The *Manual* Part 14 further details this type of splice and shows the use of butt plates where the upper shaft dimensions result in less than 100% direct bearing. In the absence of flange plates, column shaft alignment and stability during erection are achieved by the addition of lugs for erection bolting, as shown in Figure 3-40 and further developed in the *Manual* Part 14. These lugs are usually temporary, as requirements in the construction documents may dictate their removal after welding is complete.



Column Size	Constant (in.)
W8x24 to 67	7.13
W10x33 to 112	8.86
W12x40 to 336	10.91
W14x43 to 730	12.60

Figure 3-38. Full bearing of lighter sections when they are centered over heavier sections, for the listed weight groups.

Figure 3-39. Upper and lower column sections not centered, or different nominal sections bearing on each other.

Bearing on Finished Surfaces

AISC *Specification* Section J1.4 recognizes the complete transfer of loads through bearing on finished surfaces. A column splice connection seldom would be necessary except for safety and stability during erection. The method of fastening is required merely to hold the parts securely in place. However, physical, designed splice connections are necessary when the column is resisting bending moment[3] or net uplift at the splice. A physical splice is necessary, too, when the column may be subjected to considerable loads due to accidental or construction loading prior to placing the flooring or bracing system.

On the contact area of a milled surface, AISC *Specification* Section J7 permits a bearing strength of ϕR_n (LRFD) and R_n/Ω (ASD):

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

where

ϕ = 0.75 (LRFD) and Ω = 2.00 (ASD)

F_y = specified minimum yield stress, ksi

A_{pb} = projected bearing area, in.^{2}

The term milled or finished is intended to define a surface that has been finished to a true plane by milling, sawing or any acceptable means that produces such a finish.

When the columns are of different nominal depths, filler plates and/or bearing plates are used for the load transfer. Finished filler plates are proportioned to carry bearing loads at available strengths (ϕR_n and R_n/Ω), and the connections to the column shaft must in turn be designed to carry these

[3] *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.

calculated loads. The effect of eccentricity should be considered in developing the fastening bolts or welds. Unfinished filler plates should be used when the additional bearing area is not required. The unfinished filler is intended for "pack-out" of thickness. The end of the plate is set back 1/4 in. or more from the finished column end. As it does not transfer any load, no attachment, or only a nominal attachment for shipping, is required.

Bearing (butt) plates are used frequently on welded splices where the upper and lower shafts are of different nominal depths and splice plates are not generally used. The bearing plates usually will not be economical on a bolted splice because the "pack-out" fillers cannot be eliminated and they can be designed to serve as load bearing fillers. Bearing plates are normally selected to be 1 1/2 in. thick for a W8 over a W10 splice, and 2 in. thick for W10 to W12 and W12 to W14.

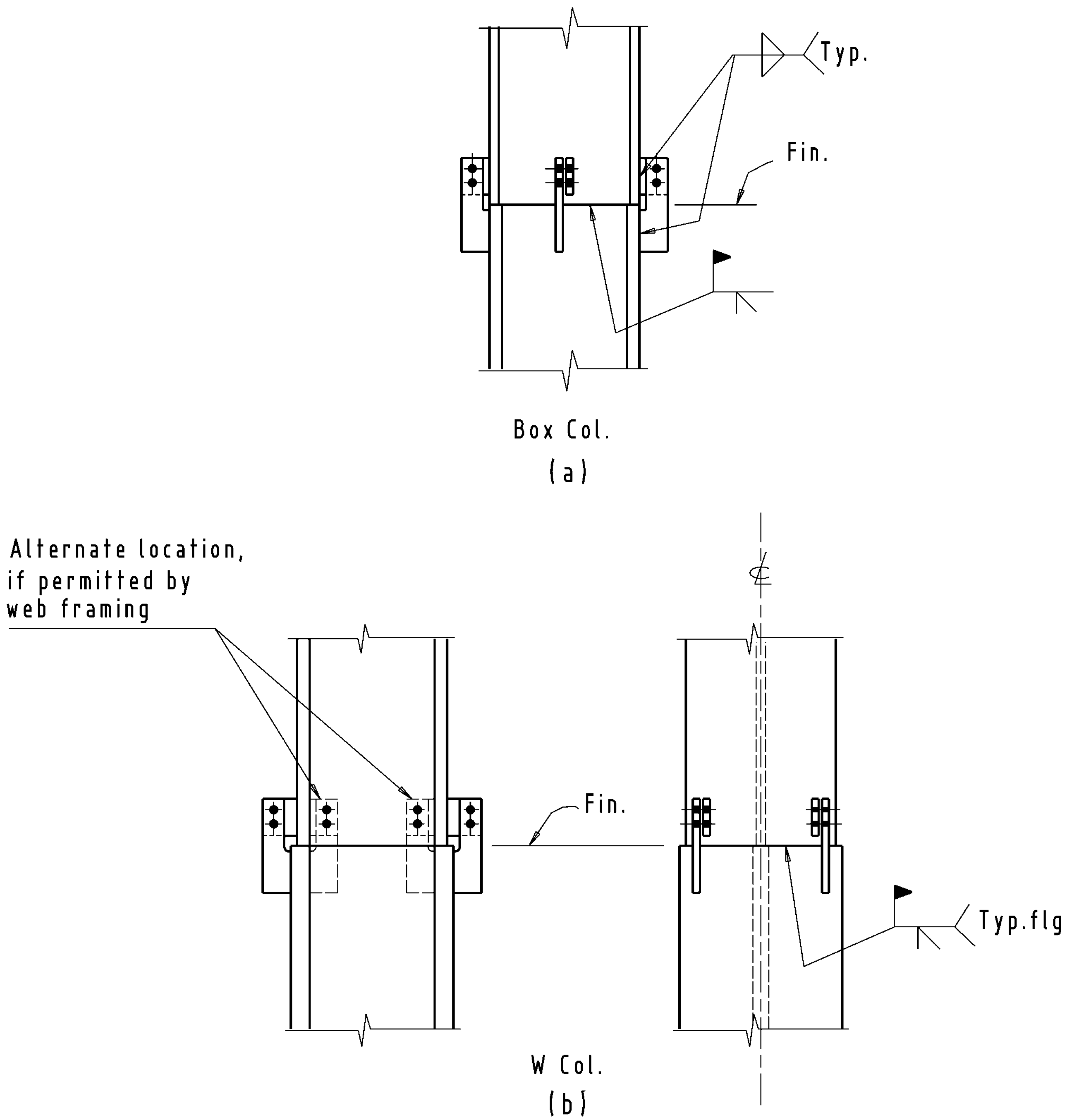


Figure 3-40. Column splices.

Bearing plates that are subject to substantial bending loads, such as required on boxed columns, require a more careful review and analysis. One method based on extensive experience is to assume a load transfer through the plate on a 45° bevel and then check the thickness obtained for shear and bearing strengths. The steel detailer is referred to AISC *Specification* Section M2.8, which sets forth the finish requirements for bearing plates. Because a butt plate fits between milled column ends, having both surfaces flat and true is important.

HSS COLUMNS

The previous discussion on column connections has dealt with W-shape members. Hollow structural section (HSS) columns are finding use in numerous structures such as schools and one- or two-story office buildings. Connections to these columns are similar to those already discussed except that generally, the connection material (framing angles, seat angles, single angles and single plates) is shop welded to the column. End-plate connections on beams are field welded to HSS columns.

TRUSS CONNECTIONS

When trusses are used in a design, the design drawings must show the detail of the truss joint, or provide the information necessary for the design and detailing of the truss joints. Additional information about these connections is presented in the *Manual* on page 13-11.

Truss Panel Point Connections—Welded Trusses

The truss shown in Figure A3-41 of Appendix A is the shop detail drawing of the truss whose design is shown in Figure A3-42. For convenience, truss joints are labeled U_0, U_1, L_1, L_2, etc. Normally, this is not done in detailing a completely shop assembled truss.

Figure 3-43a shows an enlarged detail of joint U_3 and represents the typical treatment of intermediate web connections. Note that the gravity axes of the vertical and diagonal members intersect on the neutral axis of the tee chord.

In order to lend stiffness to the finished truss, the angles comprising one of the web members at each joint (in this case, U_3L_3) are extended to the edge of the fillet of the tee (*k*-distance). The required welds are applied along the heel and toe of each angle, beginning at their ends rather than at the edge of the tee stem.

Other things being equal, common practice is to place more weld along the heel of the angle than along the toe. The intent is to minimize any eccentricity that may arise due to the location of the welds with respect to the gravity axes of the connected angles. Tests have shown, however, that little difference in static load strength exists between balanced and unbalanced connections of this nature, and AISC *Specification* Section J1.7 permits placing these welds as the joint geometry dictates. As will be seen later, sometimes the limitations on weld size are such that the same size weld must be used at the heel and toe of the angle and the geometry of the joint may result in more weld being placed along the toe than along the heel. If clearance permits, the ends of the web members also can be welded.

The sizes and lengths of welds are determined in accordance with the principles outlined earlier in this chapter. To compensate for loads induced in trusses during handling, shipping and erection, a common requirement is to design the connection for a minimum of 50% of the member strength or a lesser amount as determined by the engineer.

A case in point is the connection of diagonal U_3L_4 (Figure 3-43a). The connections should be able to support one-half of the effective strength of the member. In this case, checking the available tensile yield strength of two $2 \times 2 \times 1/4$ angles:

$$P_n = 1/2 F_y A_g$$
$$\phi = 0.90 \quad \text{(LRFD)}$$
$$\Omega = 1.67 \quad \text{(ASD)}$$
$$F_y = 36 \text{ ksi}$$
$$A_g = 1.89 \text{ in.}^2$$
$$\phi P_n = 0.90 \times 1/2 \times 36 \times 1.89 = 30.6 \text{ kips} \quad \text{(LRFD)}$$
$$P_n/\Omega = (1/2 \times 36 \times 1.89)/1.67 = 20.4 \text{ kips} \quad \text{(ASD)}$$

Checking the available tensile rupture strength:

$$P_n = 1/2 F_u A_e$$
$$\phi = 0.75 \quad \text{(LRFD)}$$
$$\Omega = 2.00 \quad \text{(ASD)}$$
$$F_u = 58 \text{ ksi}$$
$$A_e = A_n U \text{ (since the member is welded, } A_n = A_g\text{)}$$
$$= 1.89\,(1 - 0.586/2) = 1.34 \text{ in.}^2$$
$$\phi P_n = 0.75 \times 1/2 \times 58 \times 1.34 = 29.1 \text{ kips} \quad \text{(LRFD)}$$
$$P_n/\Omega = (1/2 \times 58 \times 1.34)/2.00 = 19.4 \text{ kips} \quad \text{(ASD)}$$

Note that A_e is the effective net area and U is the reduction coefficient. U represents the decrease in efficiency of the net section because of shear lag, discussed earlier in this chapter. For values of U and the equation from which they are determined, the steel detailer is referred to AISC *Specification* Section D3.3.

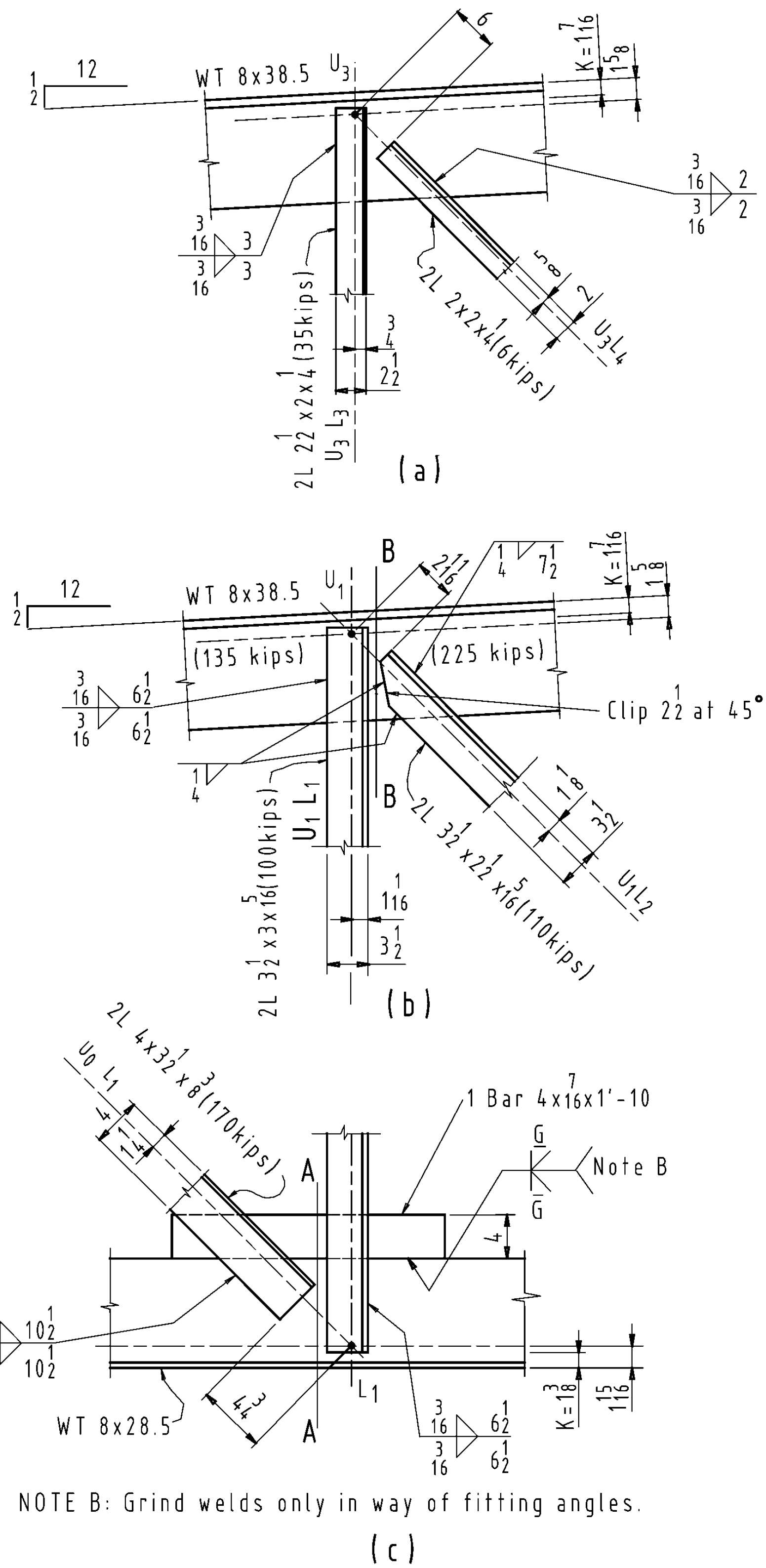


Figure 3-43. Enlarged detail of joint U_3 and the typical treatment of intermediate web connections.

As detailed, the weld strength is:

ϕR_n = 1.392DL (LRFD) and R_n/Ω = 0.928DL (ASD)

D = 3

L = 2 × 4 = 8 in.

ϕR_n = 1.392 × 3 × 8
= 33.4 kips (LRFD)

R_n/Ω = 0.928 × 3 × 8
= 22.3 kips (ASD)

The connection is adequate.

The available strength in axial compression of the double angles in member U_4L_4 (Figure A3-41) may be determined by referring to Table 4-8 in the *Manual* Part 4, which lists permissible compression loads for a wide range of paired angles of various effective lengths for steels with F_y = 36 ksi and F_y = 50 ksi. Member U_4L_4, in which the 3 × 2 × 1/4 angle legs are turned as shown in Figure 3-44 (short legs back-to-back or SLBB), has an effective length of about 7 ft. In the *Manual* Part 4, this length gives available strengths of 24.7 kips and 63.4 kips (LRFD) and 16.4 kips and 42.2 kips (ASD) about the X-X and Y-Y axes, respectively. As the strut can support safely no more than the lesser of these two values, its strength is limited to 24.7 kips (LRFD) and 16.4 kips (ASD), and the welds are proportioned for 24.7/2 = 12.4 kips per angle (LRFD) and 16.4/2 = 8.2 kips per angle (ASD). As detailed, the welds have a strength of:

ϕR_n = 1.392 × 3 × 2 × 2
= 16.7 kips per angle (LRFD)

R_n/Ω = 0.928 × 3 × 2 × 2
= 11.1 kips per angle (ASD)

which is adequate.

The diagonal member at joint U_1 (Figure 3-43b) is cut to provide needed weld length along the angle toe. It may have been possible to swing this member up and to the right and,

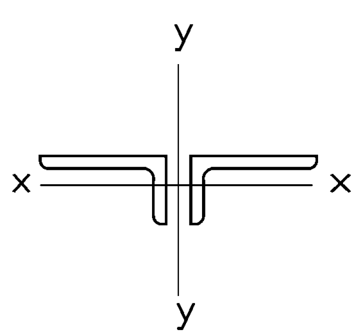


Figure 3-44. Double angles, long legs outstanding.

thereby, obtain the necessary length with a square end. To the greatest extent possible, the detailer should provide square ends on angles used as truss web members and as bracing to eliminate the added shop costs for laying out and cutting clipped ends. However, moving the diagonal also means moving its gravity axis away from the joint intersection. This will introduce eccentricity and consequent moment forces that, when added, tend to overload the chord. The recommended practice is to arrange all gravity axes to intersect at a common point at each joint. The detailer is directed to the *Manual* Part 13 for the design procedure for this joint.

Despite the preceding recommendation, placing gravity axes exactly on work lines may not be practical in some cases. Members U_2L_2 and U_4L_4 (Figure A3-41) have been moved purposely to provide connections for the bottom lateral bracing members. In both cases the resulting eccentricities are relatively small and can be neglected because of the stiffness of the joint. Should such an eccentricity be critical, the approval of the owner's designated representative for design should be obtained.

Connection Design

The ends of the truss members in Figure A3-41 are checked as follows:

- The maximum size fillet welds along toes of angles 1/4 in. or more in thickness should be 1/16 in. less than the thickness of the angle (AISC *Specification* Section J2.2b).
- The thickness of the tee stem is checked for available strength to transmit the load from the web members. Refer to "Strength of Connected Material" earlier in this chapter for the procedure to determine the strengths of the tee stems.
- Minimum size of fillet welds must meet the requirements of AISC *Specification* Section J2.2b. The thickness of the top chord tee stem is approximately 1/2 in. and that of the bottom tee stem is 7/16 in. The minimum size of fillet weld is 1/8 in. in both cases, which is governed by the thinner part joined (the 1/4-in. angles).

Amount of Weld Required

The amount of weld required at the ends of truss members can be determined by one of the following methods:

- Divide the length available for welding into the tensile or compressive force for which the connection is to be designed. This will give the required weld strength per linear inch, from which the size of fillet weld can be determined. The length available for welding can be scaled from the detail of the truss if the joint details are drawn accurately and to a scale of at least

1 in. = 1 ft 0 in. Unless a separate layout is made to a larger scale, at least 1/2-in. should be deducted from the scaled dimension in determining the length available for welding.

- Assume a size of fillet weld and divide the strength of this weld into the tensile or compressive force for which the connection is to be designed. This will give the length of weld required. The detailer is cautioned that this method is limited by the thickness of the tee stem (chord) and the gusset plate. Tension yielding, shear yielding and shear rupture of the stem and plate must be checked.
- Determine the total number of linear inches of 1/16-in. weld required, by dividing the strength per inch of a 1/16-in. weld (1.392 kip/in. or 0.928 kip/in. using E70XX electrode, LRFD and ASD, respectively) into the force for which the connection is to be designed. Then, by trying various combinations of weld size and length, the size and length of weld can be determined. This method is advantageous, particularly if different size welds are to be used at the heel and toe of the angle.

In establishing the lengths of fillet welds, a sufficient length of material must be available on which to start and stop the weld in order to obtain the full effective weld size. A commonly accepted rule in the industry provides a material length equal to the required effective length of weld plus twice the weld leg size. The detailer is referred to the *Manual* Part 13 for an example illustrating the design of a joint connection.

Figure A3-45 of Appendix A shows typical shop details of a truss of light construction. Note that the web members are single angles. Although this produces some eccentricity normal to the plane of the truss, this effect is considered by the engineer in the design of the members. Bars "pa" at the left end serve to stiffen the bottom flange stem, as well as to transmit the top chord force.

For lightly loaded trusses, single-angle web members are used often. In this case, placing all web members on the same side of the chord minimizes twisting of the chord. Staggering the web members causes a torque, $C \times e = T \times e$ on the chord, as shown in Figure 3-46.

The depth and width of the top chord tee, which may seem to be out of proportion to the other members, are required to resist bending induced by the off-panel location of purlins.

The small skewed angles in the plane of the top and bottom chords are for 7/8-in.-diameter diagonal rod bracing. Sufficient weld is furnished to resist the maximum available strength these rods can carry.

In Figure 3-47 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 that does not exceed 1 in 2 1/2 (4 3/4 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 ensure 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 run off or extension bars and backing, also known as backing bars.

Where heavy rolled shapes with flange thicknesses exceeding 2 in. or shapes built-up by welding plates more than 2 in. thick together 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 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

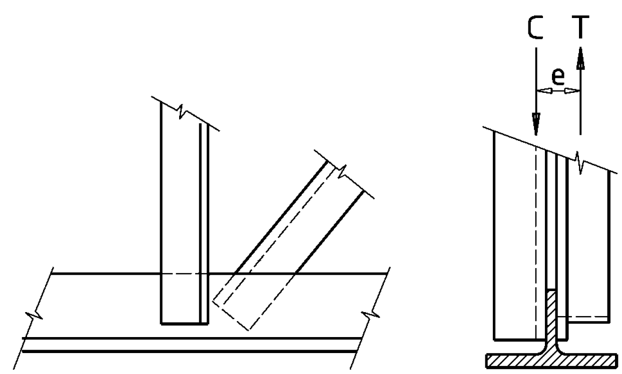


Figure 3-46. Torque on the chord caused by staggered web members.

A6, Supplementary Requirement S30 for shapes and S5 for plates. 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 *Specification* Sections A3.1c, A3.1d and J1.5 and their Commentaries for further information on the matter of splicing heavy sections.

The AISC *Specification* Table J2.5 permits available strengths for complete-joint-penetration groove welds to be equal to the available 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

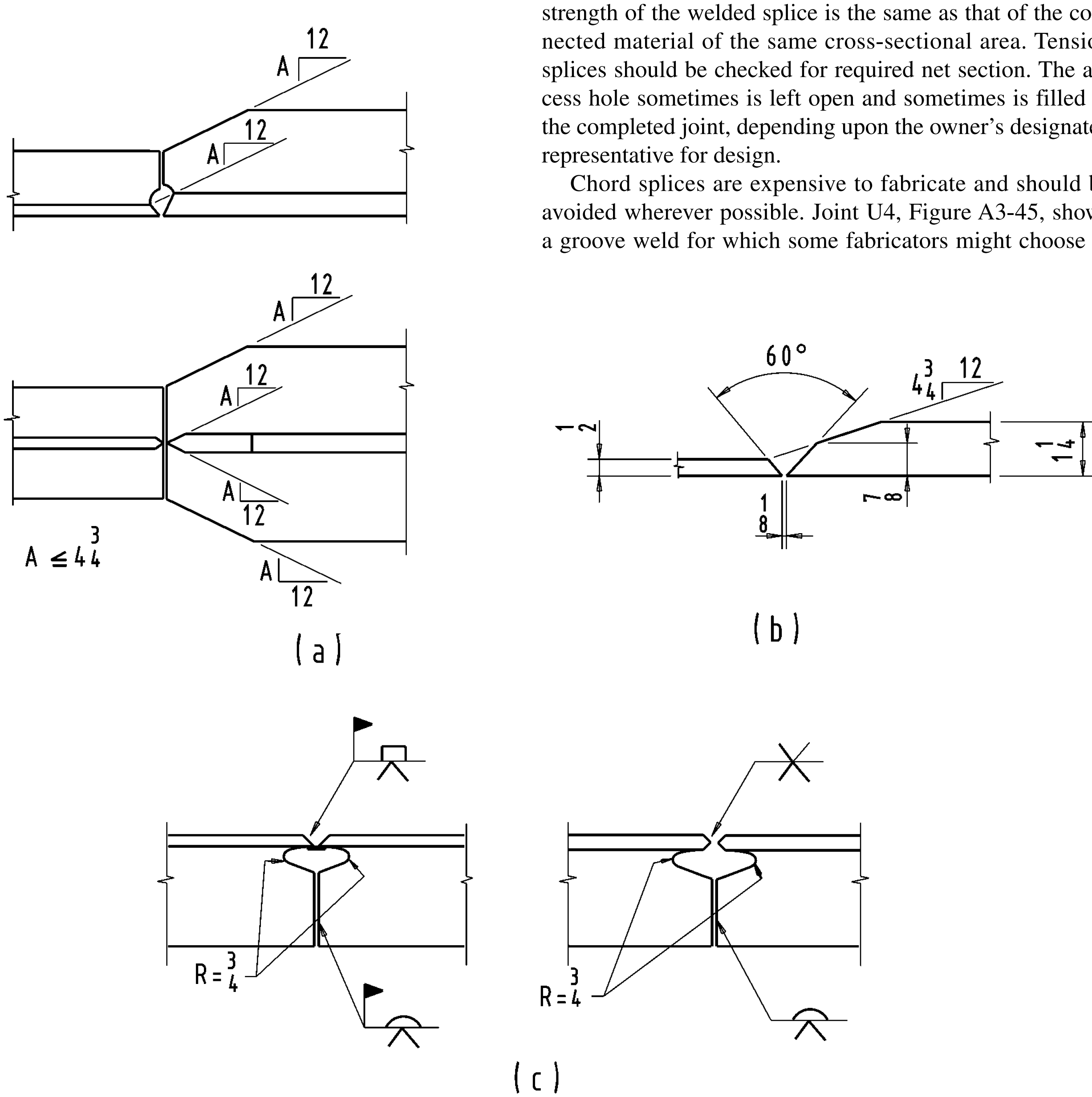


Figure 3-47. Welded truss-chord splices.

substitute a simple bead. If within the capacity of shop equipment, this is a satisfactory solution for most slightly pitched trusses.

In shop-welded, field-bolted construction, the field splices in chords require material that 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 top chord and the truss diagonal. However, the distribution of moment among these members will be proportional to the stiffness of the members. Thus, when the stiffness of the column is much greater than the stiffness of the other elements of the truss support connection, one design practice suggests that the column and gusset-to-column connection be designed for the full eccentricity. Another practice suggests designing the connection for the applicable eccentricities. Because of its importance, the truss support connection should be shown on the design drawing.

If the common intersection of the three working lines is shifted from the column centerline to the face of the column

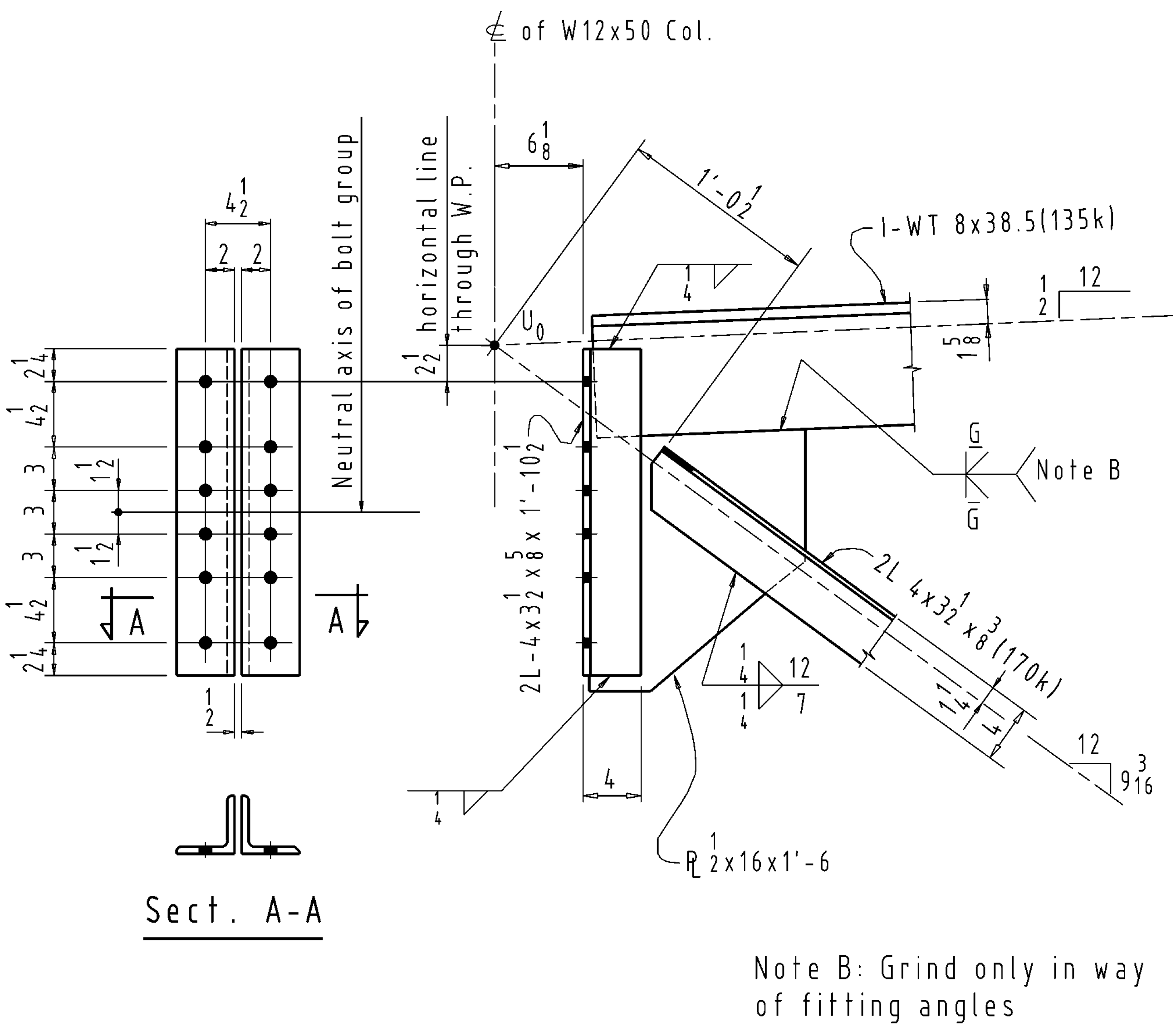


Figure 3-48. Top chord connection to column.

as in Figure 3-49, no eccentricity exists in the gusset, gusset-to-column connection, truss chord or diagonal. However, the column would be subjected to an eccentric loading and would have to be designed for that condition.

Figure 3-49 is a detail of the same joint as is shown in Figure 3-48, except for the location of the working point. In Figure 3-49 the detail has been laid out with a common working point established on the face of the column, where the working lines of the top chord and diagonal intersect. With all three working lines meeting at the same point, no unbalanced moment is present in the gusset or tee stem. The truss-to-column connection can be selected directly from Tables 10-1, 10-2 and 10-3 in the *Manual* Part 10. Comparing this detail with the one shown in Figure 3-48, the detailer may identify savings in welding, fasteners and fitting material. However, as previously stated, the supporting column in Figure 3-49 is subjected to an eccentric loading and must be investigated by the owner's designated representative for design for this condition.

Bottom Chord Connection to Column

When a truss is erected and loaded, its members in tension will lengthen and its members in compression will shorten. At the support connection this may cause the tension chord of a "square-ended" truss to encroach on its connection to the supporting column. When the connection is shop attached to the truss, erection clearance must be provided between the connection and the column face, using shims to fill whatever space remains after the truss is erected and loaded. However, in field-erected connections, provision must be made for the necessary adjustment in the connection.

When the tension chord delivers no calculated force to the connection, usually adjustment can be provided with slotted holes in the detail connection material. For short spans with relatively light loads, the comparatively small deflections can be absorbed by the normal hole clearances provided for bolted construction. Slightly greater misalignment can be corrected in the field by reaming the holes. If appreciable deflection is expected, the connection may be field welded or bolt holes may be field drilled. This is an expensive operation, which should be avoided if at all possible.

When the truss is erected in place and loaded, the various truss members will lengthen under tension or shorten under compression, and all or part of the camber will disappear. This may cause a problem when the tension chord of a "square-end" truss stretches and tends to encroach on its end connections. For short spans with relatively light loads, it is generally sufficient to provide end details in which the field bolting is through the truss chords or through the gussets in the plane of the truss. By this means, small deflections can be

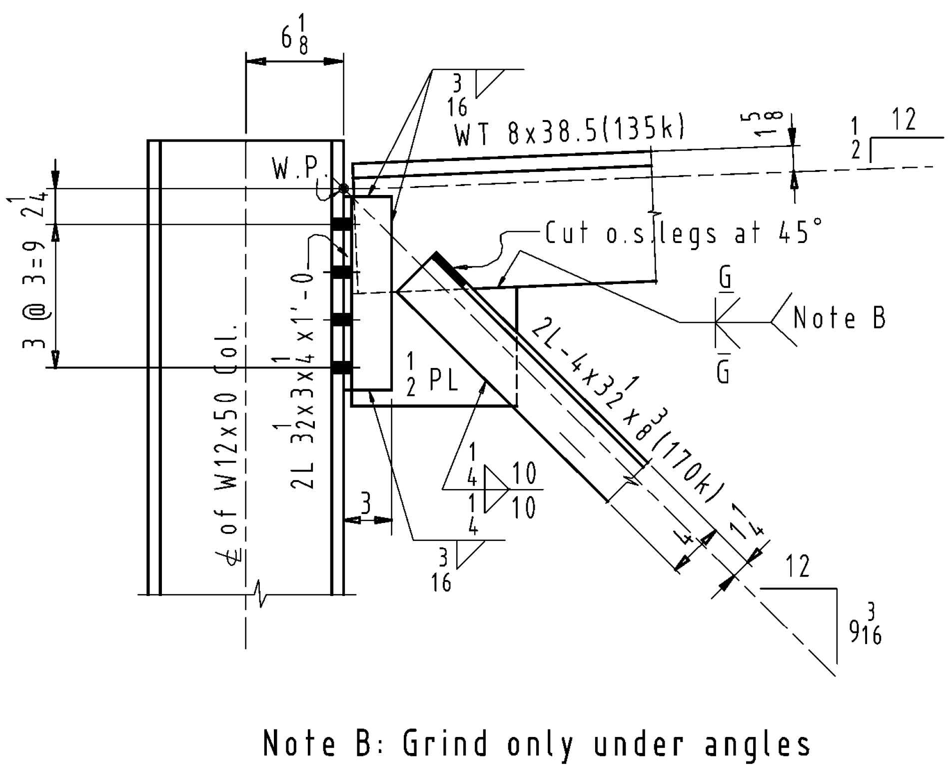


Figure 3-49. The common intersection of the three working lines is shifted from the column centerline to the face of the column.

absorbed by the normal hole clearances. Slightly greater misalignment can be corrected in the field by reaming or, if appreciable deflection is expected, by field drilling from the solid, although this is an expensive operation that should be avoided if at all possible. For bottom chord end panels in which there is no computed stress, the necessary adjustment can be provided by employing slotted holes in the detail connection material.

If shop attachment of abutting connections on the truss cannot be avoided, clearance should be allowed between such connections and the column face, and shims should be provided to fill out whatever space remains after the truss is erected and loaded.

Some idea of the amount of deformation to be expected may be determined by utilizing the stress-strain relationship of structural steel. The ratio of stress (kips per square inch) to strain (deformation in inches per inch) is called the modulus of elasticity and is taken as 29,000 ksi.

Having given the cross-sectional area of the chord and the design load in each panel, the deformation can be computed as:

$$\delta = \frac{PL}{AE}$$

where

δ = deformation, in.

P = axial load, kips

A = gross area of the chord, in.^{2}

L = length of the chord, in.

The value of δ is 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 that 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 than 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 AISC *Specification* Section J5, which describes the procedures for developing fillers in welded and bolted connections.

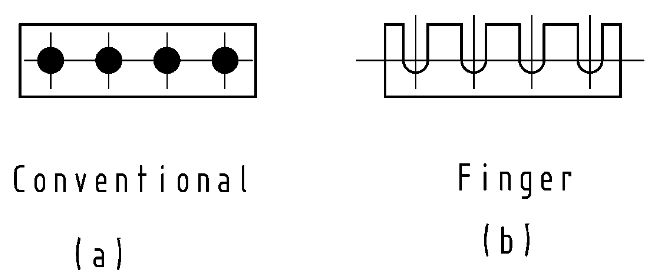


Figure 3-50. Shims and fillers.

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 $^3/_{32}$ in. high and numbers about $^5/_{32}$ 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 $^5/_8$ in. from the sketch and each succeeding line separated by about $^3/_8$ in.
8. All dimensions up to a foot are given in inches, thus: $11^{15}/_{16}$. All dimensions of a foot or over are given in feet and in., thus: 1'-0 or 1'-2$^1/_2$.
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 A1-2 in Appendix A).
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 di-

mension 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 ensure 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 A1-2). 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.
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.
37. When preparing shop drawings for beams and columns, use either 1½-in. scale for W-shapes and similar members having nominal depth up to and including 6 in.; 1-in. scale for nominal depth over 6 in. and up to and including 12 in. depth; and ¾-in. scale for nominal depth over 12 in. or as preferred by the fabricator. The scale used should make the drawing legible.
38. Where possible, make the gages in both flanges of W-shapes, channels and similar shapes the same.
39. Where permitted, clip the corners of web stiffener plates (cut the corners at 45°) to clear flange-to-web fillets of rolled shapes and flange-to-web welds of plate girders.
40. Where permitted, use partial-depth stiffeners instead of full-depth fitted stiffeners.

Bolting and Welding

41. Holes should be lined up on the same gage lines. Never break gage lines, except where doing so is unavoidable. Also, avoid having more than one hole size in a web or flange.
42. Never use the word "weld" on a drawing as a symbol. Always apply the appropriate AWS symbol to a welded joint.
43. Never use more welding than is necessary, as it adds to the cost of fabrication and/or erection and may cause the member to warp. Too much weld does not make a better joint.
44. Indicate the weld detail once for each different assembly piece and once on each shipping piece. By association the weld becomes part of the assembly piece.
45. Avoid mixing grades of bolts of the same diameter. For example, if a project requires the use of ASTM A325 and A490 bolts, use a different diameter for each grade of bolt.
46. When the finished appearance of a structure (or portion of a structure) is under consideration, placing the shop and field bolt heads on the exposed side is desirable. In

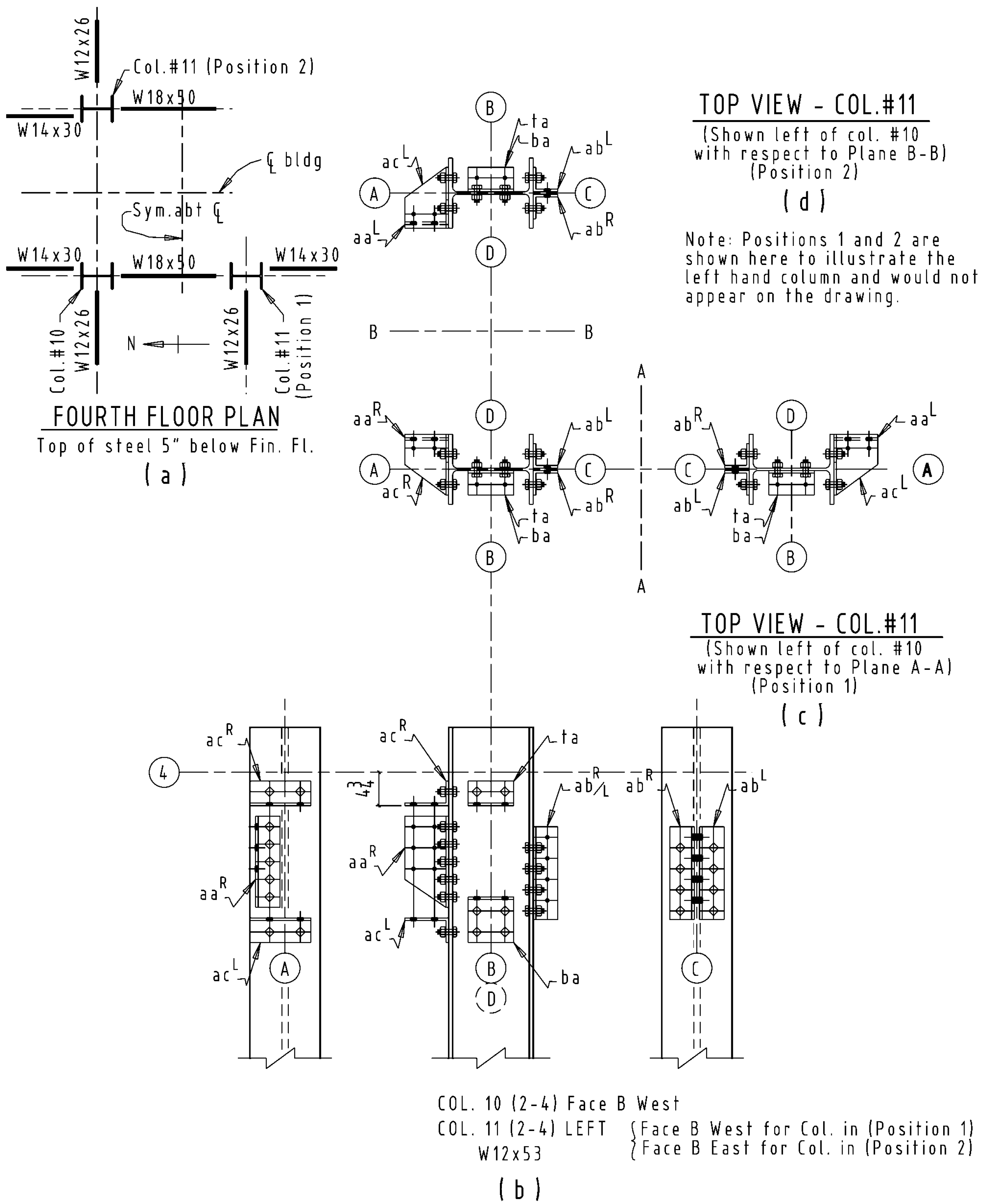


Figure 4-1. Column details, sections projected off the web view looking toward the bottom of the column.

this case, place on the pertinent shop and/or erection drawings a conspicuous note to this effect.

47. Where fillet welds on opposite sides of the same plane come to a corner, interrupt them as shown in Figure 3-25.
48. Avoid showing details that show fillet welds wrapped or returned around the ends of material (see Figure 3-26).
49. Avoid shop bolting and shop welding on the same piece, if possible. Most shops segregate these functions. Needless moving of pieces from one area of the shop to another is costly.
50. Use fillet welds whenever possible. Generally, they are more economical than groove welds.
51. Where acceptable by the owner's designated representative for design, use partial-joint-penetration groove welds in lieu of complete-joint-penetration groove, welds.
52. Never use lock washers on high-strength bolts. They are unnecessary and may be detrimental to bolt performance and inspection.

Shop and Field Considerations

53. Allow ample clearance to cover variations in cutting, shearing and coping. Approximately 1/2 in. is advisable for shop clearance and 3/4 in. to 1 in. for field clearances, depending on conditions of framing.

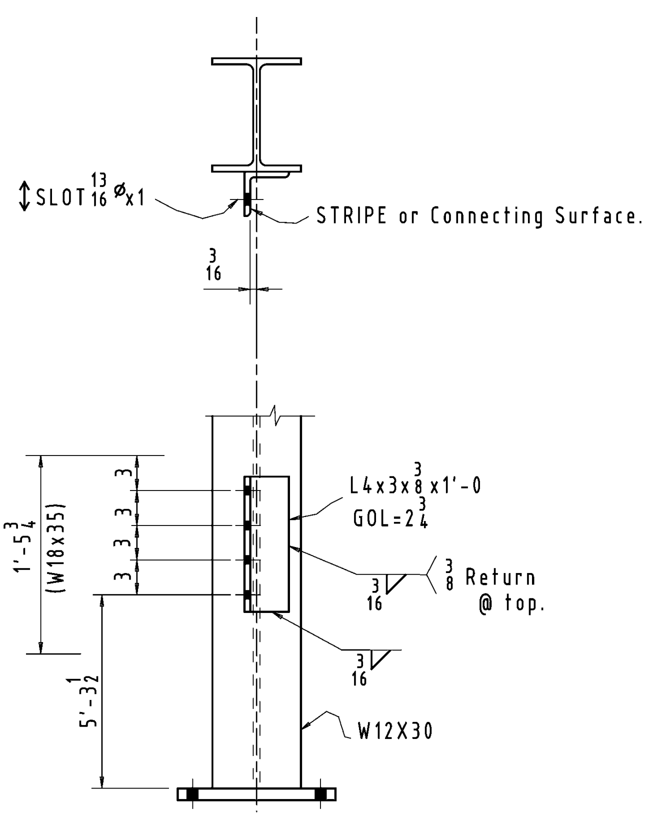


Figure 4-2. Wrap-around connection use.

54. When detailing work adjacent to existing structures or to walls, ensure the availability of suitable access so that the new work can be connected without disrupting the existing work.
55. When a project calls for shear stud connectors to be welded to the top flanges of beams or girders, OSHA requires that the shear stud connectors must be applied in the field. The top flanges of such beams and girders should not be shop painted.
56. Be alert to recognize situations requiring field adjustment and to provide means by which it can be made. Generally, such adjustments are needed in joints with slotted holes, oversized holes and shims, with due regard to bolt strength values. However, another option available for adjustment is field welding, especially when connecting to an existing structure and an accurate location of the existing member is unavailable. When welding to existing steel structures, the weldability of the existing steel must be verified.
57. Certain members could potentially be installed upside down. Verify with the erector if such members must be noted "TOP" on their upper sides.

Clearance Requirements

58. Examine all field conditions to ensure that bolts can be entered and tightened, especially in skewed configurations. Clearances for entering and tightening bolts must allow for use of an installation wrench. Also, depending upon the wrench used in the field, tension control bolts generally require clearances greater than those established for conventional high-strength bolts.
59. Remember to take fastener projections into account when determining erection clearances.

TOLERANCES

Mill tolerances are summarized in the *Manual* Part 1 under "Standard Mill Practice." Shop and erection tolerances are given in Part 16 of the *Manual* in the AISC *Code of Standard Practice*. Fabrication tolerances for built-up welded members are found in AWS D1.1.

In Chapter 3, while discussing the various types of connections, slotted holes were mentioned as a way to compensate for shop and mill tolerances. Another method is to establish lengths of beams or locations of connection angles on columns to create small gaps between supporting and supported members during erection and to fill such gaps with shims.

SYSTEMS OF SHEET NUMBERS AND MARKS

The use of some system of prefixes and suffixes in drawing numbers and shipping marks greatly facilitates shop operations. Often, it reduces the amount of marking that must be painted on the final shipping piece. Should part of the mark become obliterated in handling, the remaining lettering of the mark may permit identification of the piece. The following describes systems of numbering and marking, but the steel detailer must understand that each fabricator has a preferred system.

Sheet Numbers

A common manner of numbering shop drawings is by starting with 1 and continuing consecutively. A variation occurs when a large project is broken into shipping divisions or sequences. In this case, often the division (or sequence) number is included in a sheet number. Thus, for shipping division 1 the first sheet number is 101, followed by 102 and so on through 199. In the thirteenth division, for example, the first sheet number would be 1301. When projects are divided for shipping, the sizes of the divisions are determined to permit the fabricator and erector to meet the job schedule by furnishing the structure in sections. As a result the number of sheets in a division does not exceed 99. On very large projects the steel for the first division may be in fabrication, or even erection while shop drawings for steel in later divisions are in preparation.

Occasionally, the steel detailer will be required to prepare a shop drawing for an overhead crane runway column, a long girder or a long truss. Each of these types of members may have considerable groups of holes and numerous fittings to detail. In such a situation one sheet may be inadequate to contain all of the details and a shop bill. One method to accommodate this situation is to use more than one sheet and to add to the sheet number alpha suffixes. Thus, a long, complicated truss on drawing 10, requiring three sheets may have sheets numbered 10A/C, 10B/C, 10C/C.

Regardless of the marking system used, an easy reference between shop drawings and erection drawings must be established. This helps the steel detailers, approvers, shop personnel, erectors and inspectors.

Shipping and Erection Marks

Various systems of erection marking are used by different fabricating companies. The steel detailer should use the marking system of the employing fabricator. One commonly used system of marking shipping pieces includes a combination of the shop drawing number and the identification of the piece on the drawing. For instance, a beam detailed on drawing number 1 and given the shipping mark B1 would be marked 1B1. This reveals at a glance the drawing on which each beam is detailed. In the case of fabricating tier buildings, a fabricator may require that the shipping mark also contain the identification of the tier, floor, shipping sequence and crane (derrick) location.

Another system of shipping marks incorporating the sheet number is illustrated in Figure A1-2. The suffix 3 in each shipping mark—A3, B3, C3, etc.—indicates the sheet number on which the piece was detailed. This system may be convenient for small contracts involving a relatively small number of shop drawings.

Assembly Marks

All structural detail material on a shop drawing is identified by assembly marks, except for bolts, nuts, washers and other miscellaneous small fasteners. Assembly marks may consist of letters, numbers or a combination thereof depending on the preference of the fabricator. Marks consisting of letters may have either a single letter (beginning with "a" on each drawing and using certain letters of the alphabet) or a pair of letters with the first one describing the shape or function of the assembly piece (framing angle, seat angle, stiffener angle, base plate, cap plate, plates in general, etc.) and the second letter starting with "a" and continuing through the alphabet using certain letters. The following letters are generally considered suitable for marks:

a, b, c, d, f, g, h, k, m, n, p, t, v, w and x.

These marks are transferred to templates and to the assembly pieces as they are fabricated for identification purposes.

If identical assembly pieces appear on several shop drawings, their marks should be carried forward from the first drawing on which they appear. This saves shop costs by eliminating template work. As more than one method of carrying assembly marks forward exists, the steel detailer should follow the preference of the employing fabricator.

RIGHT- AND LEFT-HAND DETAILS

The steel detailer will have frequent reference to the concept of detail and/or main shipping pieces that have been or could be drawn and billed as "right-hand" or "left-hand" (mirror image) pieces. This is especially true of old drawings where the joining was made with rivets and the material preparation was almost always controlled by a template. Today, most fabricators are restricting the use of this short-cut concept because, while the cost of the detail drawing may be reduced somewhat, the opportunity for shop errors is increased considerably. CAD systems, while readily able to create a mirror image, are not able to handle the complexities of combination detailing. Additionally, the CAD system can quickly create the shop drawing of the piece, which would have been combined as a left-hand piece.

When used, the steel detailer must be able to visualize the concept of right-hand/left-hand details in order to avoid inadvertent errors of similarity. The following brief comments will illustrate the general principles. Although the comments and illustrations are directed toward typical W-shape columns, the information is applicable to other types of members (such as beams, plate girders, trusses, etc.) and HSS as well. A more thorough coverage of this concept can be found in any standard textbook on technical drafting.

The conditions that give rise to right- and left-hand columns are due solely to the arrangement of framing in the plan view. The presence of a right and left situation depends on the symmetry of the column framing with respect to assumed vertical planes. In Figure 4-4a the framing to columns M1 and M2 is symmetrical about vertical plane A-A, which is also perpendicular to the column webs. If the corresponding framing at each column requires the same connections and has the same relation to floor levels, these two columns will be right and left.

Likewise, the framing at columns N1 and N2 in Figure 4-4b is symmetrical with respect to vertical plane B-B, which in this case is parallel to the column webs. Other things being equal, these columns also will be right and left. Rights and lefts involving more than a pair of columns are possible, as shown in Figure 4-4c. Here, two axes of symmetry are shown by planes A-A and B-B. This relationship can occur between adjacent columns or at four widely separated corners.

Visualizing right and left situations by reference to axes of symmetry on the design drawings is one method available to the steel detailer. Another method is to visualize the right and left relationship by considering such columns paired about their Y-Y (web) axis, regardless of their positions in the building.

Direction marks on left-hand columns may be affected by the position they take with respect to the right-hand columns. Figures 4-1a and 4-1b illustrate cases where either the web or flange faces carry the same letter pointing North (or South). One note can do for each pair of columns. Figure 4-4c shows the notes that are required where pairs of identical rights and corresponding lefts, because of orientation, must have different direction marks.

As the steel detailer gains more experience in working with existing structures, the following variations of nomenclature may be encountered:

Right-hand	Left-hand
Right	Left
Thus	Reverse
As-shown	Opposite-hand

A convention has developed around the “handing” of columns, and other fabricated pieces. When the shipping pieces are identical in all respects except for being of the other hand, they are referred to as “Rights” and “Lefts” or “Thus” and “Reverse.” When differences of punching or in detail material exist, the “As-shown” and “Opposite-hand” nomenclature is used as described later.

These various nomenclatures can be considered as so-called industry standards even though the practice varies widely. In addition to columns they have been applied widely to beams, plate girders, trusses and other types of fabricated members. A Right/Left, Thus/Reverse or As-shown/Opposite-hand situation depends on the locations of holes and attachments to the member with respect to a vertical plane through its longitudinal axis.

As-Shown and Opposite-Hand Columns

As mentioned earlier, many fabricators do not permit the use of the nomenclatures right/left, thus/reverse or as-shown/opposite-hand on their detail drawings. Nevertheless, prior to the advent of CAD systems, these concepts were used extensively to fabricate many existing structures and the steel detailer should be familiar with them to be able to interpret shop drawings showing these nomenclatures. When a fabrication contract involves altering or tying into an existing structure, often the steel detailer is given shop drawings of that structure to use as a reference.

Where as-shown and opposite-hand terminology for detailing columns is employed, the principles are essentially the same as for true rights and lefts. All features of the as-shown piece are reproduced in the “left” sense on the opposite-hand column, if they apply to the opposite-hand column. The exception to true rights and lefts may consist of:

- Fittings and fabrication required on the as-shown piece and not on the opposite-hand piece; or,
- Fittings and fabrication required on the opposite-hand piece only.

The usual method of handling this on a drawing is to include all items that are required on both pieces on the as-shown piece. Fabrication and fittings that are needed only on the opposite-hand piece are located and drawn right hand, as though right-hand framing required their presence. The absence or presence of fittings and fabrication on either piece is covered either by notes or by noted views.

The partial details in Figure 4-5 illustrate the opposite-hand relationship of columns. In the fourth floor plan note that the W14×30 and W18×50 beams produce truly right and left framing conditions at columns 12 and 13. Their connections are shown in the right-hand position on the A and C faces of column 12. However, the web framing beams, although at the same elevation, are of different depths and require different connections. Rather than attempt to show both of these seated connections on the B face for column 12, and so complicate the dimensioning, the steel detailer chose to show separate web views and note which view applied to each column. Observe that the connections on the web view of column 13, opposite-hand column, as well as its projected section, appear in the same right-hand sense exhibited by corresponding connections on the web face and projected section of column 12.

In the case of a single unsymmetrical detail piece appearing only on the opposite-hand column, that piece will carry a right-hand assembly mark on the drawing, but will appear in the shop bill with a left-hand mark.

The W16×36, 3 ft 9 in. below the floor frames to column 12, but has no counterpart framing to column 13 (see Fourth Floor Plan). Holes A used to connect this beam are, therefore, shown on face C, but are noted for punching in column 12 only. Had this latter situation been reversed, with the W16 framing to column 13, holes A would appear in exactly the same place, but would be noted for punching in column 13.

Details on Right and Left Columns

Unsymmetrical detail fittings (i.e., those for which a left-hand sketch can be drawn) are identified on the right-hand piece by the superscript letter R appended to the identifying mark. In Figure 4-1a, angle aa^{R} on the flange (A face) of column 10(2-4) illustrates this condition. The views in Figures 4-1c

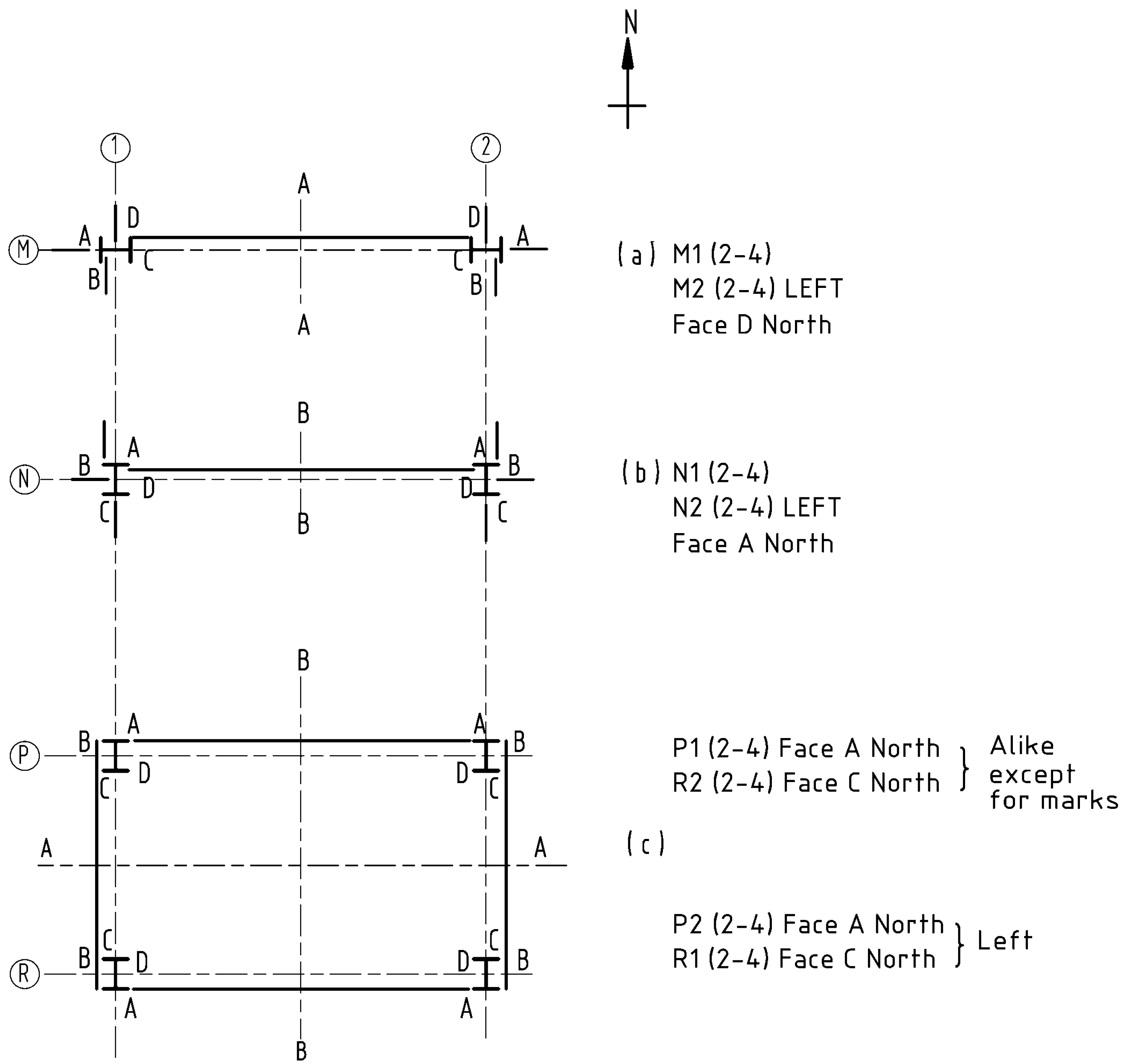
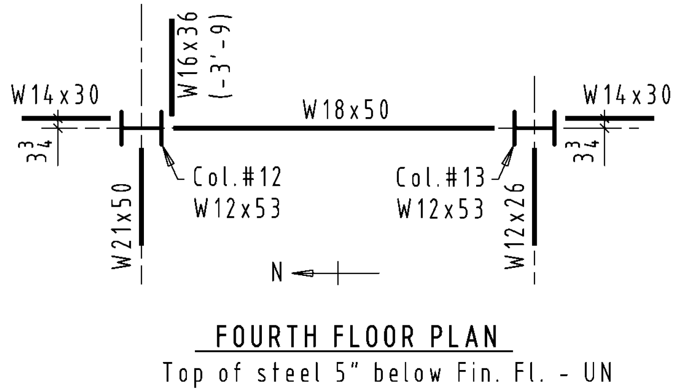


Figure 4-4. Typical framing details.



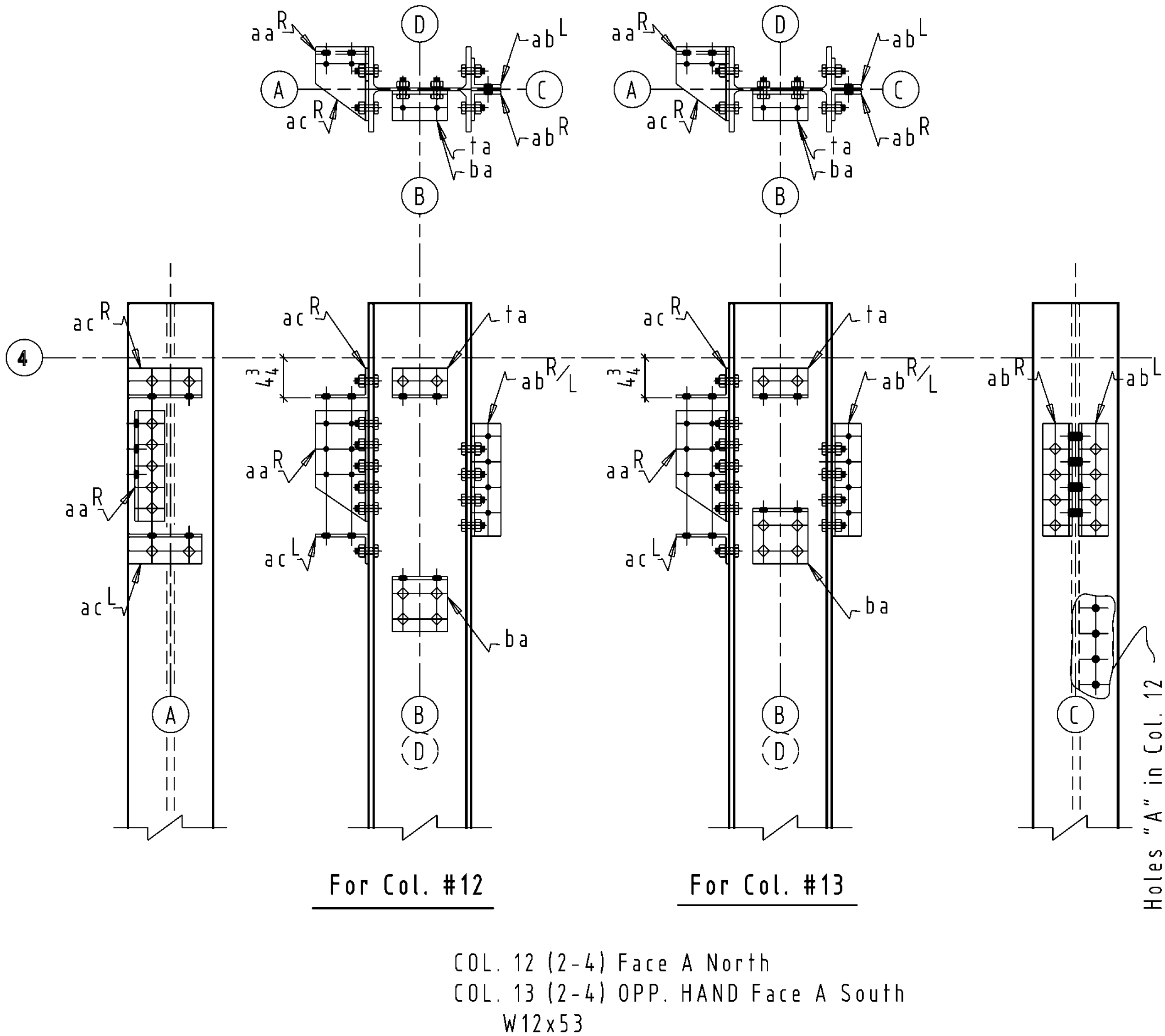


Figure 4-5. Partial details: the opposite-hand relationship of columns.

and 4-1d, representing sections of left-hand column 11(2-4), in either of two possible positions (Position 1 or Position 2), show this same angle as a left, labeled aa^L.

When two fittings on the same shipping piece are right and left with respect to each other, they will appear on that piece labeled R and L. Which of the two is to be marked R and which is L is of no consequence. The choice is optional with the steel detailer. Corresponding fittings on the left-hand shipping piece also are identified as R and L, but their location will be such that lefts will oppose rights and rights will oppose lefts when compared with the original right-hand shipping piece. This relationship is shown by angles ab^R, ab^L and ac^R, ac^L in Figure 4-1. Angles ta and ba, being symmetrical pieces, simply appear in their left-hand locations on the left-hand piece and no R and L marking is required. Note that the symmetry in plans between columns 10 and 11 exists with reference to vertical plane A-A for the Position 1 relationship or plane B-B for the Position 2 relationship.

Examples of marking right and left fittings on a shop drawing and of billing them in a shop bill are shown in Figure A4-6 in Appendix A. Note that the quantity of fittings required is an even number. The shop will make half the quantity as right-hand pieces and half the quantity as left-hand pieces. Other examples of billing right and left fittings are illustrated in Figures A1-2 and 7-46.

Some fabricators prefer shop drawings without shop bills. Examples of billing right and left fittings at their locations on the shipping pieces can be seen in Figures 7-45 and 7-49.

The right and left concept described for fittings extends to every aspect of fabrication of columns, beams, girders, trusses and other shipping pieces including all copes, cuts and patterns of punched holes for which a left-hand arrangement is possible.

STEEL DETAILING ECONOMY

Shop and drawing room time can be saved by the use of standardized connection material. Most fabricators have developed standard beam connections, based on the framed and seated connections shown in the *Manual*, which can be used on columns with a minimum of dimensioning. Such standards may include coded marks that describe each connection completely. Standard templates can be stocked for immediate use.

A similar practice with even greater potential savings is the employment of job standards. These cover connection material of all descriptions that repeat throughout a particular structure. Complete details and material billing with assigned standard assembly marks are developed and drawn on sheets separate from the shop drawings. By this means, fittings, for connections from simple splice plates to complex moment connections, may be copied onto shop drawings, using only those dimensions required to locate the connection and fabricate the main material. Job standard sheets are used in the template shop to produce the templates keyed to them by standard marks on the details.

The steel detailer is cautioned not to use cross-noting to the point where the drawing becomes a puzzle to the shop. Similarly, notes that refer to other notes should be avoided as this can cause confusion, especially if right and left details are concerned. Even greater confusion may ensue if revising a sketch that is repeated elsewhere by note becomes necessary when the revision applies only to the original sketch. The steel detailer is cautioned to limit the use of short-cuts of this nature to those sanctioned by past practice and included in the employer's example and reference drawings.

Another possible time and work saver, which may benefit both the shop and drawing room, is "subassembly detailing." This system presupposes a number of members (columns, girders, trusses, etc.) that have identical main material, but which differ in some degree as to detail fittings or other minor fabrication. Subassembly details are prepared showing only the fabrication that repeats exactly on all the members. This may include the assembly of trusses, the welding of box or girder sections, drilling or punching of all holes that repeat, the attachment of splices and other fittings, and any other work that is common to all members. Subsequent to this, final details are prepared, usually on reproductions of the original subassembly drawing, which complete all the work, including addition of fittings, etc., for each different piece.

The advantage of this procedure is that the shop can complete the bulk of the work on a run of identical subassemblies more efficiently than would be the case if each piece were worked individually, perhaps from separate shop drawings. Partially completed work may then be stockpiled until needed for final fabrication and shipment. Of course, this system should be used only after consultation with the shop and with the shop's full concurrence.

BOLTS

Most of the information available on bolts is found in the RCSC *Specification*, which can be found in the *Manual* Part 16. The steel detailer should be familiar with this document and use it in conjunction with this text.

Identification

ASTM A325, F1852 and A490 bolts are distinguished from each other and A307 bolts by various coding, the ASTM designation and the manufacturer's mark. Figure 4-7 shows the appearance and code markings of high-strength bolts.

Symbols

Shop bolts are identified on the shop drawings with notes specifying the type, diameter and length of the bolts. General or special notes on erection drawings specify the type of field

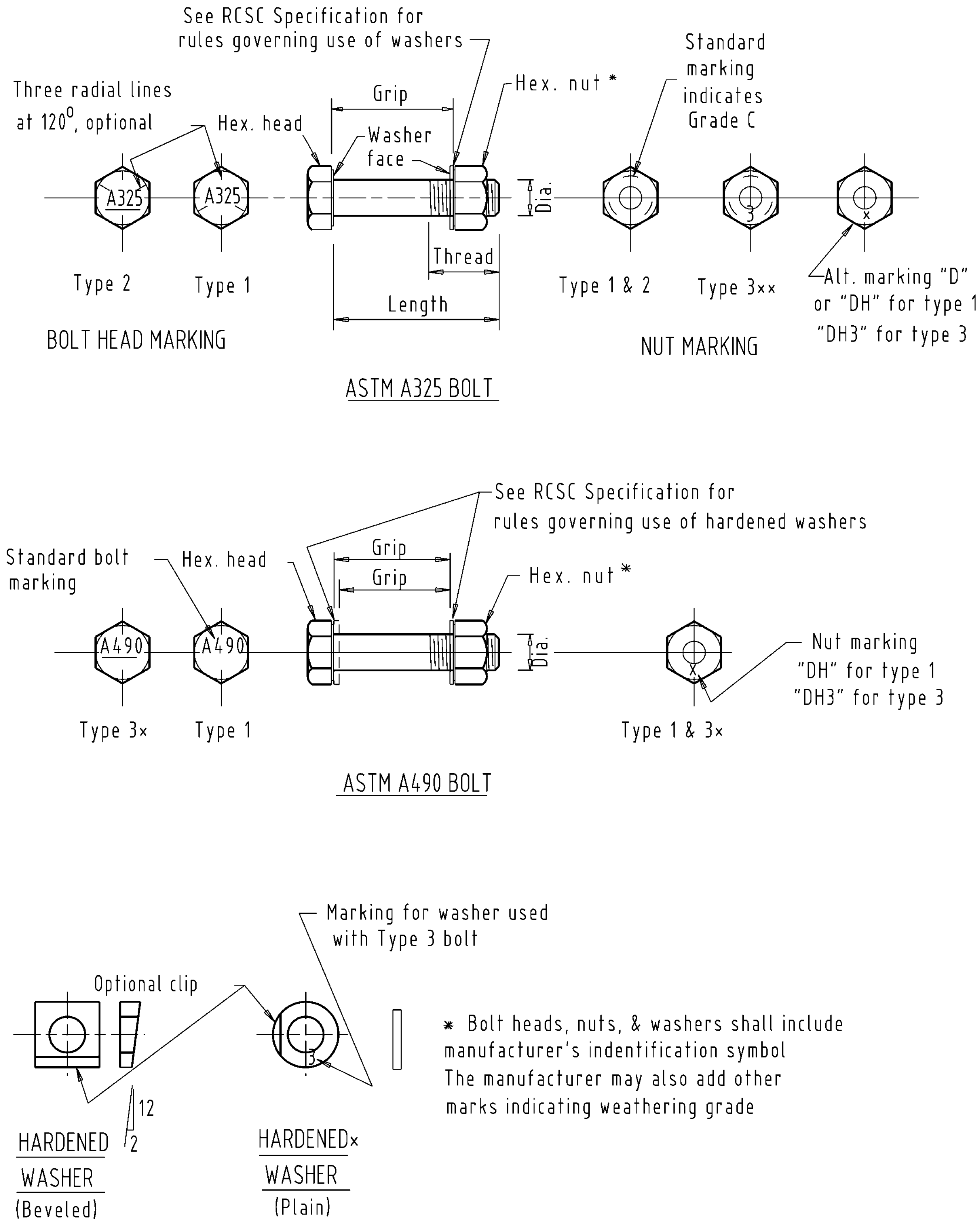


Figure 4-7. Bolt markings.

fasteners. Figure 4-8 shows conventional signs or symbols for bolts.

Holes

Standard hole sizes for bolts are made $^1/_{16}$ in. larger in diameter than the nominal size of the fastener body. This provides a certain amount of play in the holes, which compensates for small misalignments in hole location or assembly, and aids in the shop and field entry of fasteners. However, certain conditions, which may be expected in the performance of the structure and/or encountered in field erection, require greater adjustment than this clearance can provide. Consequently, on the basis of extensive laboratory testing (and subject to the approval of the owner's designated representative for design) the RCSC *Specification* sanctions the use of oversized and slotted holes. AISC *Specification* Section J3.2 and Tables J3.3 and J3.3M also establish the maximum sizes of bolt holes for standard, oversized, short-slotted and long-slotted conditions.

When the contract provides for the fabricator to furnish holes for the attachment of nonstructural material or to accommodate the work of other trades, the information giving the sizes and locations of such holes must be made available to the steel detailer in a timely manner so that the preparation of shop drawings will not be delayed. As much as possible, the holes so provided should be the same size as those required for the structural connections, especially in the main member.

A discussion of holes for anchor rods will be found in Chapter 7.

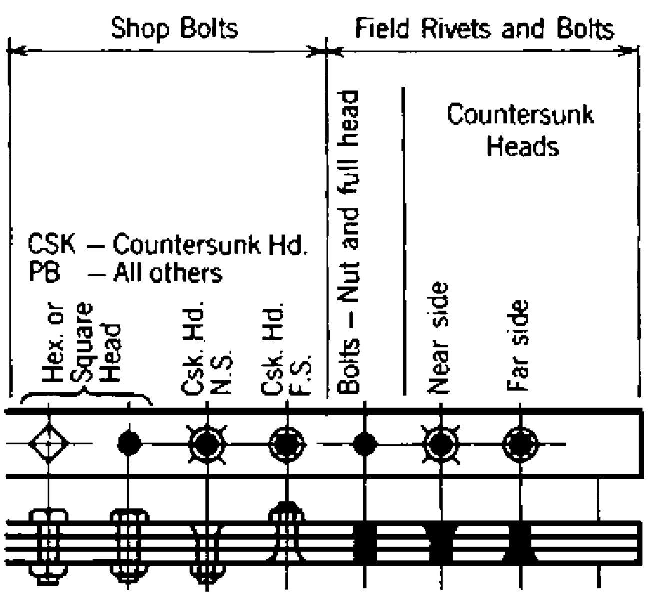


Figure 4-8. Conventional signs or symbols for bolts.

Installation

As indicated in the RCSC *Specification*, snug-tightened installation can be used for many bolts in steel buildings. When pretensioned installation is required, the methods approved in the RCSC Specification are:

- turn-of-nut method;
- calibrated wrench method;
- twist-off-type tension-control bolt method; and,
- direct tension indicator method.

The snug-tightened condition is defined in the RCSC *Specification* as "… the tightness that is attained with a few impacts of an impact wrench or the full effort of an ironworker using an ordinary spud wrench to bring the connected plies into firm contact." Bolts required to be pretensioned are first brought to the snug-tightened condition and subsequently pretensioned using one of the foregoing methods.

Proper location of hardened washers is an important element of a detail for the proper performance of the fasteners. Shop and erection drawings must reflect clearly the number and disposition of washers, especially the special washers that are required for the several slotted hole applications or to compensate for a lack of parallelism (see Figure 4-7). See RCSC *Specification* Section 6 for washer requirements.

WELDING

Welding is a process in which two pieces of metal are melted and fused together to form a joint. In structural welding this is accompanied by the addition of filler metal from an electrode.

The functions of welds in structural connections are similar to those of fasteners (see Chapter 3). Welds are used to transfer shear, tension and compression forces to and from structural joints and to transfer calculated forces from one part of a built-up member to another. They also are used to stitch together component parts of an assembly and to seal the edges of contact surfaces against moisture.

A secondary use of welds in shop assembly is the tacking in place of main and detail material prior to final bolting or welding. Tack welds normally are not shown on the drawing, but rather, are employed at the discretion of the shop. However, job specifications, the presence of material unsuitable for welding, or dynamic loading conditions may require notes prohibiting their use. Although tack welds are nonstructural, their presence may affect structural performance. Requirements for tack welding are included in the provisions of AWS D1.1. Some fabricators also practice temporary welding for shipping purposes, in lieu of bolting. Such welds are made small enough to permit easy removal in the field. To ensure safety in handling and shipment, these welds should be dimensioned and spaced on the drawing.

Specification requirements for welded connections in building construction are provided in the AISC *Specification*, which makes the appropriate references to the American Welding Society *Structural Welding Code—Steel*, AWS D1.1.

JOINT PREQUALIFICATION

Welded joints that conform to all AISC *Specification* and AWS D1.1 provisions for design, material and workmanship are designated as prequalified joints. This prequalification is intended to establish the joint profile and geometry that will permit the deposition of sound weld metal by a suitable welding process and is based on a long history of field experiences. Such joints include specific fillet- and groove-welded joints of sufficient variety to cover most requirements of structural work. These joints have been tested thoroughly and are recommended for general use. Although the use of prequalified joints is recommended, other joints and welding methods can be used after qualification as prescribed in AWS D1.1. Many such joints are in current successful use.

The *Manual* Part 8 duplicates the prequalified welds (dimensions in inches) shown in AWS D1.1 and establishes the dimensions and tolerances of a designated joint, process and position. See Figures 3-17a and 3-17b in this text for nomenclature of the common terms related to fillet and groove welds. Plug and slot welds are described in AISC *Specification* Section J2.3b. More detailed information on qualified joints, welding processes, material, design considerations, structural details, weld deposition qualification procedures, and inspection may be found in AWS *Structural Welding Code—Steel*, AWS D1.1.

The mere fact that a joint is designated as prequalified does not make it an appropriate joint design for all applications. Good joint design considers many factors, such as type and magnitude of loading, thickness and specification of base material, welding access and position, equipment available for handling, welding process available, proper material cleaning, edge preparation, and control of distortion, to name a few.

Particular attention should be given to any welded joint that may be highly restrained. Weld shrinkage can create stresses in the through-thickness dimension of material. This shrinkage, combined with a sufficiently restrained joint, can lead to material separation or joint failure called lamellar tearing. Figure 4-9 shows several joint configurations that illustrate this type of distress. For more information on lamellar tearing, refer to the paper "Commentary on Highly Restrained Welded Connections" published in the AISC *Engineering Journal* (AISC, 1973).

All prequalified and qualified welds should have a written welding procedure specification (WPS) prepared by the fabricator in the detail and outline given by AWS D1.1. This procedure should be available to the owner's designated representative for design and the inspector. The owner's designated representative for design may accept evidence of prior qualification of the joint welding procedures, as well as the welders, welding operators and tackers. Once qualified, a joint remains qualified unless an "essential variable" is changed. Welders, welding operators and tackers maintain their qualification indefinitely unless (1) there is a time period exceeding six months in which the individual has not been engaged in a given process of welding or (2) there is a specific reason to doubt the individual's ability. Plants that have obtained AISC Certification are expected to have developed the experience and documentation necessary to ensure that prior qualification of such welding procedures, as well as individual welder qualifications, is acceptable. Significant economies can be accomplished by accepting prior qualifications.

Note that AWS D1.1 is not a design specification in the sense of member sizes, loadings, strengths, locations, etc. Specifications such as those of AISC or local building codes will control the structural design aspects, but typically will include references to AWS D1.1 and its requirements.

AWS D1.1 is based on certain weldable grades of steel as listed therein by ASTM designation. Also, it contains all the materials accepted by AISC *Specification* Section A3.1a plus several additional grades. Filler metal and flux specifications are AWS specifications exclusively.

When welding is used extensively as a fastening method, it is not without problems of its own. As a result of its inherent rigidity, welded members are subject to severe restrictions in nominal strengths when they are subjected to repeated variations in stress through cyclic loading (fatigue). Abrupt changes or discontinuities in a section alter the stress path and create stress concentrations that are detrimental under cyclic loading. Gradual transitions of section will help to al-

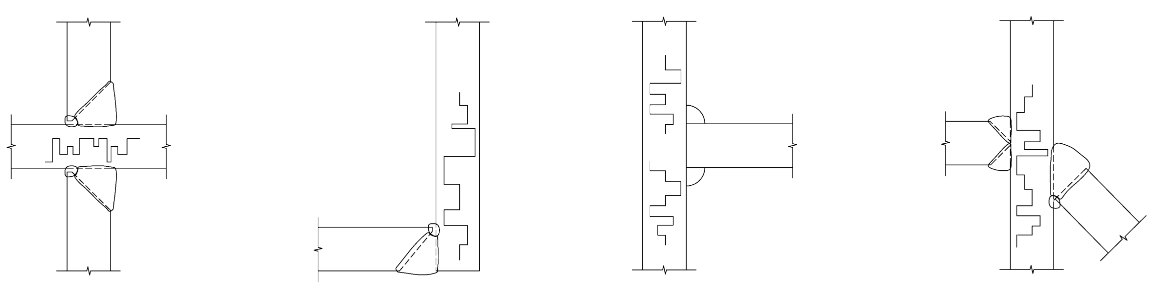


Figure 4-9. Weld configurations that can cause weld distress from shrinkage.

leviate these concentrations. The AISC *Specification* illustrates and controls certain details of welded and bolted connections in the Commentary, Appendix 3—Fatigue. Another problem inherent in welded fabrication is that of distortion and shrinkage of the parts being joined. Where these characteristics could affect the shape or strengths of a member, the fabricator develops procedures to control and minimize their effects.

WELDING PROCESSES

Shielded Metal Arc Welding (SMAW)

Shielded metal arc welding, an important welding procedure in both shop and field, sometimes is referred to as manual, stick or hand welding. An electric arc is produced between the end of a coated metal electrode and the steel components to be welded. The arc heats the base metal and the electrode to the point where they melt and form a molten pool on the surface of the work. As the arc is moved along its path, the pool solidifies behind it to form a homogeneous weld, which is fused with and becomes an effective part of both joint components. Figure 4-10a illustrates this sequence.

During the welding process the electrode coating serves two purposes: (1) it forms a gas shield to prevent absorption of impurities from the atmosphere, and (2) the flux in the coating purifies the molten metal. The flux forms a slag on the weld face and causes weld spatter, which sticks to the surrounding metal. Both of these deposits are removed readily by chipping or scraping for inspection.

Specifications for electrodes for the SMAW process are published in *Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding* (AWS A5.1) and *Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding* (AWS A5.5).

Submerged Arc Welding (SAW)

Submerged arc welding is an important process. It is performed either by an automatic or semi-automatic method in only the flat or horizontal position. The process is similar in principle to shielded metal arc welding. However, a continuous bare wire electrode is used instead of a coated "stick" electrode and the flux is supplied in granular form. Loose flux is placed over the joint to be welded and the electrode wire is pushed through the flux. As the arc is established, part of the flux melts to form a slag shield, which covers the molten metal (see Figure 4-10b).

Submerged arc welding results in deeper weld penetration and a considerably faster deposition rate of welding than is possible in the shielded metal arc process. The weld surface is smooth and weld spatter does not form. Here, too, the slag must be removed. Unmelted flux is recovered and reused.

In the automatic process an electrically controlled machine supplies the flux and wire through separate nozzles as it moves along a track. The semi-automatic method employs a hand-guided device, which automatically feeds both wire and flux through a single nozzle.

Specifications for flux-electrode combinations for submerged arc welding are published in *Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding* (AWS A5.17/AWS 5.17M) and *Specification for Low Alloy Steel Electrodes and Fluxes for Submerged Arc Welding* (AWS A5.23/A5.23M).

Sometimes, this process is used with multiple electrodes in the automatic process, where larger weld sizes and longer lengths justify this type of equipment.

Gas Metal Arc Welding (GMAW)

The gas metal arc welding process is important in structural shop applications because of its adaptability to all-position work. This process, which may be automatic or semi-automatic, utilizes an uncoated solid wire electrode with arc and weld metal shielding provided by a stream of gas (see Figure 4-10c). For this reason, its use in field welding may require the use of temporary screens to protect the shielding gas from the winds frequently encountered on construction projects. By changing the type of gas or the arc polarity, the cross-section of the deposited metal can be changed to produce broad, shallow penetration or relatively deep penetration at the weld centerline. This process also is known as MIG (metal inert gas) welding. When the arc is shielded with carbon dioxide, it may be called CO_2 welding.

Specifications for electrodes and recommendations for shielding gas for gas metal arc welding are published in *Specification for Carbon Steel Electrodes and Rods for Gas Shielded Arc Welding* (AWS A5.18) and *Specification for Low Alloy Steel Electrodes and Rods for Gas Shielded Arc Welding* (AWS A5.28).

Flux Cored Arc Welding (FCAW)

This process also may be automatic or semi-automatic. The electrodes are tubular and contain a flux-filled core, which provides shielding for the arc and weld metal. It has the same applications as the gas metal arc process with the added advantage that the arc shielding is not as likely to be affected by the wind in exposed conditions. Additional shielding for arc stability may be obtained from externally supplied gas or mixture of gases. Figure 4-10d illustrates this process.

The *Specification for Carbon Steel Electrodes for Flux Cored Arc Welding* (AWS A5.20) and *Specification for Low Alloy Steel Electrodes for Flux Cored Arc Welding* (AWS A5.29) give electrode and shielding requirements for these processes.

Electrogas Welding (GMAW-EG) or (FCAW-EG)

Electrogas welding is a method of gas metal arc welding or flux cored arc welding applicable to vertical position joints in material up to 3 in. thick. It is a fully automatic method,

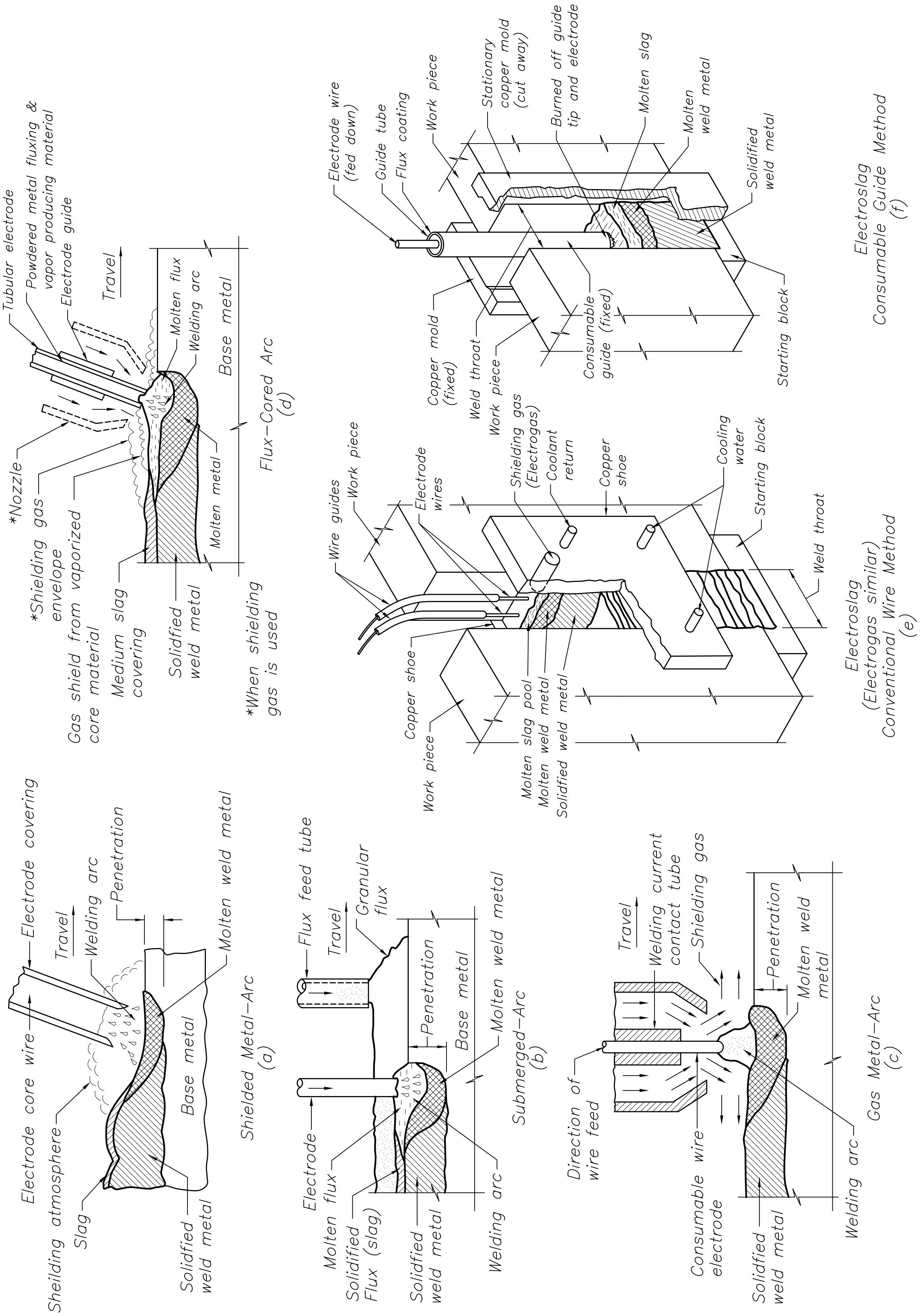


Figure 4-10. Welding processes.

wherein one or two wires are fed into the weld cavity formed by joint edges (butt joints) or between edges and surfaces (tee joints). Water-cooled copper dams on the sides of the weld cavity contain the molten weld metal and automatically rise as the weld metal cools and the weld progresses upward. Arc and weld metal shielding is provided either by slag and vapors from the flux core, by streams of shielding gas or by both gas and slag. Figure 4-10e, with the addition of gas ports at the tops of the copper dams, illustrates the apparatus for electrogas welding.

The *Specification for Carbon and Low Alloy Steel Electrodes for Electrogas Welding* (AWS A5.26/A5.26M) gives the electrode and shielding requirements for this process.

Electroslag Welding (ESW)

In the electroslag welding process, welds also are made vertically from bottom to top in the weld cavity to form butt or tee joints. Water-cooled or solid copper molds on either side of the joint contain the molten weld metal and slag as the weld is produced upward. At the start of the operation, and thereafter as necessary, granular flux is employed and the electrode extends through it to make the initial arc. One or more electrodes, depending on the weld size, are fed into the molten slag, where they are consumed to form the weld pool. The action in this process differs from other processes in that, once the slag has become molten, all arc action stops and the current passes through the slag, generating temperatures up to 3,500 °F. The hot slag fuses both the work surfaces and the electrodes, with the liquid metal combining in the pool below.

Because of the uniform distribution of heat throughout the joint, electroslag welding is preferred for applications requiring minimum distortion of joined members. Joint thicknesses up to 20 in. have been welded successfully.

Two methods of electroslag welding are in general use: (1) the conventional wire method, in which wire guides and molds move upward as the weld is completed (Figure 4-10e), and (2) the consumable guide method, in which stationary molds enclosing the full depth of the joint contain the weld, while fixed bare or flux-coated electrode guides, initially extending to the bottom of the joint, are consumed along with the electrodes as the weld surface rises (see Figure 4-10f).

The *Specification for Carbon and Low Alloy Steel Electrodes and Fluxes for Electroslag Welding* (AWS A5.5) provides the electrode and shielding requirements for this process. See AWS D1.1 for the restrictions on the use of this process.

Stud Welding

This is a process by which specially manufactured shear stud connectors are end-welded to structural members. A gun-type device with special controls holds the shear stud connector in position, strikes an electric arc between the stud and the member and, when fusion temperature is reached, automatically pushes the stud into the molten pool of metal. Fluxing arrangements vary, depending on the manufacturer. A ceramic collar (ferrule) encloses the molten metal during the welding process and is discarded after the welding is completed. No external wire or rod is used in this process.

Shear stud connectors also may be welded using SMAW, GMAW or FCAW fillet welding processes. The minimum permissible size of fillet weld used depends upon the diameter of the welded stud.

The Occupational Safety and Health Administration (OSHA) requires that shear stud connectors for composite construction should be installed in the field, not in the shop. Otherwise, they are a tripping hazard for erector personnel on the walking surface of steel beams.

Caution must be exercised in welding shear stud connectors in positions other than flat, as gravity tends to pull the molten metal down from the joint to produce a less-than-complete weld. The larger stud diameters are more susceptible to this problem.

AWS D1.1 Section 7—Stud Welding, sets forth the rules and regulations concerning the manufacture, installation and testing of stud welding.

Resistance Welding

Heat for fusion in resistance welding is generated by resistance to electric current flow as it passes from the work through the contact area where the weld is to be made. When the metal reaches a suitable temperature, pressure is applied that unites the two components in the joint area.

In structural work, resistance welding is used principally in the fabrication of lighter types of work such as open-web joists and other specialties produced by some fabricators. This process, which includes spot, seam, projection and flash welding, is of minor importance in structural welding.

WELDING ELECTRODES

The E70XX (AWS A5.1) class of electrodes (70-ksi tensile strength) is most commonly used in structural work for shielded metal arc welding. The E70XX (AWS A5.5) class covers low alloy electrodes for the same purpose. Comparable submerged arc flux-electrode combinations are in the F7XX-EXXX (AWS A5.17) class. Gas metal arc electrodes are represented by the ER70S-X (AWS A5.18) class. Flux cored electrodes are in the E7XT-X (AWS A5.20) class. As can be seen in Table 3.1 of AWS D1.1, which lists the various electrode classes in an ascending order of weld strength, the 70 grade is compatible with most of the carbon and high strength low alloy steels listed in the AISC *Specification*.

The terms E70XX, F7XX-EXXX, ER70S-X and E7XT-X are abbreviations for electrode or flux-electrode classes. The letter E stands for electrode, F stands for flux, S stands for solid electrodes, and T indicates a flux-cored tubular electrode. The number 70 is the specified minimum fracture strength of deposited weld metal, in ksi. The number 7 appearing in F7XX-EXXX is also a minimum specified fracture strength of deposited weld metal, but expressed in 10-ksi units (7 = 70 ksi). The suffix X's stand for numbers and are replaced by numbers when design considerations so require. Thus, E70XX might be expressed as E7014, in which case the number 1 indicates the optimum welding position and 4 indicates the electric current to be used, as well as the type of electrode covering.

The various X sequences in electrode classes for the other processes have similar or additional significance. The determination of these numbers and their interpretation may be made by reference to the applicable electrode specification.

WELD TYPES

Welds are identified by their profile or cross-section. The two most important types of structural welds are fillet welds and groove welds [which includes both complete-joint-penetration (CJP) and partial-joint-penetration (PJP) groove welds]. Back welds, used in conjunction with single CJP groove welds most often after gouging the root to sound metal, serve to complete the weld penetration at the weld root. Furthermore, in double-bevel complete-joint-penetration groove welds, backgouging and filling root passes may be required. Backgouging is the removal of weld metal and base metal from the weld root side of a welded joint to facilitate complete fusion and complete joint penetration upon subsequent welding from that side. Other structural welds are plug, slot, and flare welds, which are useful but limited in application.

Fillet Welds

Fillet welds are welds of theoretically triangular cross-section joining two surfaces approximately at right angles to each other in lap, tee and corner joints. They are used also in conjunction with groove welds as reinforcement in corner joints. Figure 4-11 shows several applications of fillet welds.

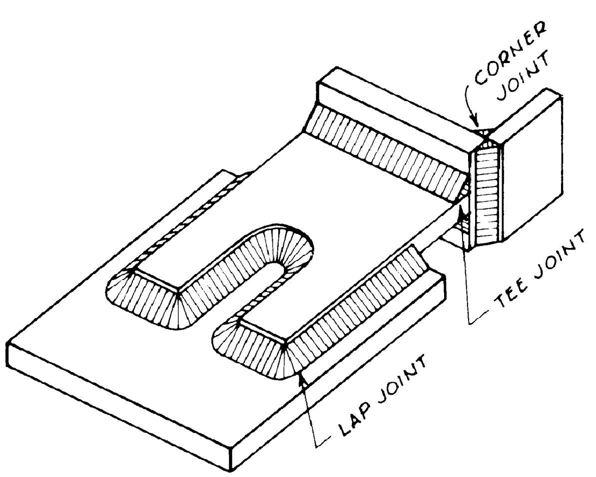


Figure 4-11. Fillet welds.

The cross-section of a typical fillet weld is a right triangle with equal legs. Figure 3-17a illustrates a fillet weld section and gives the nomenclature of its parts. The leg size designates the size of the weld. The root is the point at which the legs intersect. A line perpendicular to the weld face and passing through the root locates the weld throat; the length of this line from root to face is the normal throat size. When this line extends beyond the root, as permitted by AISC *Specification* Section J2.2 for the submerged arc process because of its deeper penetration, it becomes the deep-penetration throat size illustrated in Figure 3-17a. A plane passed through the throat and root lines contains the effective throat area. The effective length of a fillet weld is the distance from end to end of the full size fillet measured parallel to its root line. For curved fillet welds, illustrated in Figure 3-17a, the effective length is measured along the centerline of the throat.

The actual cross-section of a fillet weld may differ from the proportions pictured in Figure 3-17a in several ways. The included angle of weld deposit may vary from 60° to 135°, as shown in Figure 4-12, or it may be desirable to employ unequal leg welds, as shown in Figure 3-17a. In weld strength calculations, the use of the normal throat size will, in most cases, be conservative. However, when the included angle of weld deposit is greater, substantially, than 90°, the effective throat size should be determined from the actual dimensions.

Convexity or concavity of the weld face usually is present in all fillet welds. This is not necessarily a defect. The limits of such deviations are subject to AWS D1.1 provisions. Note that in Figure 3-17a the geometry of the weld is determined from the largest 45° right triangle that can be inscribed within the convex or concave outline.

Figure 4-12 illustrates the limitations of angle and edge preparations for a prequalified skewed tee-joint fillet weld. It also illustrates the intent of AISC *Specification* Section J2.2b for the maximum size fillet weld along the edge of the material.

Groove Welds

Groove welds are made in a groove between adjacent ends, edges or surfaces of two parts to be joined in a butt joint, tee joint or corner joint (see the *Manual* Part 8, "Prequalified Welded Joints"). The standard types of groove welds are:

- Square groove
- Single-vee groove
- Double-vee groove
- Single-bevel groove
- Double-bevel groove
- Single-U groove
- Double-U groove
- Single-J groove
- Double-J groove

The edges or ends to be groove welded usually are prepared by flame cutting, arc-air gouging or edge planing to provide square, vee, bevel, U- or J-shaped grooves that are straight and true to dimension. Relatively thin material may be groove welded with square cut edges.

Groove welds are classified further as either complete-joint-penetration groove welds or partial-joint-penetration groove welds (see Figure 3-17b). A complete-joint-penetration groove weld is one that achieves fusion of weld and base metal throughout the depth of the joint. It is made by welding from both sides of the joint or from one side to a backing bar or back weld. For complete-joint-penetration groove welds where backing bars are employed and are not to be removed, specifications require that the weld roots generally must be chipped or gouged to sound metal before making the second weld. For purposes of strength computation, the throat dimension of a complete-joint-penetration groove weld is considered to be the full thickness of the thinner part joined.

Partial-joint-penetration groove welds are employed when strengths to be transferred do not require complete penetration, or when welding must be done from one side of a joint only and it is not possible to use backing bars or to gouge the root for back welds. The application of partial-joint-penetration groove welds is governed by AISC *Specification* Section J2.1, which limits the effective throat thickness or the thickness of the material on which they are to be used. The effective throat thickness of a partial-joint-penetration groove weld is the minimum distance from the root of the joint to its face, as shown in Figure 3-17b.

From the foregoing, the steel detailer can understand that, from a practical standpoint, complete-joint-penetration groove welds allow for no joint defects and, thus, are more expensive to produce, inspect and, where necessary, correct than partial-joint-penetration groove welds.

Edge preparation of base material for partial-joint-penetration groove welds is similar to that for complete-joint-penetration groove welds, but it usually covers less than the full joint thickness (see Figure 3-17b). The effective

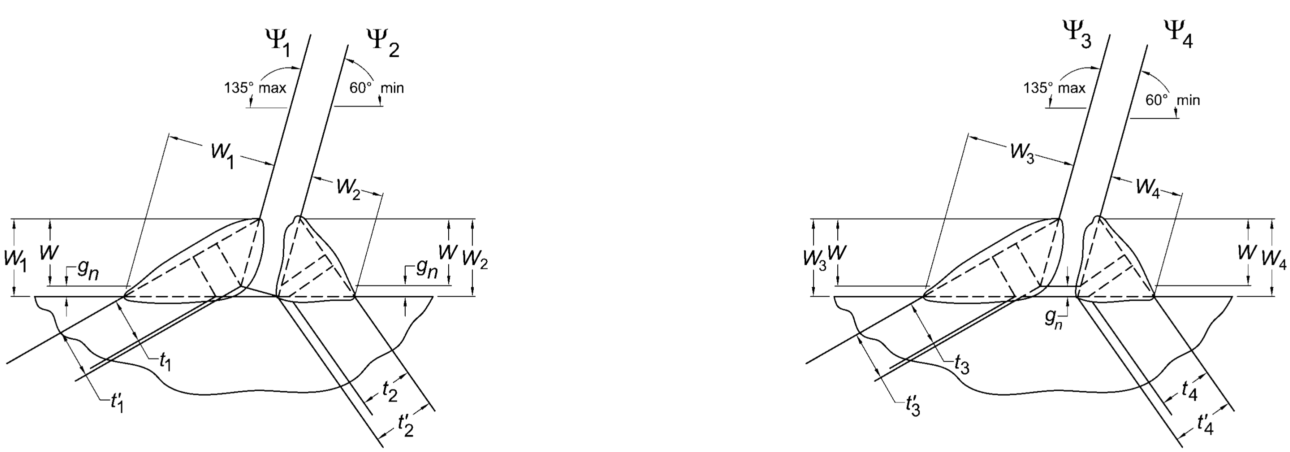
Base metal less than ¼ thick

(a)

Base metal ¼ or more in thickness

(b)

Maximum size of fillet weld along edges



(c)

(d)

Skewed T-joints

Figure 4-12. Limitations of angle and edge preparations for prequalified skewed tee-joint fillet weld.

throat thickness and, hence, the weld strength of partial-joint-penetration groove welds is limited normally to less than the full joint thickness.

Use of partial-joint-penetration groove welds is subject to AWS D1.1 and other specification provisions. These are more restrictive in bridge specifications than in building codes.

The nomenclature of groove welds is given in Figure 3-17b. The terms refer to the preparation of material and the relationship of abutting parts, as well as to the welds themselves. Figure 4-13 shows typical examples of partial-joint-penetration groove welds. Figure 4-14 shows typical examples of complete-joint-penetration groove welds. Reinforcing fillets, shown in Figure 4-14, are usually specified in buildings only when design requirements dictate their use. The proportions and dimensions of groove welds that may be used without qualification are shown in the *Manual* Part 8 and in AWS D1.1. Note that the submerged arc process requires relatively wide root faces and either zero root openings or the use of backing bars. Preparation for submerged arc welding must be made carefully and the joint faces or the backing bars drawn up tightly to prevent bleeding of the molten metal or burning through the joint due to the high heat and deep penetration.

Flare welds are special cases of groove welds in which the groove surface of one or both parts of a joint is convex. This convexity may be the result of edge preparation, but more often one or both joint components consists of a round rod or a shape with a rounded corner or bend. Figure 4-15 illustrates several types of flare welded joints. The effective throat dimension is used in design calculations. Complete-joint-

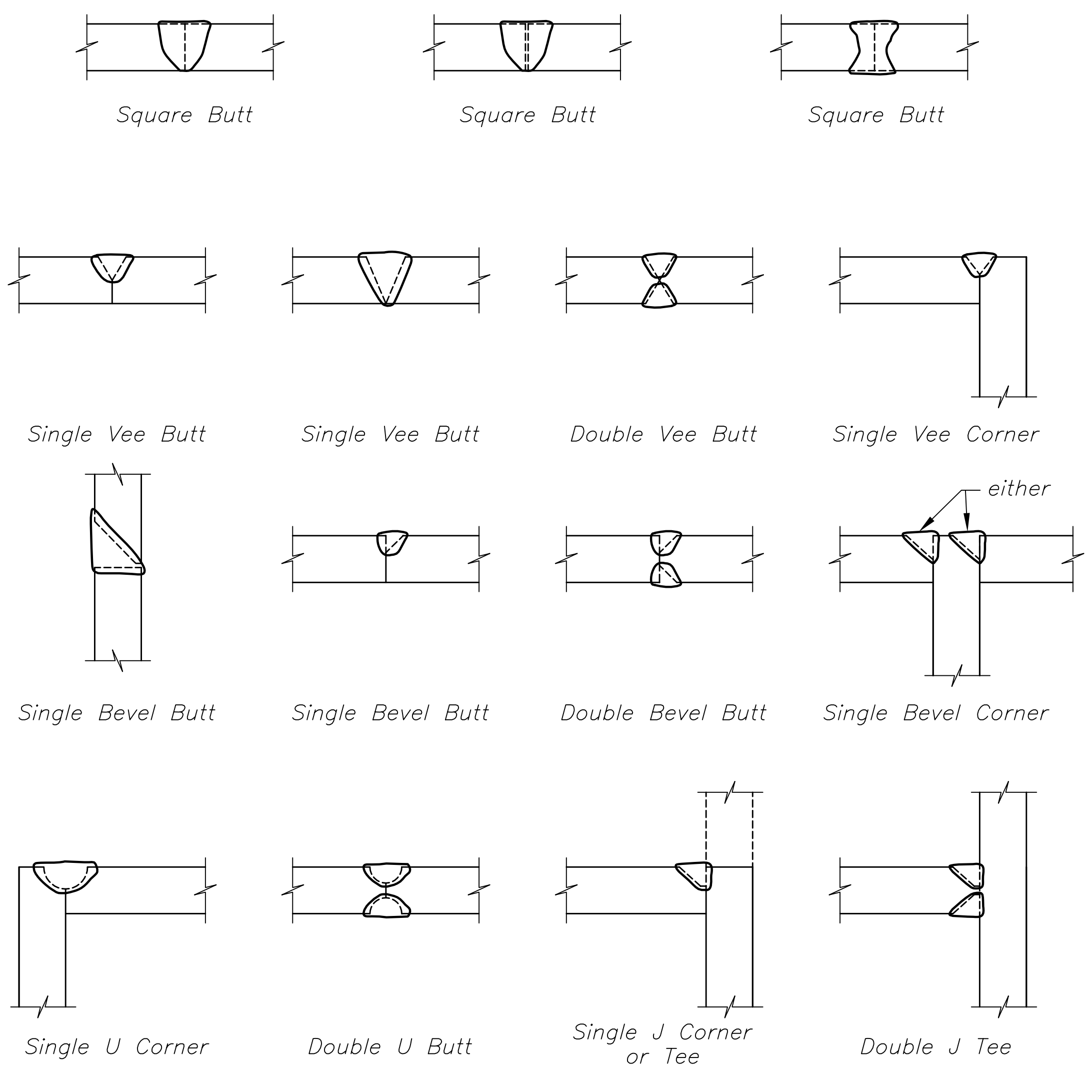


Figure 4-13. Typical partial-joint-penetration groove welds.

penetration in a flare weld usually is difficult to achieve and design values should be applied conservatively. AWS D1.1 discounts the effective throat size that the designer can use because of the difficulty in obtaining penetration.

AISC *Specification* Section J2.1a establishes the effective throat thickness of a flare groove weld, when the weld is flush (tangent) to the surface of the solid section, as shown in Figure 4-15. AISC *Specification* Table J2.2 defines the ratio of radius to effective throat.

Welding of bars should not be undertaken without a welding procedure that considers the chemistry of the base metals to be joined. The weldability of concrete reinforcing bars, in particular, is not a part of the ASTM specifications for several popular plain and deformed bars. ASTM A706, *Standard*

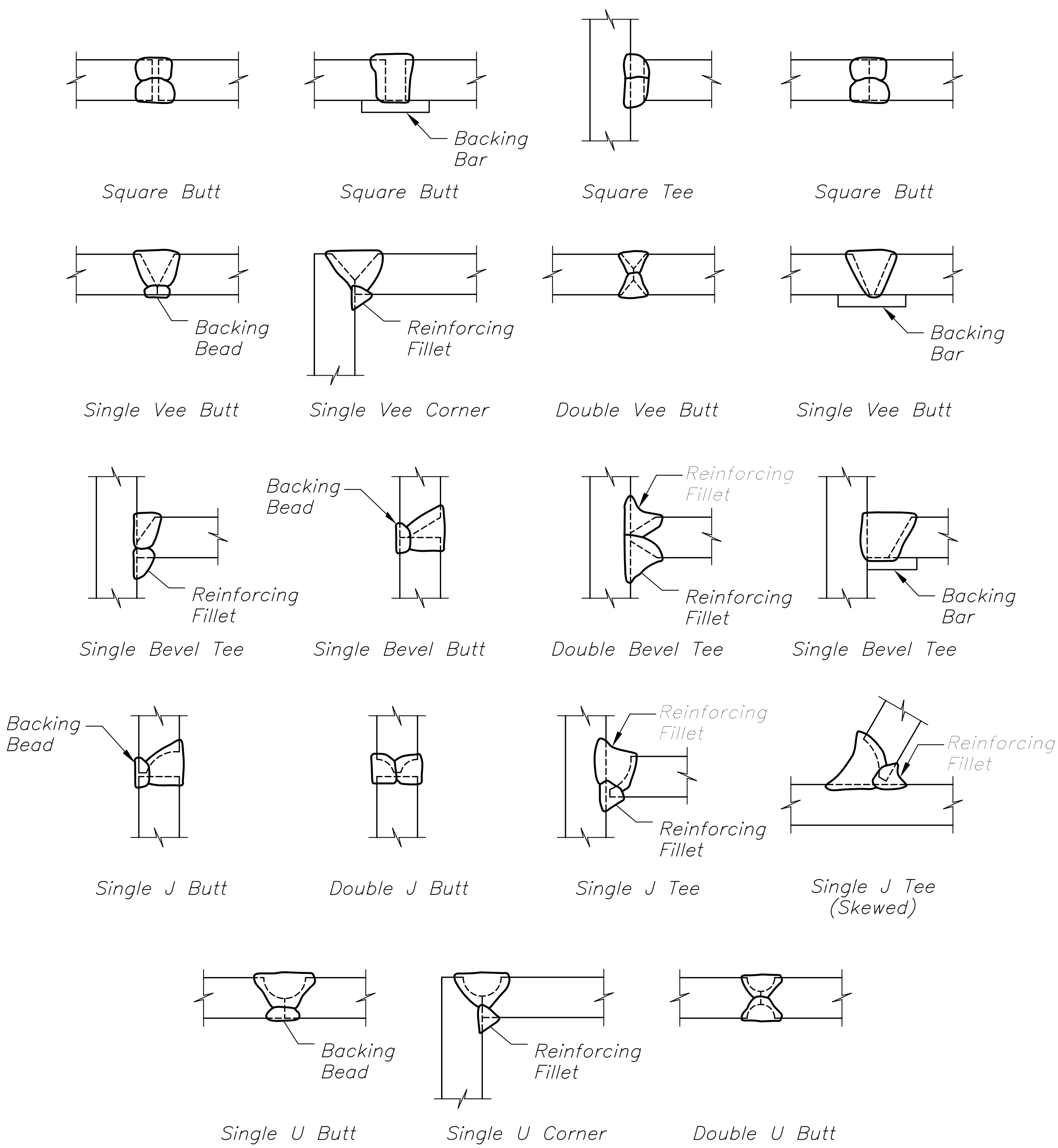


Figure 4-14. Typical complete-joint-penetration groove welds.

Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement, can be specified if weldability is a concern. Improperly welded bars have been known to crack and fall off during handling and shipping. AWS D1.4, *Welding Code—Reinforcing Steel*, offers information on proper welding technique and should be consulted.

Plug and Slot Welds

Plug and slot welds are used in lap joints to transmit shear loads, prevent buckling of lapped parts or join component parts of built-up members. Round holes or slots are punched or otherwise formed in one component of the joint before assembly. With parts in position weld metal is deposited in the openings, which may be filled partially or completely depending on the thickness of the punched material. Figure 4-16 illustrates slot welds used in conjunction with fillet welds to stitch a wide area of tee stem to a beam web. The same figure shows plug welds used to attach a guide strip to the top of a beam flange.

The proportions and spacing of slots and holes and the depth of weld are covered in detail by AISC *Specification* Section J2.3b, which stipulates minimum dimensions. These relationships are illustrated in Figure 4-17.

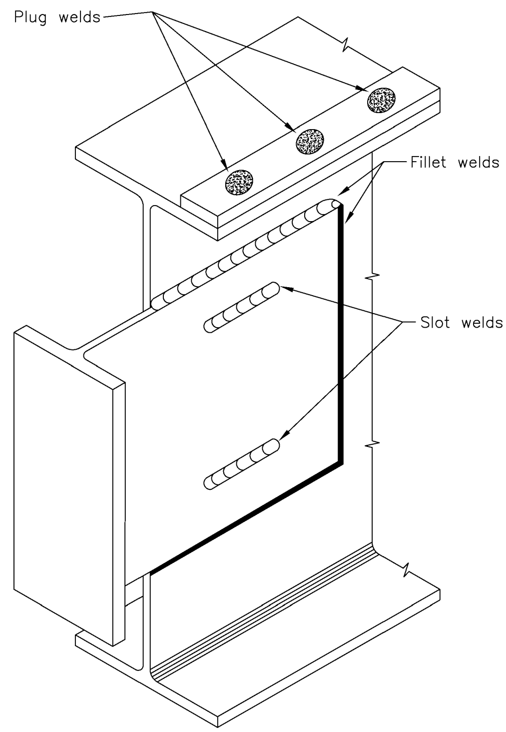


Figure 4-16. Slot welds used in conjunction with fillet welds to stitch a wide area of tee stem to a beam web.

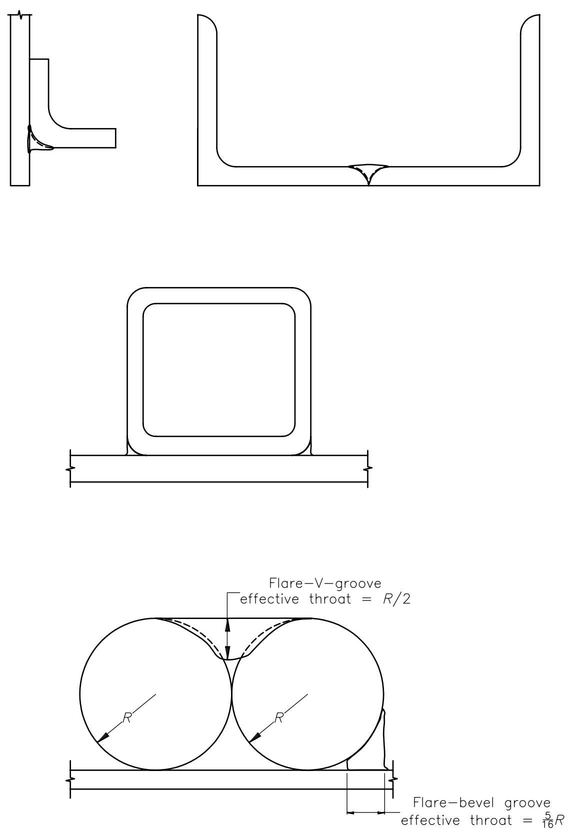


Figure 4-15. Several types of flare welded joints.

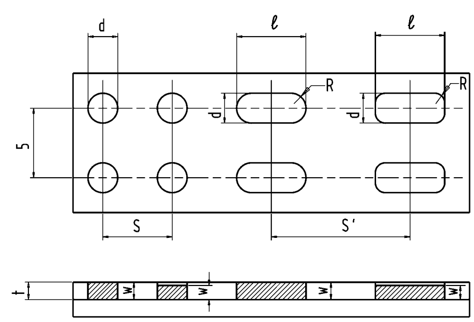


Plate thickness in.	Min hole diam. or slot width. d in.	Hole and slot proportions, spacing and depth of weld
$\frac{3}{16}$ & $\frac{1}{4}$	$\frac{9}{16}$	$d \geq (t+\frac{5}{16})$, round to next higher odd $\frac{1}{16}$; also $d \geq 2\frac{1}{4}w$
$\frac{5}{16}$ & $\frac{3}{8}$	$\frac{11}{16}$	$S \geq 4d$ $S' \geq 2\ell$ $\ell \leq 10w$ $R = \frac{d}{2}$ $R \geq t$ Where $t \leq \frac{5}{8}$, $W=t$ Where $t > \frac{5}{8}$, $W=\frac{t}{2}$, but not less than $\frac{5}{8}$
$\frac{7}{16}$ & $\frac{1}{2}$	$\frac{13}{16}$	
$\frac{9}{16}$ & $\frac{5}{8}$	$\frac{15}{16}$	

Figure 4-17. Proportions and spacing of slots and holes.

Fillet Welds in Holes and Slots

Fillet welds placed around the inside of a hole or slot are different from plug and slot welds. In the case of such fillet welds, the shear resistance is based on the effective throat area (i.e., the product of the throat size and the weld length measured along the line bisecting the throat area, as previously discussed and shown in Figure 3-17a). However, if the effective area determined in this manner should exceed the area of the hole or slot, rules applying to plug and slot welds must govern.

Fillet welds around the inside of a hole or slot require less weld metal than plug or slot welds of the same size. Note that the diameters of holes and widths of slots are somewhat larger than for plug and slot welds in the same thickness of metal. Fillet welds in a slot are easier to make and inspect and are usually preferred over fillet welds in round holes or plug welds. Recommended minimum hole diameters or slot widths, based on plate thickness for fillet welding, are shown in the table in Figure 4-17.

WELDING POSITIONS

The position of a joint when the welding is performed has a definite structural and economic significance. It affects the ease of making the weld, the size of electrode, the current required, and the thickness of each weld layer deposited in multi-pass welds. The following basic weld positions are shown in Figure 4-18:

- Flat: The face of the weld is approximately horizontal and welding is performed from above the joint.
- Horizontal: The axis of the weld is horizontal. For groove welds the face of the weld is approximately vertical; for fillet welds the face is usually at 45° to the horizontal and vertical surfaces.
- Vertical: The axis of the weld is approximately vertical.
- Overhead: Welding is performed from the underside of the joint.

AWS D1.1 prescribes limits of angular deviation from true horizontal and vertical planes for each of these weld positions. Special requirements for HSS and steel pipe welding may be found in AWS D1.1.

The flat position is preferred in all types of welding because weld metal can be deposited faster and easier. For example, a 5/16-in. manual fillet weld may require 50% more time to deposit in the horizontal position than in the flat position. Note that submerged arc welds generally are restricted to the flat position. Vertical and overhead welds may take three times as long as the same weld made in the flat position.

In the shop the work is usually positioned to permit flat or horizontal welding. This is done either by turning the work over (as when joining flat plates with welds on both sides) or by use of welding positioners, which tilt the work to a suitable position for flat or horizontal welding.

As field welding seldom permits positioning, vertical and overhead welds often cannot be avoided. However, careful planning in the drawing room can minimize the need for such welds by arranging field welded joints for flat or horizontal welding wherever possible. Figure 4-19a illustrates the placement of a single-vee groove weld with the face upward and a backing bar underneath to eliminate overhead welding.

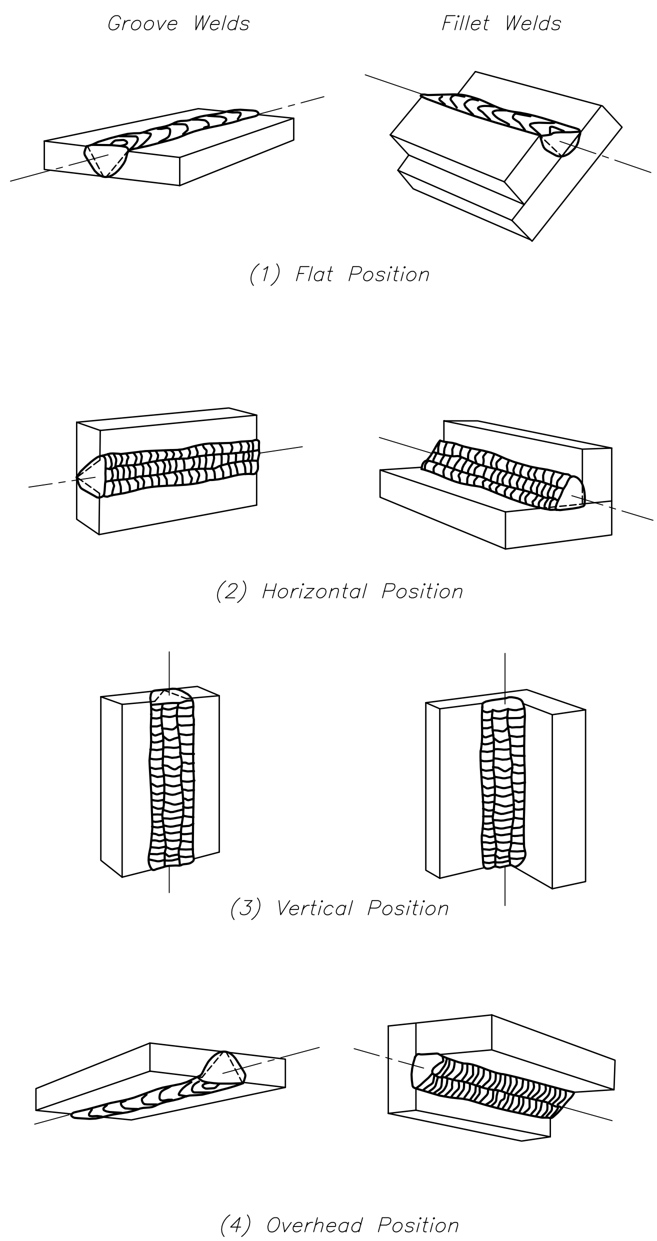


Figure 4-18. Basic weld positions.

Figure 4-19b shows a double-vee groove weld with an unsymmetrical profile, with the smaller groove placed on the bottom to reduce the amount of work in the overhead position.

ECONOMY IN SELECTION OF WELDS

In addition to detailing joints for the best welding position, the steel detailer can achieve economy by selecting welds that require a minimum amount of metal and can be deposited in the least amount of time. As shown in Chapter 3, the strength of a fillet weld is in direct proportion to its size. However, the volume of deposited metal and, hence the cost of the weld, increases as the square of the weld size. A $^5/_8$-in. fillet weld contains four times the volume required for a $^5/_{16}$-in. weld, yet it is only twice as strong. For this reason specifying a smaller, longer weld (rather than a more costly larger, shorter weld) is often preferable.

Cost control for manual fillet welds is usually based on the $^5/_{16}$-in. size. Welds of this size, and smaller, can be deposited in a single pass in most positions by the welding operator. Larger welds must be laid in multiple passes that require appreciably more time and weld metal at increased cost.

Double-bevel, double-vee, double-J and double-U groove welds often are more economical than single-bevel welds of the same type because the contained volume of weld metal is less. Nevertheless, where relatively thin edges are involved, single-bevel groove welds are more economical because of the smaller weld metal volume and the less costly edge preparation. As a rule, bevel and vee cuts can be flame-cut and, therefore, are less expensive than J- and U-grooves, which require planing or arc-air gouging. When joints can be groove welded in the flat position, the submerged arc, gas metal arc, or flux cored arc processes generally will be more economical then the shielded metal arc welding process.

WELDING SYMBOLS

Shop and erection drawings for welded construction must provide specific instructions for the type, size and length of welds and their locations on the assembled piece. This information is given usually by means of welding symbols. Structural fabricators generally follow the basic method described in the American Welding Society publication *Standard Symbols for Welding, Brazing and Nondestructive Examination*, AWS A2.4. The symbols in the system, commonly used in structural work, are shown in Figure 4-20.

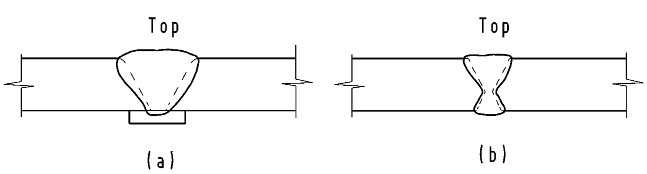


Figure 4-19. Vee and double-vee groove weld placement.

Several major rules and applications are given in the "Note" in the bottom part of Figure 4-20.

Three basic parts are needed to form a welding symbol: an arrow pointing to the joint, a reference line upon which the dimensional data is placed, and a basic weld symbol device indicating the weld type required. Examples of fillet welding symbols are shown in Figure 4-21, first as separate parts and then in assembled form.

A fourth part of the welding symbol, the tail, is used to supply necessary, additional data such as joint designation, specification, process or detail references. Welding symbols with tails containing typical references are shown in Figure 4-22. A reference note such as "Note A" in Figure 4-22 refers to a note elsewhere on the drawing that gives further instructions regarding the particular weld. An indication of specification references in the tail is necessary only when two or more electrode classes are required for the welding on a particular drawing. Normally, this information is carried in the general notes or on a job data sheet. The same is true for process references. However, as specification references usually determine the process, such references will be needed only for electrogas, electroslag, stud or other kinds of welding where the electrode specification does not describe the process or method. When the references are not needed to supplement the welding symbol, the tail is omitted.

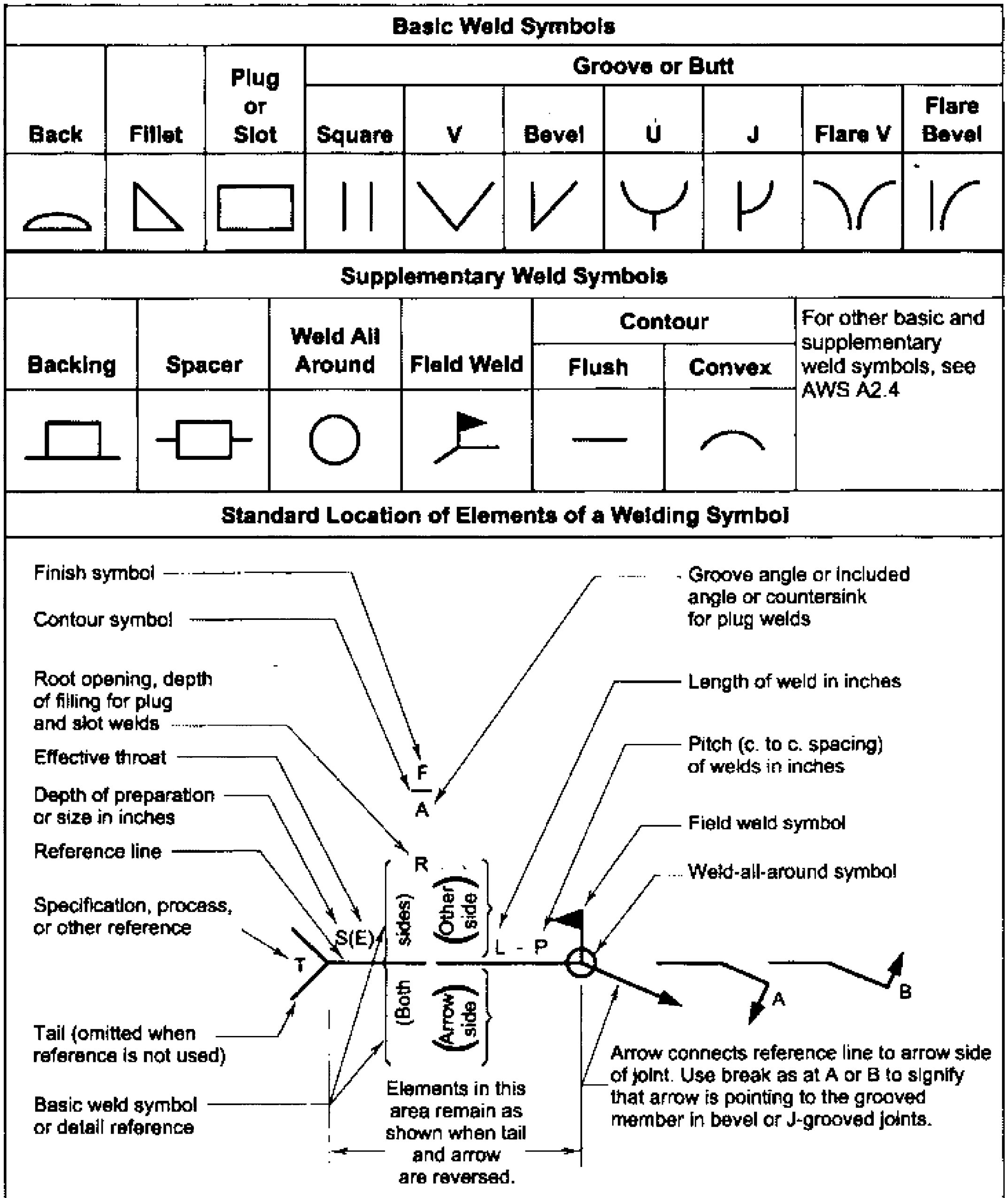
The symbols for welds should be made large enough to be understood and recognized easily. Some fabricators furnish welding symbol templates to their steel detailers. Templates may also be purchased from suppliers of drafting equipment.

Shop Fillet Welds

The basic weld symbol or device used to represent a fillet weld is an isosceles right triangle drawn with one of its equal legs on the reference line and with its hypotenuse always to the right of the vertical leg.

In fillet welds the joint is that portion of a surface common to the two parts to be connected. Thus, in each of the examples in Figure 4-23, area ABCD is the joint to be welded. Because fillet welds are deposited along the boundaries of a joint in the corner formed by the parts connected, the symbol arrow is drawn and pointing to and touching one or more of the boundary lines of the joint. When the boundary to be welded appears as a point in a particular view, the arrow is directed to this point.

When fillet welds are required on opposite sides of a joint, as is often the case, a single welding symbol is sufficient. The basic weld symbol is drawn as two right triangles on a common base line, one triangle on each side of the reference line. Thus, the welding symbols shown in plan, elevation and end view in Figure 4-24a each indicate that a fillet weld is required along both boundaries AD and BC of joint ABCD. In each case the arrow of the welding symbol points to only one of these two boundaries. BC is referred to as the arrow-side of the joint; AD is designated as the other-side.



Note:

Size, weld symbol, length of weld, and spacing must read in that order, from left to right, along the reference line. Neither orientation of reference nor location of the arrow alters this rule.

The perpendicular leg of ◺, V, ꓒ, ⌈, weld symbols must be at left.

Arrow and other side welds are of the same size unless otherwise shown. Dimensions of fillet welds must be shown on both the arrow side and the other side symbol.

The point of the field weld symbol must point toward the tail.

Symbols apply between abrupt changes in direction of welding unless governed by the "all around" symbol or otherwise dimensioned.

These symbols do not explicitly provide for the case that frequently occurs in structural work, where duplicate material (such as stiffeners) occurs on the far side of a web or gusset plate. The fabricating industry has adopted this convention: that when the billing of the detail material discloses the existence of a member on the far side as well as on the near side, the welding shown for the near side shall be duplicated on the far side.

Figure 4-20. Prequalified welded joints.

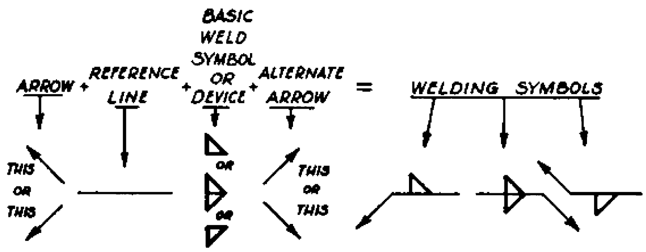


Figure 4-21. Fillet weld symbol.

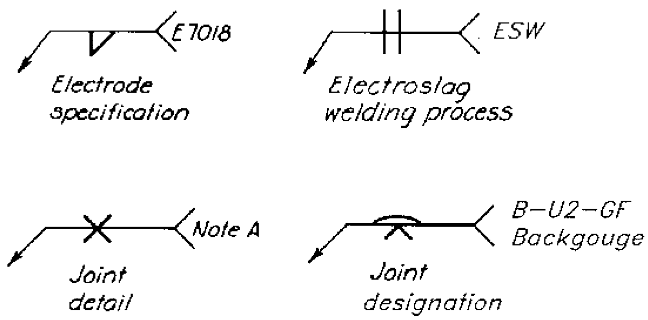


Figure 4-22. Weld symbols.

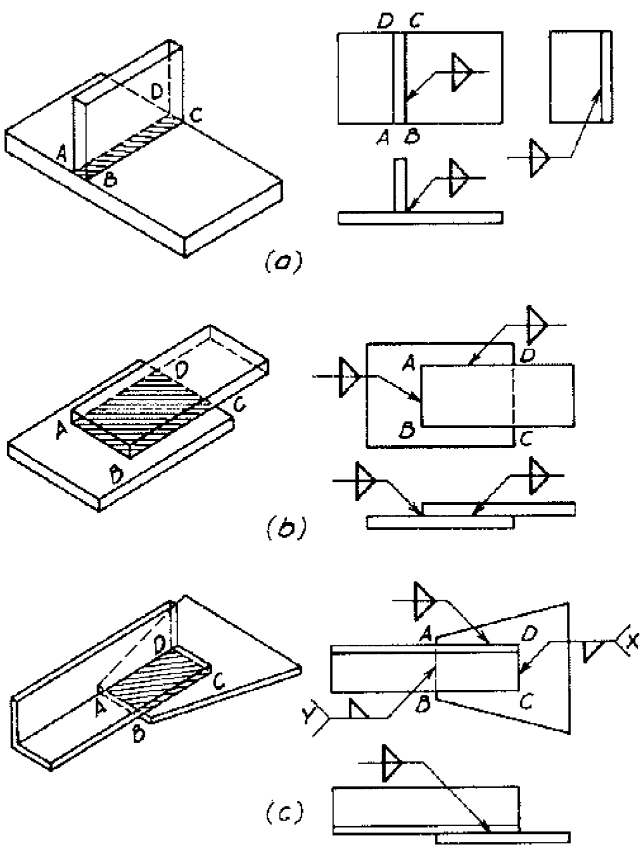


Figure 4-23. Fillet welds.

Note that the term "other-side" is used to denote the other side of the joint, not the far side of the assembly. In the lapped plates shown in Figure 4-24b, side DC, opposite the arrow-side AB, does appear to be on the far side of the assembly. However, it is in the plane of the joint and must be welded in accordance with the other-side symbol.

A single triangle drawn below the reference line (Figure 4-24a) indicates that a fillet weld is required on the arrow-side of the joint (see weld X pointing to boundary CD in Figure 4-24c). A single triangle drawn above the reference line (Figure 4-24b) indicates a fillet weld on the other-side of the joint opposite the arrow-side. In Figure 4-24c, the arrow for weld Y points to boundary AB, but the location of the triangle requires a weld along CD. Thus, welds X and Y are really the same weld. This duplication of welding symbols throughout Figure 4-23 is for illustrative purposes only. On a shop drawing only one of these symbols would be shown for each configuration.

Observe that reference lines and any information placed on them are arranged to read like other notes on a drawing: from left to right if the reference line is horizontal and from bottom to top if it is placed vertically on the sheet. Accepted steel detailing practice is to place reference lines in horizontal or vertical positions.

The arrow may be located at the right or left end of the reference line and may point upward or downward from it. The arrow is drawn at an angle of about 45° to the reference line, except when some other arrangement is necessary to avoid crowding a portion of the drawing. The arrowhead never should be placed on the reference line or on a continuous extension of the reference line. Some angular break should always be employed.

Weld dimensions are placed on the welding symbol with the weld size to the left of the basic weld symbol or device and the weld length (always in inches) to the right of it (Figure 4-24c). Weld dimensions are placed on the same side of the reference line as the device. Whether or not the arrow-side and other-side welds have the same size and dimensions, both must be dimensioned. This is a basic change in symbol construction that was instituted in 1976 and may not be evident on older drawings (Figure 4-24d).

Rarely, deviations from an equal leg fillet weld profile are necessary, in which case such welds are shown in cross-section and the legs dimensioned (Figure 4-24e). If a weld length is required, it is placed at the right of the basic weld symbol. Note that AWS provides a scheme whereby unequal leg fillets are dimensioned on the welding symbol. As the AWS method frequently requires a sketch to show orientation of the legs, fabricators generally prefer to dimension the sketch, rather than the symbol. This leads to a uniform practice that is less subject to error or misinterpretation.

If the fillet welds on a drawing are substantially of one size, the size may be omitted from the weld symbol and given

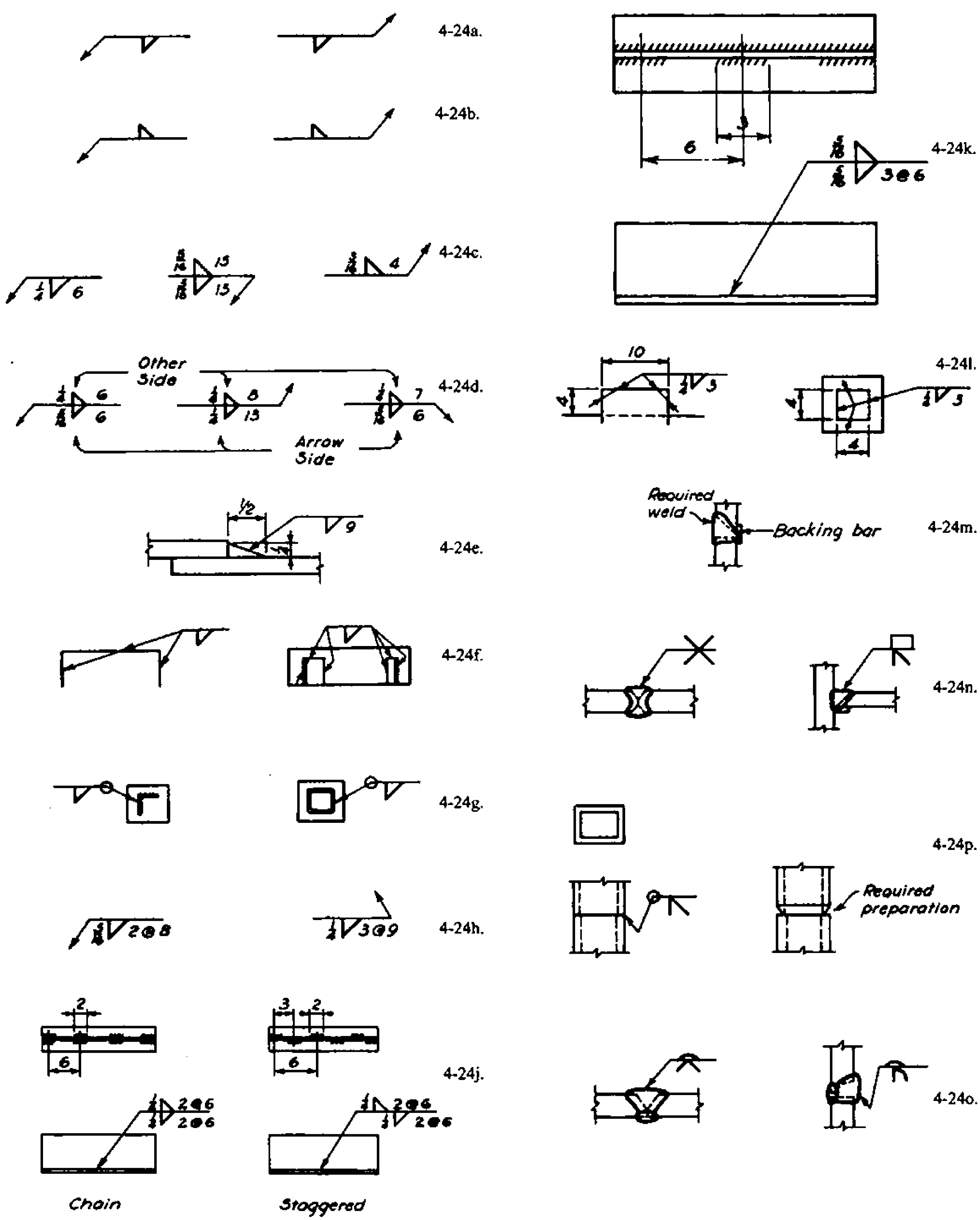


Figure 4-24. Miscellaneous weld symbols and details.

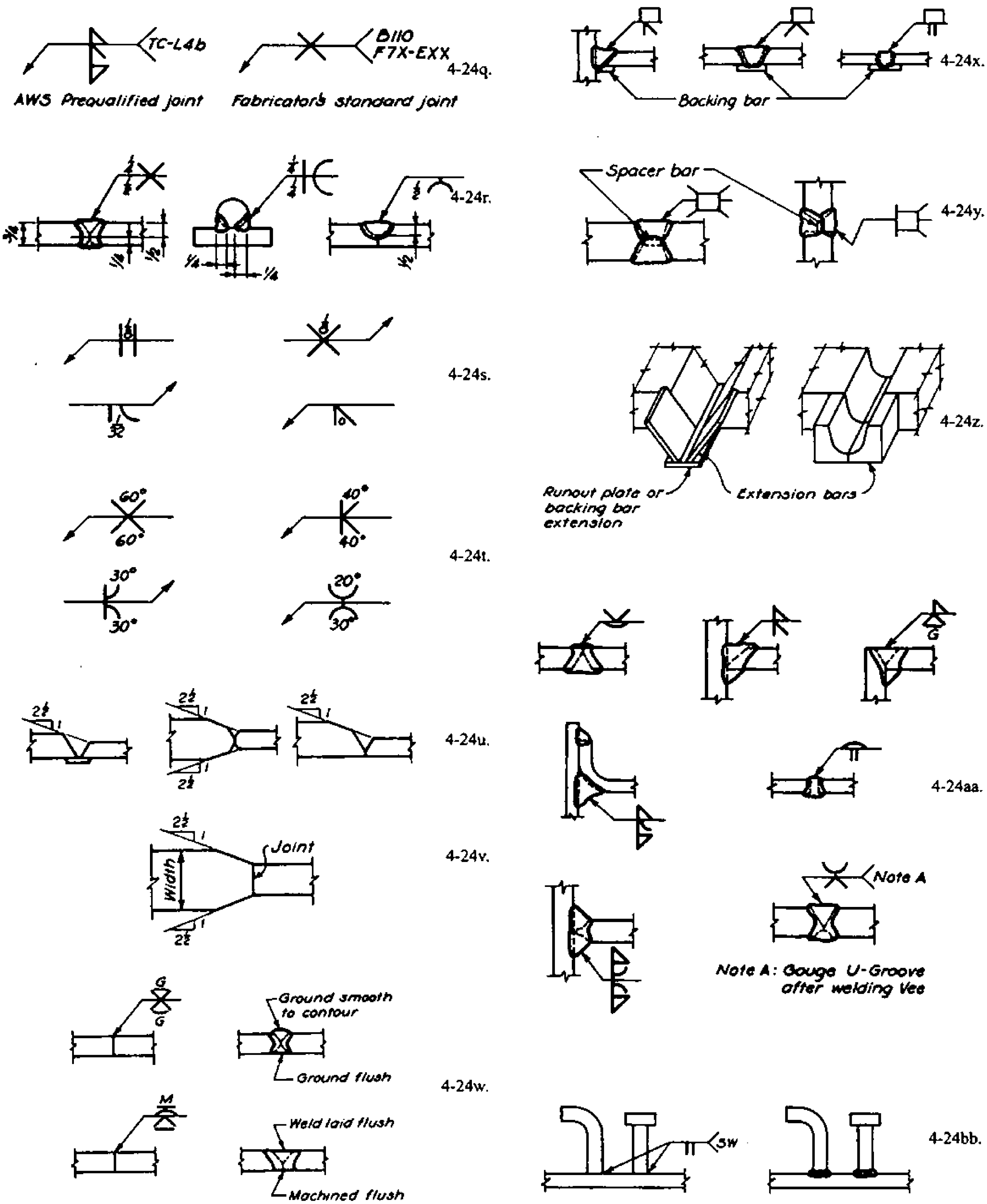


Figure 4-24 (continued). Miscellaneous weld symbols and details.

instead in the general notes. A note such as, "All fillet welds 5/16 unless noted otherwise" will relieve the steel detailer of showing the weld sizes, except where they deviate from the standard noted size.

When a weld is required along the entire length of one side of a joint, the length dimension is omitted from the welding symbol. This omission is understood to mean that the weld is to extend the full distance between abrupt changes in that part of the joint outline toward which the arrow points.

A single arrow pointing to one segment of the joint boundary also limits the use of the welding symbol to that particular segment. If the same welding symbol applies to two or more segments of a joint boundary, or even to boundaries of other joints nearby, sufficient arrows are used as required (Figure 4-24f). As shown, arrows can originate from one or both ends of the reference line.

If the same size fillet weld is required for the full length of all sides of a joint, the use of multiple arrows can be eliminated by employing a single arrow with an open circle placed at the juncture of arrow and reference line (Figure 4-24g). This is known as a weld-all-around symbol. The triangle is always placed to signify arrow-side welding, regardless of the shape or extent of the joint. This type of weld symbol should not be used indiscriminately, as it may cause excess welding and joint distortions. Do not signify an all-around weld if the entire perimeter of the piece cannot be accessed. Also, welding all around may violate the AISC *Specification* requirement that welds on opposite sides of a common plane be interrupted at the corner (i.e., when the weld would have to wrap around the corner at an overlap).

A required length of weld may be provided as a continuous weld or as an intermittent weld. A continuous weld is one

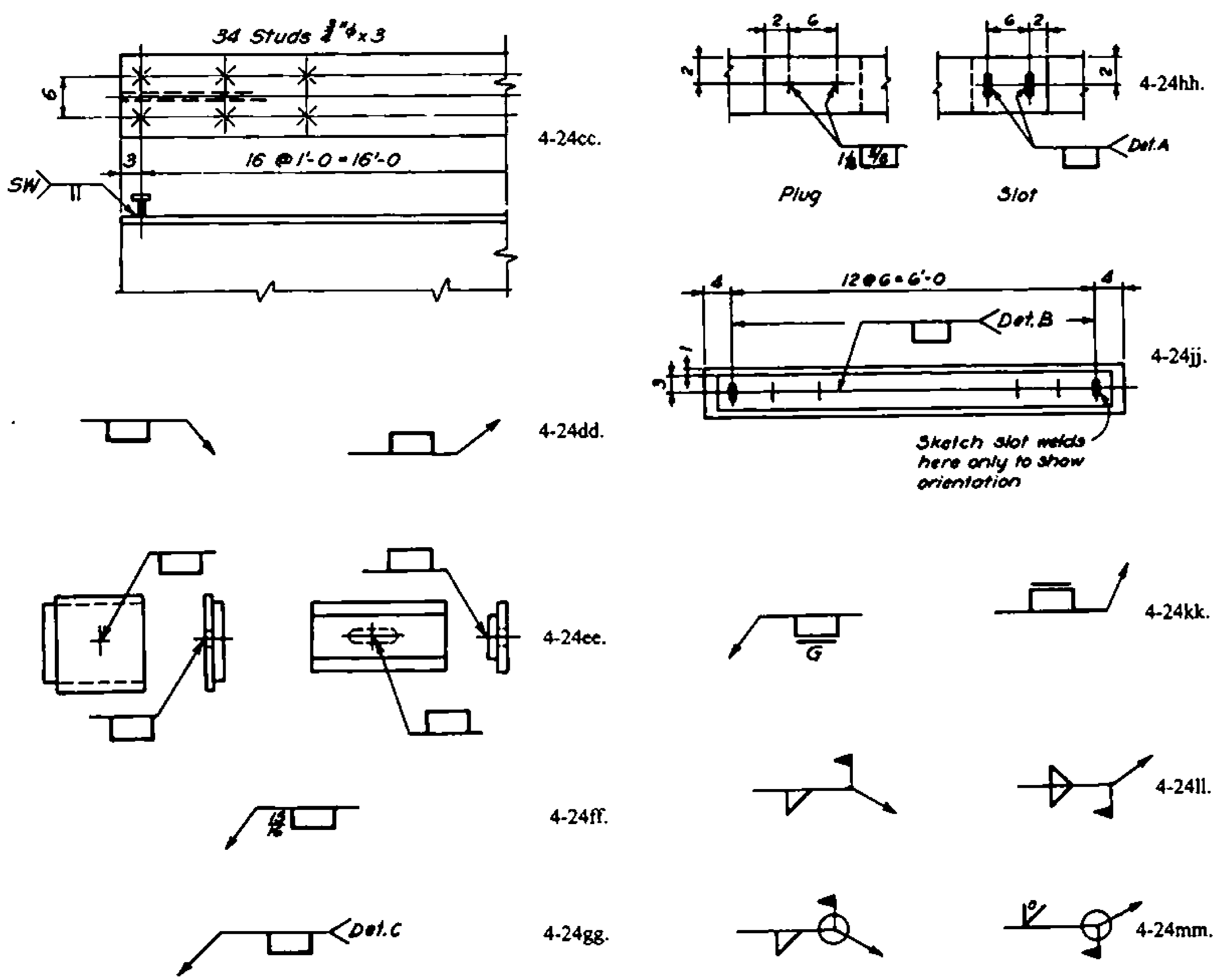


Figure 4-24 (continued). Miscellaneous weld symbols and details.

that is placed totally or partially along a joint boundary without interruption. An intermittent weld is one in which relatively short uniform lengths of weld are separated by regular spaces. Where intermittent welds are permitted, they are considered economical only when the pitch dimension (center-to-center distance between welds on the same side of a joint) is more than twice the weld length. Such welds sometimes are used when the smallest practicable continuous fillet weld would provide considerably more strength than required, or where stitching components of an assembly is required. Welding symbols for intermittent welds must, in addition to size, include the length of weld increments and their spacing or pitch (Figure 4-24h). The first number to the right of the triangle is the increment length of the weld; the second is the pitch, center-to-center of increments. These two dimensions are always given in inches and separated by a dash.

Where both arrow-side and other-side welding occurs, the weld increments may have either a chain or staggered relationship. In either case, the pitch dimension represents the center-to-center spacing on one side of the joint (Figure 4-24j). Note that the location of the triangles is meaningful. Arrow-side and other-side triangles with coinciding bases signify chain welding; those with offset bases call for staggered or alternate welding.

Where intermittent and continuous welding occur on either side of the same joint, the arrow-side and other-side triangles coincide with the necessary dimensions on both sides of the reference line (Figure 4-24k). The upper views shown in the illustrations are not required on a detail drawing. They are given here to illustrate the several combinations of spacing and also the requirement that a full weld increment must be placed at either end of a run, regardless of the intervening spacing. Intermittent welding is suitable only for the hand-guided welding processes, as the continual starting and stopping of the arc does not lend itself to automatic welding.

When the required length of a weld is less than the full distance between abrupt changes in the joint boundary or if more than one type of weld is to be shown along such a boundary, locating the ends of welds by dimension may be necessary (see Figure 4-25).

Where less than full length welds can be located by starting them at abrupt changes in the joint or if the actual placement of the weld is of no great consequence, the arrows usually can be placed so as to eliminate any need of dimensioning (Figure 4-24l). The four arrowheads in the left-hand sketch indicate that four 3-in. welds are required, each to be started at the corner of the 10-in. plate. The 3-in. welds in the right-hand sketch are to be centered on each 4-in. side.

A special practice in fillet welding symbols is required for "obscured" joints, such as the joints of connection angles, stiffeners, diagonals, etc., which occur in pairs on either side of a web. An example of this type of joint combination, shown in Figure 4-26a, is the framed connection at the end of a beam.

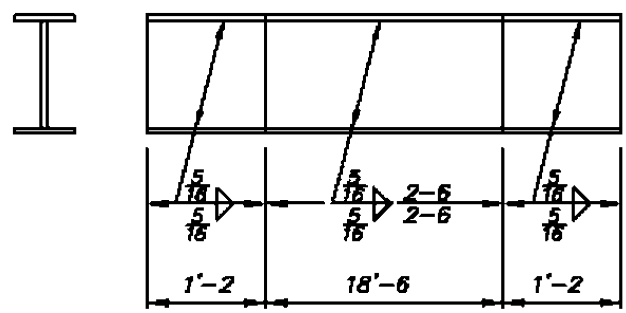


Figure 4-25. Welded joint.

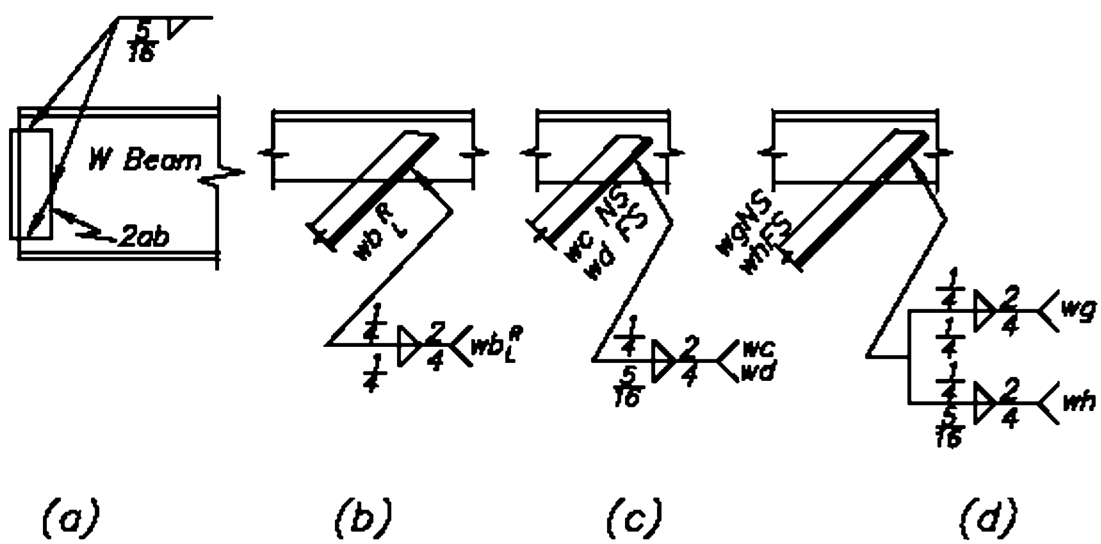


Figure 4-26. Welded joint combinations.

Where the boundaries of arrow-side and other-side joints coincide and the welding is the same for both joints, the welding symbol applied to the arrow-side is considered to apply also to the other-side joint. This practice holds for all similar joint combinations, some of which are shown in Figures 4-26b and 4-26c. However, in each case the presence of the obscured piece must be indicated clearly by the marking system identifying the members or by some other means.

If a difference exists in the welding for arrow-side and other-side pieces in an obscured connection, dual welding symbols may be provided. Figure 4-26d illustrates a dual welding symbol in which the weld locations are indicated by piece marks placed in the tails of the symbol.

Where a number of identical pieces are to be welded in an identical manner to a particular member, applying a single welding symbol to one of the pieces with the note “Typ” in the tail is sufficient. Should another one of such identically marked pieces require different welding, the exception is noted by an additional welding symbol. In Figure 4-27 a symbol noted “Typ” is shown applied (with one exception) to girder stiffeners sa. In an alternate method, the note “Typ” is omitted and a difference in welding requirements on identically prefabricated pieces is handled by assigning a different assembly mark and a separate welding symbol to the piece in question.

Special consideration is given to the case where a single-plate having square edges is welded to a main member on a skew. The *Manual* Table 8-2, Prequalified Welded Joints, Fillet Welds, 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 *Manual* Table 10-13. The steel detailer should consult with the fabricator.

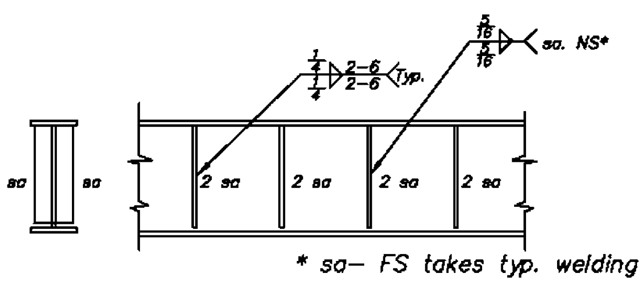


Figure 4-27. Typical welded joint.

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,

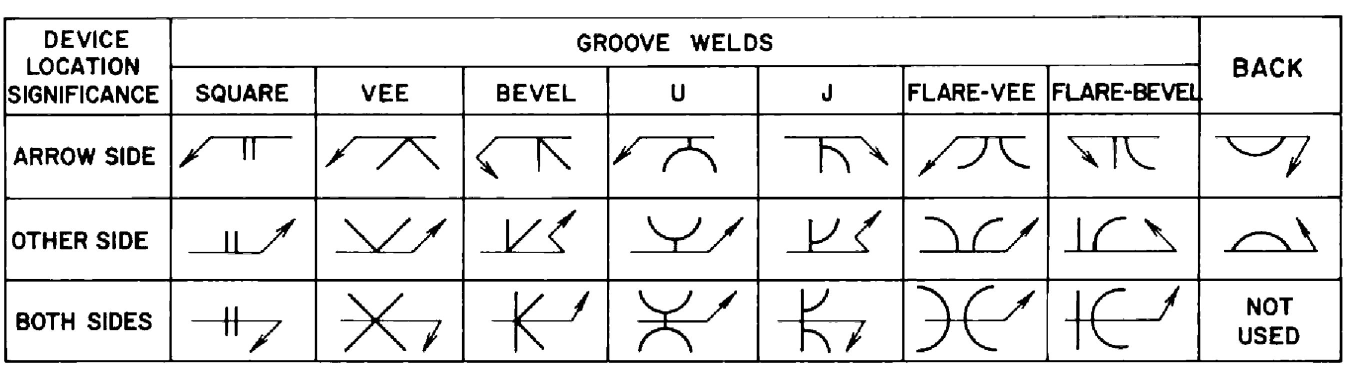


Figure 4-28. Groove weld symbols.

U and J welds when completing the second or root side of these welds is necessary (Figure 4-24p). Specifications for complete-joint-penetration groove welds require that the roots of shielded metal arc welds, gas metal arc welds, and flux cored arc welds be gouged to sound metal before the second or back weld is made. Submerged arc welds may or may not require gouging and back welds, depending on the prequalification established and project specifications.

Dimensioning groove welds involves weld size; root opening; groove angle of bevel, vee, J and U welds; and groove radii of J and U welds. Some fabricators have adopted their own standards for the proportions of groove welds generally used in structural work. These "user's standards" may be based on AWS prequalified welded joints or on other joints developed and qualified through AWS prescribed procedures. In either case the groove proportions and applicable welding processes are indicated in the tail of the symbol by a code that identifies the joint (Figure 4-24q).

Normally, the weld size of a complete-joint-penetration groove weld is understood to be the full thickness of the metal connected and its dimension need not be shown on the welding symbol. However, if the preparation of a double-groove weld is not symmetrical or if preparation will not provide for complete-joint-penetration welds, the size of welds must be shown (Figure 4-24r). Dimensions shown on the cross sections are illustrative only and need not appear on the drawing.

The root opening is shown near the root of the groove weld device (Figure 4-24s). The groove angle, in degrees, is shown within the groove faces of the device (Figure 4-24t). This angle is understood to be the total, or included, angle of the groove. Where arrow-side and other-side grooves are the same, the angle should appear on both sides of the reference line.

No provision for dimensioning groove radii of U and J welds is made on the weld symbol. Usually, this is covered by the fabricator's standard weld proportions or by reference to AWS prequalified joints. If not, it must be shown by note or sketch on the drawing.

The edge preparation for butt groove welds in tension joints involving elements of different thickness requires a transition slope which is not covered by the weld symbol. This is most conveniently shown by detail sketches (Figure 4-24u). Transitions between elements of different widths, subject to tension loads, also require fairing (Figure 4-24v). The 1 to $2^1/_2$ bevel represents the steepest slope permissible; flatter slopes are preferable. A transition with a 2-ft radius tangent to the narrower part of the center of the butt joint may be used also for unequal widths. For complete details, reference should be made to AWS D1.1.

The length of a groove weld is not placed on the symbol. The nature of grooved joint preparation is such that the weld boundary is from edge to edge of the parts joined. Any deviation from this will require special detailing with separate symbols for joint segments less than full joint width. Intermittent welds are not adaptable to groove welding; therefore, increment length and spacing will not appear on groove welding symbols.

Two supplementary weld symbols applicable to groove welds are the flush and convex contour symbols. These are used when the as-welded shape of the weld face is to be modified (Figure 4-24w). The letters shown generally will be the fabricator's standard designations for types of finish. Where the flush or convex symbol carries no finish designation, no finishing operation is required.

If complete-joint-penetration groove welds are required and welding can be done from one side of the joint only, backing bars must be provided (Figure 4-24x). Backing bars are penetrated thoroughly by the weld and often are left in place after welding is completed. Shop and erection drawings should be noted for their removal only if specifications require it to be done, as is required frequently in members subject to fatigue loading. Material used for backing must meet the requirements of AWS D1.1.

Spacer bars are used on certain double-vee and double-bevel welds, particularly if the weld is in thick material and using the minimum permissible bevel or vee angle is desired (Figure 4-24y). In such welds the root must be gouged out completely, including the spacer bar, before the second side of the groove is welded. The material for the spacer bar must be the same as that of the base material.

The weld symbol indicates the requirement for the backing or spacer bar by the appropriate symbol on the reference line (see Figure 4-20). Backing and spacer bar material should be identified and billed on the drawing.

To develop the full length of groove welds, and hence the full throat areas, extension bars are used to provide a continuation of the groove beyond the edges of the pieces joined. If backing or spacer bars are part of the joint detail, they, too, should be extended as shown in Figure 4-24z. The angle or contour of the extension bars must coincide with that of the groove. Project specifications may require that the extension bars and contained weld runout be cut flush with the edges of the joint components after welding is completed.

Whether or not the steel detailer should detail extension bars for shop use will depend on the fabricator's shop practice. However, when these items are needed for field welding, they must be detailed and furnished to the erector.

Groove weld symbols may be combined with each other, with a back weld symbol, with fillet weld symbols and with supplementary weld face contour symbols to produce a variety of weld configurations (Figure 4-24aa).

Combined groove welding symbols can become complicated and difficult to understand. The steel detailer is cautioned against developing symbols that, although technically correct, are so elaborate and cumbersome that their clarity is questionable. Unless the dimensions for such welds can be referred to AWS prequalified joints or to the fabricator's standard welds, it is strongly recommended that edge preparation and, if necessary, the weld itself be shown on the drawing by a dimensioned cross section.

Partial-Joint-Penetration Groove Welds

The preceding discussion on groove welds also applies, in general, to those welds that are not complete-joint-penetration groove welds. For these welds a description of the joint preparation dimensions is necessary. Also, the effective throat must be dimensioned (see Figure 3-17b), as it can be a variable.

AISC *Specification* Section J2.1a establishes the minimum effective area (throat thickness) of a partial-joint-penetration groove weld. Table 4-1 reproduces AISC *Specification* Table J2.3. Note that the minimum effective throat thickness is a function of the thickness of the thinner piece of material being joined.

AISC *Specification* Section. J2.1b further controls this weld type as indicated in AISC *Specification* Table J2.1 (reproduced here as Table 4-2).

Given the material thickness and weld process, these tables provide sufficient information to develop the edge preparation for a partial-joint-penetration groove weld. A somewhat similar approach is used to establish weld requirements for the flare weld, as shown in AISC *Specification* Table J2.2.

The weld symbol must specify the two elements of size S of groove preparation and the effective throat E of the weld, as well as all of the other usual elements. The symbol for a square groove weld requires only the E because no preparation is required. Examples are shown in Figure 4-29.

Figure 4-29b shows that the effective throat of the SMAW weld is the same as the depth of groove because the groove angle is 60° for the arrow-side weld. When the other-side groove angle is changed to 45°, the effective weld equals the depth of chamfer minus 1/8 in. (see Table 4-2).

Intermittent partial-joint-penetration groove welds are sometimes used in joining plates in built-up members. The preparation of the joint profile in intermittent partial-joint-penetration groove welds also must consider a transition or "fairing in" of the joint at its beginning and termination in

Table 4-1. Minimum Effective Throat Thickness of Partial-Joint-Penetration Groove Welds (AISC *Specification* Table J2.3)

Material Thickness of Thinner Part Joined (in.)	Minimum Effective* Throat Thickness (in.)
To 1/4 inclusive	1/8
Over 1/4 to 1/2	3/16
Over 1/2 to 3/4	1/4
Over 3/4 to 1 1/2	5/16
Over 1 1/2 to 2 1/4	3/8
Over 2 1/4 to 6	1/2
Over 6	5/8

*See AISC *Specification* Section J2.1b.

Table 4-2. Effective Throat Thickness of Partial-Joint-Penetration Groove Welds (AISC *Specification* Table J2.1)

Welding Process	Welding Position F (flat), H (horiz.), V (vert.), OH (overhead)	Groove Type (AWS D1.1, Figure 3.3)	Effective Throat
Shielded Metal Arc (SMAW)	All	J or U Groove 60° V	Depth of Groove
Gas Metal Arc (GMAW) Flux Cored Arc (FCAW)	All		
Submerged Arc (SAW)	F	J or U Groove 60° Bevel or V	
Gas Metal Arc (GMAW) Flux Cored Arc (FCAW)	F, H	45° Bevel	Depth of Groove
Shielded Metal Arc (SMAW)	All	45° Bevel	Depth of Groove Minus 1/8 in. (3mm)
Gas Metal Arc (GMAW) Flux Cored Arc (FCAW)	V, OH	45° Bevel	Depth of Groove Minus 1/8 in. (3mm)

order for the weld to make proper fusion with the base 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 to dynamic or cyclic loading, except as noted earlier 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

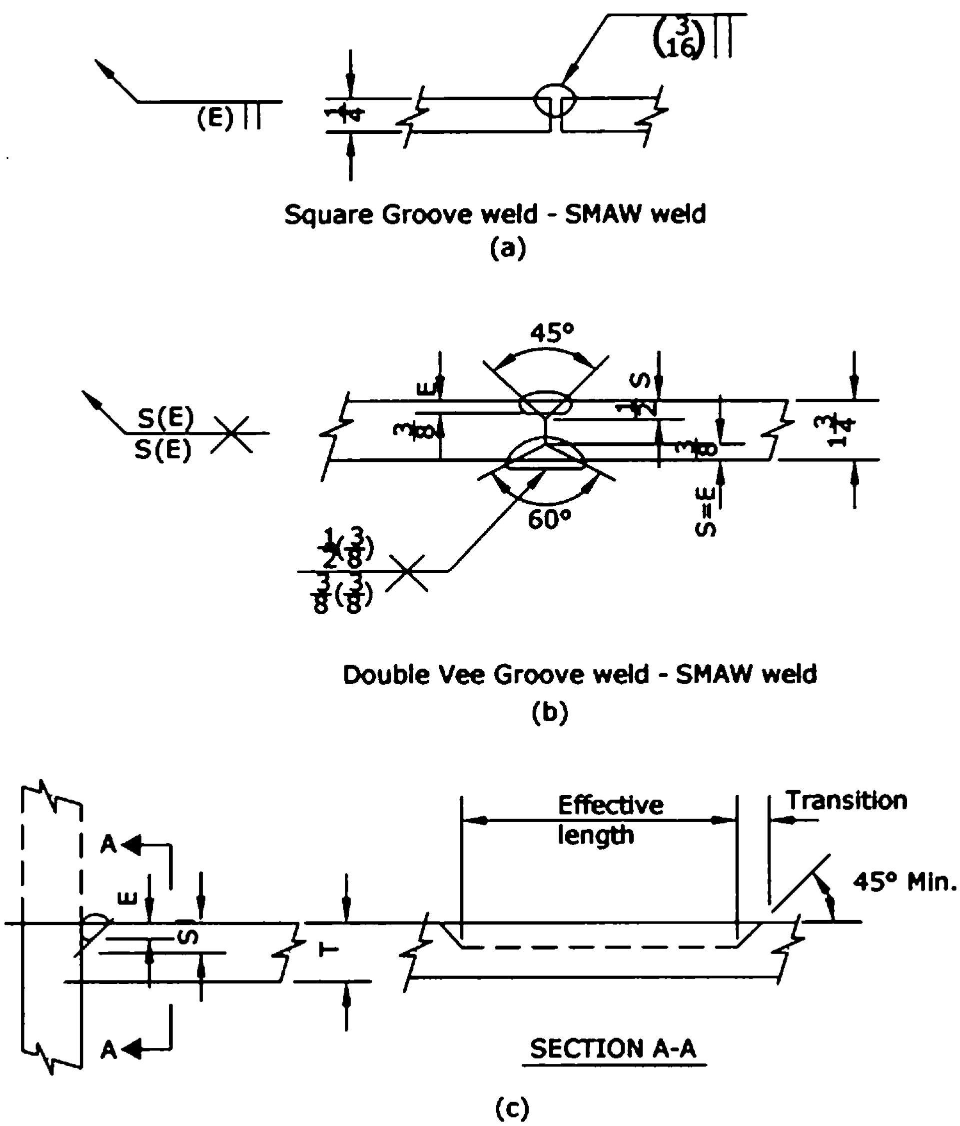


Figure 4-29. Groove welds.

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 parentheses 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 that 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 dimensioning for plug and slot welds than is recommended earlier. Nevertheless, structural shop experience has shown that dimensions appearing on the symbol are subject to misinterpretation. Fewer problems are experienced when location, spacing and special chamfering are spelled out at greater length as recommended here.

Field Welds

Symbols for field welding of fillet, groove, stud, plug and slot welds are identical to those for shop welding, except that the welding symbol is identified by a small blackened extended flag placed at the juncture of the arrow and reference line. The point of the flag must point toward the basic weld symbol (Figure 4-24ll).

The only other supplementary symbol that appears at this junction point is the weld-all-around symbol, which may be used in conjunction with the field welding symbol (Figure 4-24mm). Field welding symbols should appear only on erection drawings or on sheets showing field alterations to an existing structure or other work to be performed away from the fabricating plant. All edge preparation required for field groove welds is done in the shop and must be detailed or noted on the shop drawings. Field weld symbols are not placed on shop drawings because the instructions they convey will not be recognized or acted upon by shop personnel.

The flag symbol was adopted by AWS in 1976 to conform to international practice. The previous field weld symbol was a large round blackened circle. This symbol will be found on older structural drawings and references.

NONDESTRUCTIVE TESTING SYMBOLS

AWS D1.1 requires that all welds be visually inspected. Frequently, the quality of a weld is required to be nondestructively tested as outlined in AWS D1.1 Section 6—Inspection. When a specific weld or part thereof is required to be inspected (e.g., a tension butt splice of a girder flange plate), common practice is to identify the weld and the required type of inspection on the shop detail drawings.

Basic symbols for this nondestructive testing have been developed by AWS and are published in their publication, *Symbols for Welding, Brazing and Nondestructive Examination*, AWS A2.4. Listed here are the five most commonly used testing methods for structural steel welding inspection:

TYPE OF TEST	SYMBOL
Dye Penetrant	PT
Magnetic Particle	MT
Radiographic	RT
Ultrasonic	UT
Visual	VT

The testing symbol is built up in the same manner as the welding symbol. It is made up of the following elements:

- Reference line
- Arrow
- Basic testing symbol
- Test-all-around symbol
- Tail
- Specification or other reference

The elements of the testing symbol have standard locations with respect to each other (Figure 4-30a). The arrow connects the reference line to the part to be inspected and retains the same arrow-side, other-side significance as discussed earlier for the weld symbol. Typical examples are shown in Figure 4-30b. The nondestructive testing symbols are combined with the welding symbol as shown in Figure 4-30c.

The extent of nondestructive testing can be shown in the following ways (see Figure 4-30):

- By a dimension following the basic testing symbol.
- As a percentage of the part or weld, shown following the basic symbol.
- By a dimensioned sketch or drawing.

If no limit is indicated, the testing applies to the full length of the indicated weld or part.

OTHER WELDING AND TESTING SYMBOLS

The foregoing discussion of weld and testing symbols does not cover every possible combination or application that may be encountered on design drawings or that may arise in the preparation of shop drawings. Treatment has been limited to the more typical welding situations met in structural work. For a full development of welding and nondestructive testing symbols for almost every conceivable structural requirement, see AWS *Symbols for Welding, Brazing and Nondestructive Examination*, AWS A2.4. This reference also will aid in identifying and determining the meaning of other weld symbols not normally used in structural work.

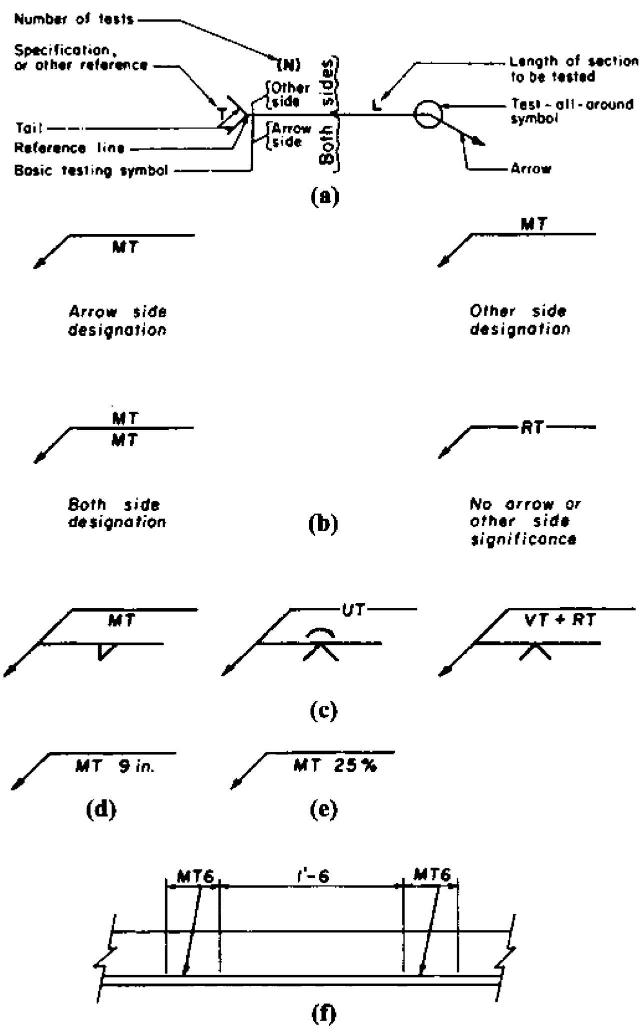


Figure 4-30. Nondestructive testing symbols combined with welding symbols.

PAINTING

The contract documents must specify all the painting requirements, including the identification of the members to be painted, surface preparation, paint specifications, manufacturer's product identification, and the required minimum dry film thickness (in mils) of the shop coat. Factors in the selection of a paint system include the owner's preference, intended service life of the structure, severity of environmental exposure, cost of both the initial application and of future renewals, and compatibility of the various components comprising the paint system. Furthermore, the documents must indicate clearly all individual members that are to be left unpainted so as to receive concrete, sprayed-on fireproofing or for other reasons. Because the inspection of shop painting must be concerned with workmanship at each stage of the operation, the fabricator provides notice of the schedule of operations and affords the Inspector access to the work site. Inspection must then be coordinated with that schedule so as to avoid delay of the scheduled operations.

Acceptance of the prepared surface must be made prior to the application of the shop coat because the degree of surface preparation cannot be readily verified after painting. Time delay between surface preparation and the application of the shop coat can result in unacceptable deterioration of a properly prepared surface, necessitating a repetition of surface preparation. This is especially true with blast-cleaned surfaces. Therefore, to avoid potential deterioration of the surface, it is assumed that surface preparation is accepted unless it is inspected and rejected prior to the scheduled application of the shop coat.

The shop coat in any paint system is designed to maximize the wetting and adherence characteristics of the paint, usually at the expense of its weathering capabilities. Deterioration of the shop coat normally begins immediately after exposure to the elements and worsens as the duration of exposure is extended. Consequently, extended exposure of the shop coat will likely lead to its deterioration and may necessitate repair, possibly including the repetition of surface preparation and shop coat application in limited areas. With the introduction of high-performance paint systems, avoiding delay in the application of the shop coat has become more critical. High-performance paint systems generally require a greater degree of surface preparation, as well as early application of weathering protection for the shop coat.

Since the fabricator does not control the selection of the paint system, the compatibility of the various components of the total paint system, or the length of exposure of the shop coat, the fabricator cannot guarantee the performance of the shop coat or any other part of the system. Instead, the fabricator is responsible only for accomplishing the specified surface preparation and for applying the shop coat (or coats) in accordance with the contract documents.

In specifying surface preparation, paint specifications, and dry film thickness, the contract documents usually refer to the Society for Protective Coatings (SSPC). This organization publishes recommended protective coating systems for steel used in the many types of structures under a wide variety of atmospheric conditions. Reference is made to a code number (SP2, SP3, SP4, etc.) of the SSPC, which describes the minimum requirements for cleaning the steel (from simple wire brushing and sweeping to elaborate blast cleaning operations) and the type(s) of paint appropriate for the protection specified. Some priming systems require the application of more than one coat of paint in the shop, each coat being a different, compatible paint. At the election of the fabricator and unless specifically excluded, paint is applied by brush, spray, roller coating, flow coating or dipping. During handling and shipping of the painted members to the job site, abrasions can be expected. Touch-up of these blemishes is the responsibility of the contractor performing field touch-up or field painting. Most steel contractors will exclude field touch-up and the required paint in their bid package.

Even if contract documents do not specify a paint system, the documents normally require submittal and approval of the fabricator's basic paint system. On each shop drawing, the steel detailer notes whether painting of the pieces on that sheet is required. Often the preprinted sheet will have a block of information near the title block for the steel detailer to complete. Usually, this block includes information on painting and cleaning. The steel detailer completes the block by indicating whether or not paint is required. If paint is required that is different from the fabricator's standard, the steel detailer will add the SSPC identification such as SP 3, SP 6, etc. (see Figure A4-31 in Appendix A).

On projects where the steel is given a shop coat of paint, usually three types of cases may be encountered where paint must be omitted:

If connections require using ASTM A325 or A490 bolts (or equivalent alternative fasteners), in noncoated slip-critical joints, paint on faying surfaces shall be excluded from areas closer than one bolt diameter but not less than 1 in. from the edge of any hole and in all areas within the bolt group pattern. See RCSC *Specification* Section 3.2.2. (Note that joints specified to have painted faying surfaces must be blast cleaned and coated with a paint that has been qualified as Class A or Class B in accordance with the RCSC *Specification*.) The drawing, typically, will have a note specifying the areas where paint is to be excluded. For example, for a typical pattern of 1-in. bolts, such a note might read "No paint within 1 in. of the perimeter of the bolt hole group." If the condition is typical for the several pieces on the drawing, the note will appear with the General Notes for the drawing. If the requirement for omitting paint around the bolt hole groups is infrequent on a drawing, the note can be placed adjacent to each affected piece. Further, if only certain holes in the piece require the note, these holes will be encircled and noted. The members connecting to these holes, also, must be noted for no paint. Finally, if the "no paint" requirement applies to the outstanding legs of connection angles, the note may read "No Paint on OSL of Connection Angles."

Paint is omitted in areas where field welding occurs. Primarily, these areas include field connections and the tops of members requiring the field application of shear studs. AISC *Specification* Section M3.5 requires that, unless otherwise provided in the design drawings and specifications, surfaces within 2 in. of any field weld be free of materials that would prevent proper welding or produce toxic fumes while welding is being done. Figure A4-6 in Appendix A illustrates one method of noting paint omission. Where small areas are involved, many fabricators prefer to use the general note, "No Paint on OSL of Connection Angles." The supporting member also must be noted for "no paint" in the location of the field welded connection. The "no paint" requirement does not apply to shop welding, because shop painting is done, normally, after the welds are made. Special, costly consideration is required when shop contact surfaces are required to be painted. For additional examples, see Figures A4-32 and A4-33 in Appendix A. An exception to the foregoing exists in the case of some paints, which are formulated such that they do not impair the quality of the weld or emit toxic fumes when welding is done. The fabricator must advise the steel detailer when this is the case.

If the steel is to receive sprayed-on fireproofing, typically the steel should not be painted. Otherwise, the sprayed-on fireproofing may not meet the adhesion requirements of the Underwriters Laboratories (UL). Again, the shop drawing will note these pieces "no paint."

GALVANIZING

Galvanizing is the name given to the process of applying a protective coat of zinc to steel products where protection of the surface from corrosion is required. The process is used extensively for transmission towers; electrical substation and switching structures; and corrugated sheets for roofing, siding, flashing, highway guard rail, etc. The zinc coating is applied usually by the hot dip process. This involves dipping the material in a series of cleansing and rinsing tanks (a process called pickling) to thoroughly clean the steel before dipping it into a tank of molten zinc, which is at a temperature of approximately 840 °F. The steel must remain in the tank long enough to bring its temperature up to that of the molten zinc. Following the dip in the zinc, the steel is removed and quenched in plain water. The quenching operation retains as much bright zinc surface as possible and reduces the time needed for handling. This method deposits a coating of about 2 oz. of zinc per square foot of surface.

The first requisite in galvanizing is proper preparation of the material. All surfaces must be free of paint, grease, oil, dirt, mill scale and weld slag. Galvanizing of structural members must meet the requirements of ASTM Specification A123, *Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products*. However, because they require removal of excess zinc, bolts, nuts and washers are governed by ASTM Specification A153, *Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware*.

The following checklist is presented to identify the most common considerations for the steel detailer to use when preparing shop drawings for galvanized material:

- Identification of galvanized material on the shop drawings
- Maximum size that can be galvanized
- Bolted connections
- Welded connections
- Seal welding requirements
- Use of galvanized bolts
- Drain and vent holes
- Field welding precautions

Preferably, pieces to be galvanized should be detailed on drawings separate from other fabrication. Instead of noting the paint and cleaning requirements, the steel detailer would note "GALVANIZE." When pieces to be galvanized are shown on the same shop drawing as pieces not to be galvanized, the sketches of those to be galvanized are noted "GALVANIZE." An entry (such as "GALV") is made in the "Remarks" column of the shop bill on the same line as the description of each piece to be galvanized. For purposes of this presentation, "pieces" can either be individual members (such as beams, columns and fittings) or shop welded assemblies (such as handrail and trusses). Steel members to be galvanized cannot have any of the customary markings applied by paint, crayon or chalk (markings will be removed during the cleansing operations prior to galvanizing). Thus, fabricators generally mark the individual pieces in one of two ways, although other means of marking are used to a lesser extent:

1. Provide erection or other identifying marks by stamping them into the pieces prior to galvanizing, using characters at least $^1/_2$ in. high and making an imprint at least $^1/_{16}$ in. deep. Marks should consist of as few characters as possible.
2. Identify pieces to be galvanized by attaching to each piece a metal tag with the mark stamped into it. Care must be exercised by the fabricator in securely fixing these tags to the pieces so that they do not become lost during handling, shipping or galvanizing.

Whichever method of maintaining identification is used depends on the fabricator's preference and does not affect the preparation of shop drawings.

Before detailing pieces for galvanizing, the steel detailer must know the sizes of the pieces that the galvanizer can handle. This depends on the size of the zinc tanks, the tanks in the cleaning line, the crane capacities and the building structure clearances within a particular galvanizing plant. The most economical and highest quality galvanizing is achieved when steel members are sized to enable the member to be immersed totally in the molten zinc in a single dip. If steel members are too large, they must be "double-dipped," which adds considerably to the cost. In this method 50% or more of the surface of the piece is immersed in the molten zinc. When the galvanizing of that portion is completed, the piece is repositioned so that the remaining uncoated portion can be lowered into the zinc and galvanized. The steel detailer should obtain the information about appropriate sizes from the fabricator, who will arrange for galvanizing of the fabricated pieces.

Attachments are not bolted to shipping pieces in the shop before galvanizing. Usually, they are galvanized separately and shipped loose for field bolting using galvanized bolts. Sometimes, the pieces are returned to the fabricator for assembly. Neither holes nor slots need be increased in size to accommodate galvanized bolts.

The American Galvanizers Association publishes two useful guides on design and detailing for galvanizing: *Design of Products to be Hot-Dip Galvanized after Fabrication* and *Recommended Details for Hot-Dip Galvanized Structures*.

Galvanizing welded joints is a major concern. The concern is the seepage of the pickling solution, which is part of the cleaning operation, into a welded joint. After galvanizing, the solution will bleed out of the joint, causing zinc removal and objectionable stains. To prevent the seepage, joints may be sealed by continuous welds all around the joint and the shop drawing so noted. Such a note would be, "All welded joints on galvanized pieces to be sealed."

The American Galvanizers Association (AGA) recognizes the potential for explosions to occur if trapped pickling solution vaporizes and expands when the part is dipped into the hot galvanizing bath. This can happen as the result of a weld containing porosity. AGA states in its design guidelines that, "Pinholes from welding are very dangerous in items to be galvanized and must be avoided." Thus, the AGA guidelines furnish information as to when venting is appropriate and how to provide it. Before considering the welding of any joints

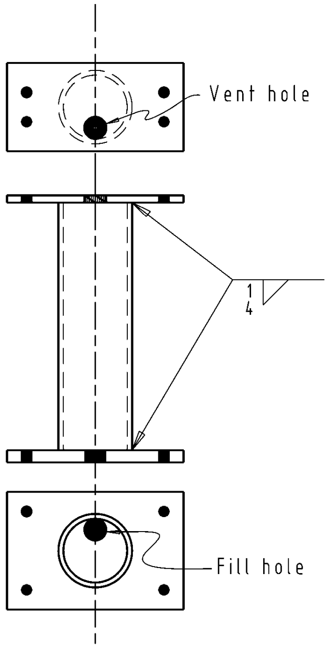


Figure 4-34. Tubular post with plates welded to each end.

on members to be galvanized, the steel detailer should receive instructions from the fabricator.

To the greatest extent possible, enclosed spaces should be avoided. If enclosed spaces are unavoidable, such as in a tubular post with plates welded to each end (Figure 4-34), fill/vent holes must be added to each end of the piece to permit entry of cleaning fluids and zinc during dipping (to overcome the natural buoyancy of hollow pieces in the tanks) and elimination of pockets of zinc as the piece is withdrawn from the galvanizing bath. Furthermore, the holes allow cleaning fluids and molten zinc to reach internal surfaces so that they can be cleaned and coated completely. The sizes of holes recommended vary with the size of shape being galvanized and are available in literature from the galvanizer. These holes may be drilled or burned by the fabricator. The steel detailer must include these holes in the details of the pieces.

In addition, holes are required to drain areas where cleaning fluids and molten zinc would become trapped unless the piece were turned to permit draining, which is an added cost. A typical situation occurs when a base and cap plate are welded to the ends of a W-shape. Usually, a semicircular hole (called a "weld access hole" or sometimes referred to as a

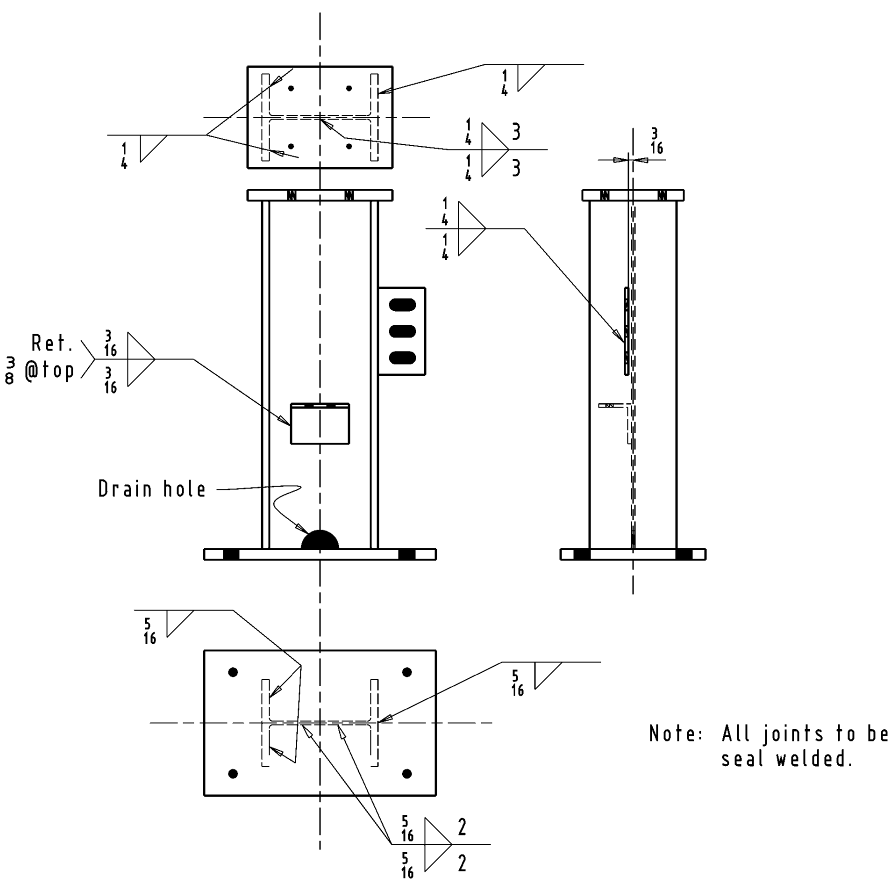


Figure 4-35. Weld access holes.

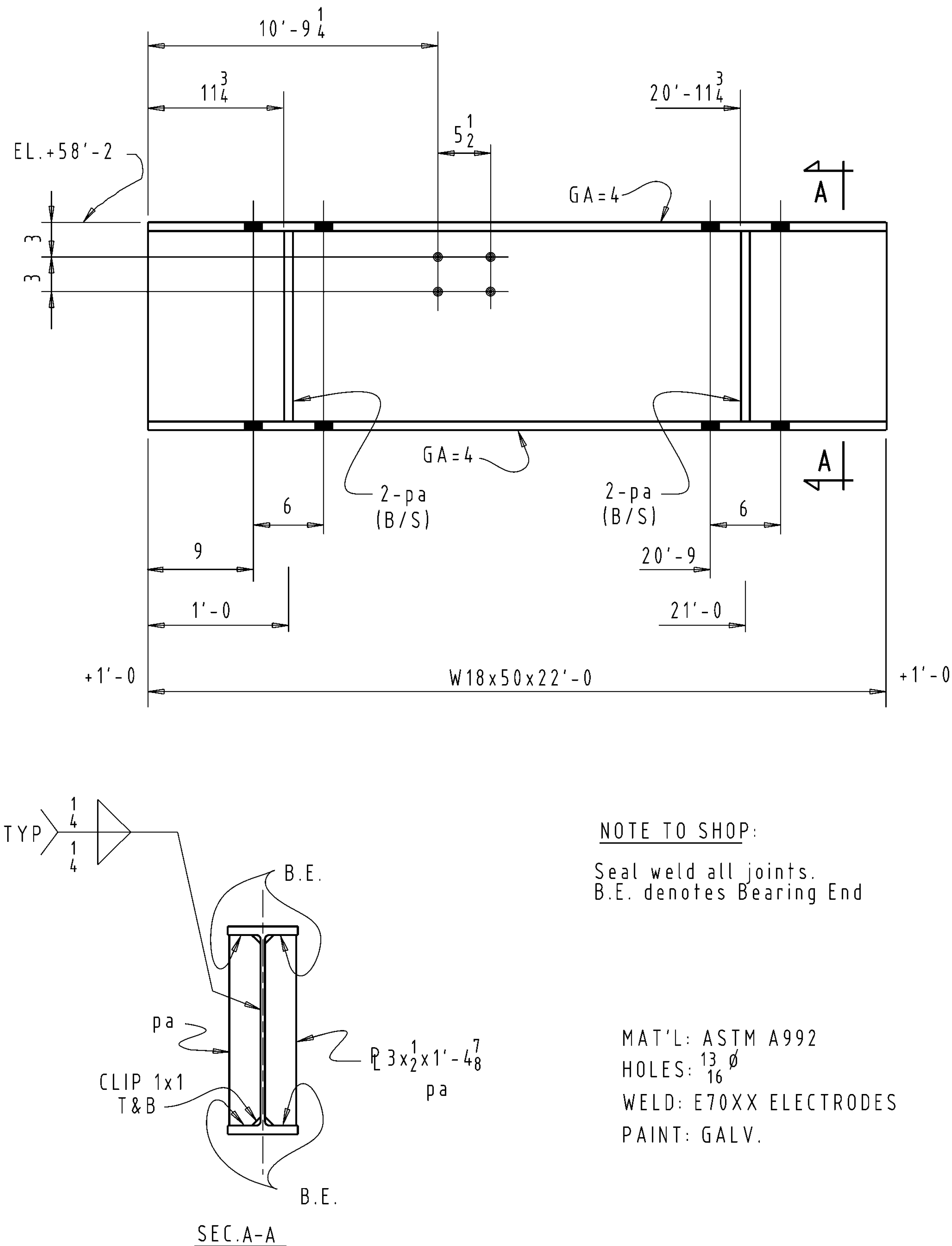


Figure 4-36. Web stiffeners welded to the flanges of a W-shape or plate girder.

"mouse hole" or a "rat hole") is burned in the web of the W-shape at its attachment to the plate (Figure 4-35). In the case of web stiffeners welded to the flange(s) of a W-shape or plate girder, the clipped corner(s) of the stiffeners (where they clear the beam fillet or girder web-to-flange weld) act as drain holes (Figure 4-36).

Field joints are made with galvanized bolts, nuts and washers. ASTM A307 bolts and ASTM A325 Type 1 high-strength bolts are permitted to be galvanized; A490 bolts are not permitted to be galvanized. Nuts are tapped after galvanizing. The *Manual* Part 7 describes the two methods available for galvanizing under "Galvanizing High-Strength Bolts." These methods are hot-dip or mechanical processes.

When at all possible, avoid situations that require welding on galvanized surfaces, particularly in the shop. Special ventilation must be provided in the shop to exhaust the toxic fumes that are produced. In addition, the galvanizing usually must be removed by grinding in the area to be welded. This requires the subsequent touching up with cold galvanizing compound after welding and cleaning. All of these operations add cost.

If field welding or torch-cutting is required or if the coating is damaged during shipping, handling or erection, touch-up methods are available to repair the surfaces. These methods include (1) painting with an organic cold galvanizing compound, (2) coating with a zinc based solder, or (3) metallizing with a zinc metal spray. Each method involves a prescribed application procedure. When welding galvanized pieces, adequate ventilation must be provided to evacuate the fumes or the welder must use a respirator.

Occasionally, a piece becomes warped during the galvanizing process to the extent it is unsuitable for erection. When this occurs, the fabricator and galvanizer must agree upon the terms for a method of repair or replacement.

ARCHITECTURALLY EXPOSED STRUCTURAL STEEL

Occasionally, contract documents identify a portion of the structural steel as "Architecturally Exposed Structural Steel" (AESS). This designation imposes additional fabrication and erection requirements on the members thus identified. The care and tolerances with which these members are fabricated and erected exceeds the usual requirements specified in the AISC *Code of Standard Practice for Steel Buildings and Bridges* in the *Manual* Part 16. Section 10 of the *Code* specifies the more restrictive requirements. AESS members may require stringent cleaning and painting. Such requirements must be noted in the contract documents.

For the most part, preparation of shop drawings for AESS members is the same as for the customary members, except that the AESS pieces must be identified as such on the shop drawings. Also, they should be identified on the erection drawings. The steel detailer should be aware of any joints between AESS members, which are to have a gap in order to detail the pieces so that the required gap can be obtained at the job site.

Unless specified otherwise in the contract documents, the additional requirements include, but are not limited to:

- The as-fabricated straightness tolerances for rolled-shapes and built-up members are one-half the standard camber and sweep tolerances in ASTM A6.
- Copes, miters and butt cuts in surfaces exposed to view are made with uniform gaps of 1/8 in. if shown to be open joints, or in reasonable contact if shown without a gap.
- Where welds are exposed to view, the welded surfaces should be reasonably smooth and uniform.
- Members fabricated from weathering steel must not have identification marks on surfaces that will be exposed in the final structure.
- Special care is taken to deliver AESS members to the job site without any distortions.
- Similarly, at the job site the erector must take special care during unloading, handling and erecting these pieces to avoid distorting them or damaging the paint.
- In plumbing, leveling and aligning the AESS members, the erector generally must satisfy tolerances that do not exceed one-half those normally required by the *Code of Standard Practice*.

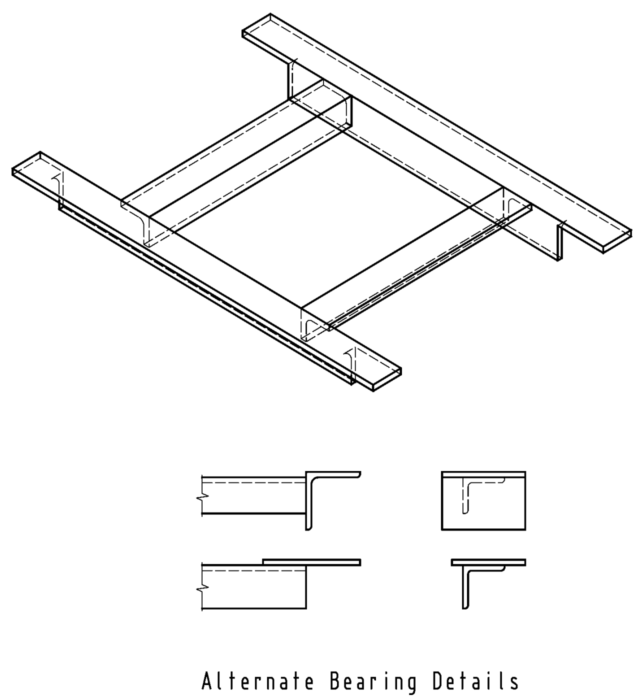


Figure 4-37. "Drop-in" all-welded roof frame.

SPECIAL FABRICATED PRODUCTS

Depending upon their geographical location and the market they serve, fabricators may develop special products. One such product commonly found at job sites is a "Drop-In" all-welded roof frame (see Figure 4-37). It consists of relatively lightweight angles, two of which span between a pair of roof beams, fabricated steel trusses, or open-web steel joists. According to the fabricator's preference, the main angles may bear on their supports in one of several ways as shown. The frames are used for openings for hatches, ventilators, exhaust stacks and air conditioning units in metal deck-supported flat roofs. The decking is field welded to the angles surrounding the opening. The hole in the deck is cut by the trade requiring the hole (HVAC, plumber, etc.) immediately prior to the installation of the equipment requiring the hole. The spaces between the angles are determined from the size of the required opening, sometimes provided to the steel detailer in a manufacturer's catalog.

OSHA SAFETY REQUIREMENTS AND AVOIDING UNERECTABLE CONDITIONS

Up to this point in the text, several situations have been described with respect to the steel detailer's ascertaining that the pieces being detailed can be erected. This involves familiarity with rolling mill and shop tolerances and the clearances required for erection. Also involved is accuracy in matching connections such as, for instance, the quantity and location of holes in a knife connection on a column flange matching the holes at the end of the beam that will connect to the angles. Another condition that can cause erection problems occurs when bent curb plates are shop welded to the top flange of a spandrel beam which connects to the web of a column. As the bent plate is notched to fit around the flange of the column, the piece is awkward to erect. Further, if the plate is expected to support curtain wall framing within tight tolerances, the customary combination of rolling mill, fabrication and erection tolerances may be unacceptable. In this situation, the solution likely would be to field weld the curb plate to the beam with curved plates top welded to spandrel beams. For additional guidance, refer to the *Manual* discussion on Building Façade Tolerances, page 2-27, and AISC Steel Design Guide 22, *Façade Attachments to Steel Frames* (Parker, 2008).

In order to erect beams onto seats in column webs, the steel detailer must remember that the beams are to be lowered between the flanges of the column and that other fittings on the column must provide the required clearance to permit erection.

OSHA (Occupational Safety and Health Administration) issues workplace rules. These rules establish the quality and safety of the work area environment. Such items as lighting, toxic fume control, dust control, methods of handling material, and work practices in the fabrication shop fall under the regulations of OSHA. Similarly, OSHA has established rules to which the erector is expected to adhere. These rules are concerned with such matters as fall protection, minimum field connections prior to final welding or bolting, protection from falling through floor openings, design and installation of temporary railing around edges of floors on multistory structures during steel erection, equipment safety, and material handling. Also, OSHA prescribes minimum requirements for industrial stairs, ladders and railings. Fabricators maintain a volume of OSHA regulations pertinent to their operations. *Code of Federal Regulations* Part 1926, Subpart R—Steel Erection (OSHA, 2001), found in the OSHA regulations, contains requirements with which the steel detailer should be familiar.

CHAPTER 5

PROJECT SET-UP AND CONTROL

Definition of the detailing conventions that are project specific and should be established when starting a new project.

PRE-CONSTRUCTION CONFERENCE

Frequently, soon after the structural steel fabricating contract is awarded, a "pre-construction" conference is held between the fabricator and the general contractor. The purpose of this meeting is for all parties to meet to review the high points of the project and the means and methods to be used to bring the project to a successful conclusion. Thus, the expertise of each party in the process can be employed at a time when implementation of economical ideas is possible. The sharing of ideas and expertise is key to a successful project. Topics of the meeting might include:

- Schedule
- Lines of communication
- RFI procedures
- Connection design
- Approval procedures
- Erection procedures
- Coordination with other trades
- Procedures for added cost to the project
- Painted or unpainted steel

Those attending this meeting could include:

- The owner or the owner's representatives for design and for construction
- The general contractor's senior representative, project manager and estimator
- The fabricator's senior representative, project manager and estimator
- The architect and engineer representative
- The Structural Engineer of Record
- The steel detailer
- The erector
- The inspector

Prior to this meeting, an internal meeting has been held between the fabricator's sales and production personnel. The main topics of this meeting are the transferring of pertinent estimators' documents from sales to production, discussing the contractual high points and reviewing any agreements made by the sales personnel. If subcontractors are a part of the fabricator's production team, they usually are present for part of the meeting. While they are present, the project requirements are defined and the team forms a plan to complete the project.

The process of obtaining design information and clarification will often require initiating a "Request for Information" (RFI) form, which is a written statement of the fabricator/steel detailer's questions. For purpose of expedience, the RFI is generally transmitted via fax or electronic mail. An example of an RFI is presented in Figure 5-1, where the problem to be resolved (beam reactions) is being addressed during the early stages of the project. In this case, the fabricator's project manager electronically issued the RFI to the owner's designated representative for construction, with a copy to the owner's designated representative for design. Although the form in this example is typed, frequently they are handwritten on prepared forms. In many instances, the RFI process can be expedited by a simple phone call to the owner's designated representative for design, in which case a formal, confirming RFI or memo should follow.

Having direct communication between the fabricator/steel detailer and the owner's designated representative for design requires the fabricator/steel detailer to keep the general contractor informed on all matters of design discrepancies and design drawing interpretations and of issues that may affect other trades. Some of the steel detailer's questions may involve sending sketches to the owner's designated representative for design to show, for instance, a possible solution to a problem or to illustrate a framing problem. Such sketches are sent via electronic mail or fax. Here, too, a phone call alerting the owner's designated representative for design that a sketch is being sent will serve to speed the response.

Aside from delivery dates for the structural steel, the steel detailer will be concerned with delivery dates for items to be embedded in concrete and masonry. These items include, but are not limited to, column anchor rods, wall anchors for decking or grating supports (usually angles or channels), embedded plates to which framing angles on beams will be field welded, and beam bearing plates. Such items are required at the job site early in the construction schedule, weeks before the structural steel. Therefore, the steel detailer must prepare the shop and erection drawings for them immediately.

A sample pre-construction/pre-detailing meeting agenda has been included at the end of this chapter.

The Best Fabricators
P.O. Box 000
Weldersville, US 22222

RFI No. 1

Request For Information

	Fantastic Building Co.	FROM:	Ken Doe
ATTN:	Bill Doe	FAX #:	(777) 777-7777
FAX #:	(333) 333-3333	REF:	ABC High Rise
DATE:	June 7, 2000	JOB #	1847

Total Number of Pages (Including This Cover Sheet) = One

NOTE: Time is of the essence for this request. Please return by June 9, 2000

Due to the short length of many beams on this project, the use of maximum allowable uniform loads for determining beam end reactions results in unreasonably high values. Having performed some preliminary calculations, we would like to suggest the following upper-bound reactions for the following beam sizes:

W8x10 – 10 kips
W10x12 – 10 kips
W12x14 – 18 kips
W12x16 – 18 kips
W12x26 – 18 kips

Please advise as to the acceptability of this proposal. We also request that you furnish the actual design end reactions for all W12x72 beams, since the uniform load reactions appear to be excessive in all cases.

Your prompt reply is greatly appreciated.

Ken Doe
Project Manager

Cc: File

Figure 5-1. Sample request for information.

PROJECT-SPECIFIC CONNECTIONS

Information on design drawings and specifications regarding connections appears in a variety of ways. Most commonly a note will state that connections are to be developed in accordance with the AISC *Specification*. Such a general note leaves the fabricator the option to select the types of connections with which it prefers to work. The various types were discussed in Chapter 3. Rarely, however, does a design note specify that the connections must be either bolted or welded, leaving that choice to the fabricator. In the event that such a note is on the construction documents and conflicts with the fabricator's usual shop operations, the fabricator should request a change from the owner's designated representative for design. When no connection note is given, the AISC *Code of Standard Practice* states that the connections are to be in accordance with the requirements of the AISC *Specification*.

At the pre-construction conference, one item to be discussed would be a design note that stipulates that all high-strength bolted connections are to be slip-critical (SC-type) connections. The RCSC *Specification* lists specific situations where SC-type connections must be used; otherwise, bearing-type connections are acceptable. The differences between these two types of connections concern the number of holes in a connection, the number of bolts in a project, the requirement for specified pretensioning and possible shop painting activities (such as masking field bolt holes).

Parts 9 through 15 of the *Manual* offer tables of connection strengths and supporting text to which the steel detailer can refer. A CD of examples is also provided. These include reinforcement of coped beam webs, moment connections, hangers, brackets, beam bearing plates, trusses and other aspects of connection design. Some of these connections may be complex and require a structural engineer to develop them.

COORDINATION WITH OTHER TRADES

The structural steel detailer usually coordinates his/her activities with those supplying the shop drawings for significant items such as open-web joists, decking and/or miscellaneous metals (ladders, stairs, etc.). The information is furnished to the steel detailer by the fabricator and general contractor. The joist detailer will need to furnish to the structural steel detailer information on location and sizes of joist seat holes for inclusion in the details of the supporting beams and columns. Similarly, the detailers of the decking and miscellaneous metals connecting to the structural steel are interested in the types of supports being provided for their products. Structural beams may require holes in them to support stairs or ladders. Holes for these kinds of items should be made the same size as those for structural steel to avoid special punching or drilling in the shop.

When shop drawings of joists, decking and miscellaneous metals are sent by the respective item suppliers to the owner's designated representative for design for approval, a copy should be sent to the steel detailer to ascertain that connections match. A variation of this approval procedure may occur if the fabricator included furnishing one or more products of other trades (such as joists, decking and miscellaneous metals) in the fabrication price. In this situation the fabricator may send the shop drawings of these trades for approval, but the structural steel detailer should receive a copy of them. Regardless of the approval procedure, the information from the other trades must be made available to the steel detailer so as not to delay the preparation of the structural steel shop drawings.

Other significant items that should be coordinated with the steel detailer's work include, but are not limited to:

- Roof and floor openings
- Cladding attachment
- HVAC equipment
- Edge of slab dimensions and details when using bent plate pour stops
- Deck support at columns with moment connections
- Attachment for wood
- Folding partition supports
- Precast connections
- Framing for mechanical supports
- Placement of shear studs for concrete work

ADVANCE BILL FOR ORDERING MATERIAL

The majority of steel for most contracts is ordered directly from steel rolling mills. Actual steel fabrication cannot be started until this material is received at the fabricator's plant. The acquisition of all raw material required for a given contract is a responsibility of the fabricator's purchaser, the person responsible for buying all the material required. Material may have to be acquired from a number of sources—some from one or more rolling mills, some from a steel service center, and some from the fabricator's own stock. Before the purchaser can place any orders with suppliers, the steel detailer must prepare a complete and detailed description of all the raw material that will be required. Such a list is prepared on a form variously referred to as an advance bill of material, an advance bill, a reserve bill, a material list or a preliminary bill. The terms "advance" and "preliminary" emphasize the very early stage in a contract when material is ordered.

Ideally, ordering would be postponed until shop details are made and checked. Then, the required dimensions for each piece could be taken directly from the shop detail drawings. Unfortunately, this seldom is possible if delivery dates are to be met. Therefore, ordered lengths must be determined in advance. If project specifications state that steel is not to be ordered prior to the approval of shop drawings, this fact should be brought up at the pre-construction conference and the ramifications of an extended fabrication schedule explained.

The precise form and detailed arrangement of these bills will vary with different fabricating companies. Essentially, they are similar to the computer generated printout shown in Figure 5-2. The steel detailer enters information into the computer for each piece of material as it is read from the design drawings. From that input the computer produces a printed document (Figure 5-2) showing material listed in groups by size, weight and length for rolled shapes and by thickness, width and length for plates. Usually, groups are listed alphabetically by nomenclature (C, HSS, L, PL, S, W, etc.) Shape subgroups are listed in order by size and weight. Plate subgroups are listed in order by plate thickness and width.

The printout is used by the purchaser to obtain material. Referring to Figure 5-2, horizontal rows are numbered consecutively for ease in identifying the particular item shown on any given line. Each line is divided to provide space for the information furnished by the steel detailer for:

- A statement of the total number of pieces included in each item.
- A complete description of each required item, giving dimensions.
- An indication of the amount of extra length to be provided for trim or finishing, if required.
- Unit and total weights calculated in pounds by the computer for each line item of material.

Some advance bill forms provide space for remarks and notes, which are helpful to the order department and the steel detailer. Refer to Figure 5-2 and note the following points, which are typical of the practice used in preparation of advance bills of material:

- The Best Fabricators will furnish fabricated steel for the DEF High Rise.
- All W-shapes listed are to meet the specification requirements of ASTM A992 (latest adoption date understood). The balance of shapes and plates shall meet the specification requirements of ASTM A36.
- In some shops, the length allowed for finishing is established by the order department. They are guided by notes such as B2E (bevel two ends), B1E (bevel one end), by the size of the shape to be finished, by finishing requirements, and by the method used in finishing. However, some numerical allowances are given in the guidelines for preparing advance bills (listed later).

Short lengths of same-size/weight shapes are combined by the purchaser for ordering in lengths available from rolling mills. From these long lengths the shop will cut the required pieces. Similarly, plates may be ordered in sizes from which several smaller plates (of the same thickness) can be shop-cut. This is termed "ordering in multiples." The steel detailer writes the advance bill showing actual lengths of pieces; the purchasing department performs the multiples function. An exception is when the steel detailer recognizes that some material may be cut as shown in Figures 5-3b and 5-3c. In these cases, the steel detailer prepares the advance bill by giving the length of material required to furnish the pieces plus the kerf. The kerf is the width of material removed during the cutting process and depends upon the equipment used. The steel detailer should obtain this dimension from the fabricator.

The purchasing department may consist of one or more individuals whose function is to purchase raw structural steel and any other items (paint, bolts and nuts, washers, bronze bearings, etc.) needed to complete a project. Some fabricators may split the purchasing function among individuals in the shop office and in the estimating department.

A copy of the purchase order is made available to the shop office. A copy of the computer-printed advance bills is retained in the drafting department. A copy of the printout is sent to the estimating department to compare the estimated weight with the weight of ordered material. This comparison provides a check on the steel detailer's understanding of the scope of work and the accuracy of the estimate.

Ditto marks are not used in the preparation of an advance bill. In listings of this type, each item should be described completely, independently of all other items. Ditto marks are not used because at some future time a particular item may be changed by a revision. Also, a carelessly made ditto mark may be mistaken for the numeral "11."

Each piece of raw material will be marked with the fabricator's order number when it is shipped from the rolling mill. Upon arrival at the fabricator's plant, it will be checked against a copy of the purchase order. Also, it will be sorted for easy access and placed in the incoming storage yard.

An important function of the steel detailer is to ensure that each piece billed on the shop drawing has been ordered or is available from shop stock. Fabricators have developed systems through use of the computer to have material obtained for the project "applied" to each line of material billed on the shop drawing. When the shop drawing is sent to the shop for fabrication, a clerk copies the contents of the shop bill into the computer. The input is matched to data existing in the computer, identifying each line of material on the advance bill with a source identification—mill item number, shop stock or a steel service center item number. Thus, a separate computer-generated shop bill is produced that provides the information the shop needs to apply the correct material to the shop drawings.

When the advance bill is prepared, the steel detailer must exercise extreme care to ensure the correct placement and

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Updated Advance Bill of Material ABM File: ABM6-28 Page 1

Sequence.. 1
Steel Grade... A992
Member Type... Beams

Line	Qty.	Material	Length	End Prep	Unit Weight	Total Weight
1	1	W24x68	35- 2 9/16	B2E	2395	2395
2	1	W24x68	35- 1 11/16	B2E	2390	2390
3	3	W24x68	35- 1 9/16	B2E	2389	7167
4	1	W24x68	35- 1 9/16	B2E	2389	2389
5						
6	1	W24x62	35- 2 9/16	B2E	2183	2183
7						
8	1	W24x55	35- 1 11/16	B2E	1933	1933
9	1	W24x55	35- 1	None	1930	1930
10						
11	3	W21x50	35- 1	None	1754	5262
12	1	W21x50	35- 1	None	1754	1754
13	1	W21x50	35- 1	None	1754	1754
14						
15	1	W16x31	39- 8	None	1230	1230
16	2	W16x31	39- 7 7/16	B2E	1228	2456
17	1	W16x31	39- 7 1/16	None	1227	1227
18	1	W16x31	39- 7 1/16	None	1227	1227
19	1	W16x31	39- 7	None	1227	1227
20	3	W16x31	39- 7	None	1227	3681
21	1	W16x31	38- 5 5/16	B2E	1199	1199
22	1	W16x31	38- 7 13/16	B2E	1198	1198
23	1	W16x31	38- 7 5/16	B2E	1197	1197
24	1	W16x31	35- 1	None	1088	1088
25						
26	1	W16x26	39- 7	None	1029	1029
27						
28	1	W14x22	21- 1 1/2	None	465	465
29						
30	1	W12x50	35-1	None	1754	1754
31						
32	1	W10x12	22- 1 1/4	B2E	265	265
33						
34	1	W8x10	15- 7 5/8	B2E	156	156
35	1	W8x10	15- 7	None	156	156
36	1	W8x10	13- 0 1/2	None	130	130
37	1	W8x10	11- 5 13/16	B1E	115	115
38	2	W8x10	11- 1 13/16	B2E	111	222
39	1	W8x10	7- 1 1/2	None	71	71

Sub Totals: Material Count: 38 Weight: 49,250 Surface Area: 5,923

Figure 5-2. Sample advance bill printout.

clarity of each entry on the form. This is done in strict conformance with computer program requirements. As neither the computer operator nor the computer is expected to interpret unclear or dubious information, the steel detailer is solely responsible for the clarity and correctness of the advance bills.

ADVANCE BILL PREPARATION

The preparation of advance bills to permit efficient ordering of material requires an experienced steel detailer, thoroughly grounded in structural detailing, familiar with mill practices and conversant with specialties that often become part of structural steel contracts. As avoiding waste is extremely important if savings are to be realized, material must be ordered economically and accurately. Before advance bills are submitted for processing, they should be checked by an experienced checker.

Material that must be shipped in advance, such as anchor rods, leveling plates, base plates, etc., should be ordered at the earliest possible date. Some fabricators require that the advance bills for this type of material be labeled "Advance Shipment." Similarly, items that must be purchased for assembly with a shipping piece must be expedited so as not to delay a shipment of fabricated pieces.

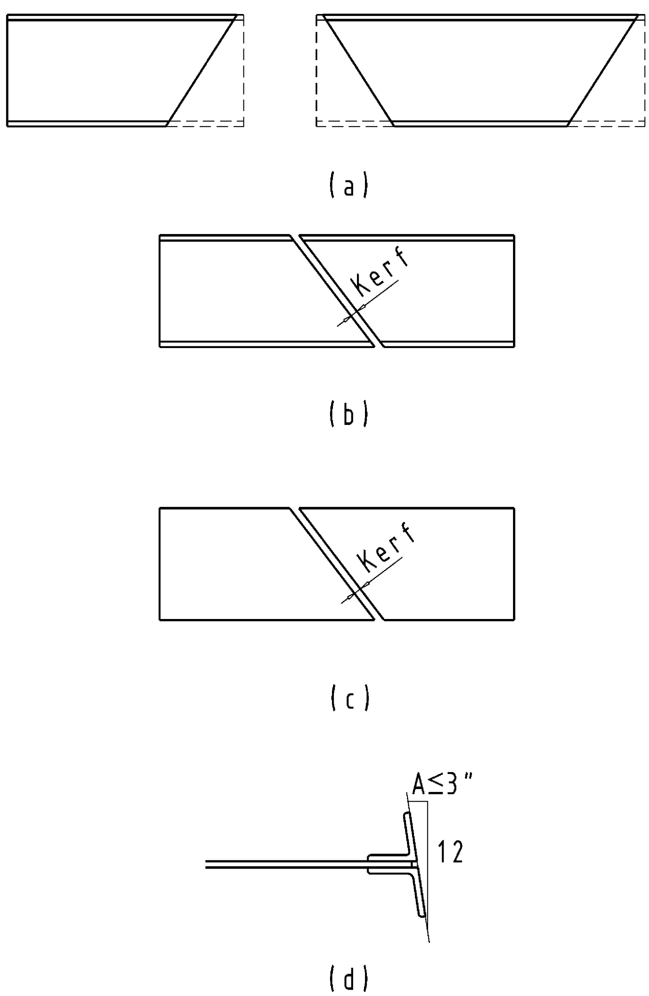


Figure 5-3. Kerf, diagonal and skew.

Finishing requirements for base plates are given in AISC *Specification* Section M2.8. Base plates not greater than 2 in. thick need not be milled if satisfactory contact in bearing can be achieved. Where this condition cannot be realized and for thicker plates, Table 14-1 in the *Manual*, Part 14 lists the various finish allowances required. When preparing advance bills for base plates, the size ordered includes the finished thickness with the finish allowance noted. For purposes of economy, if permitted by the owner's designated representative for design, base plates should be ordered in as few widths and thicknesses as possible.

If a project has no divisions or shipping sequences, advance bills may be numbered thus: 1, 2, 3, etc. On the other hand if the project has numerical divisions (or shipping sequences), such as Division 1, Division 2, etc., number the advance bill pages thus: 101, 102, 103, etc., in Division 1; 201, 202, 203, etc., in Division 2; 301, 302, 303, etc., in Division 3, etc. Different fabricators may have variations of this system.

Listed next is a set of guidelines. These guidelines (or a variation of them) are employed by fabricators when ordering material. Although fabricators maintain their own requirements for cutting and finishing allowances, the numbers given are representative of the amount of allowances preferred. Nevertheless, the steel detailer should ascertain the fabricator's specific requirements. Whether the steel detailer shows the allowance at the time the advance bill is written or the allowance is applied by the fabricator's purchasing department depends upon the preference of the fabricator.

Columns

- Wide flange shapes are ordered to match the detailed length plus a finish allowance. The amount of the allowance depends on the fabricator's practice and the available equipment. As straightness is desired in W-shapes used as columns, the application of mill tolerance for column straightness is limited to sections that have approximately the same depth as flange width. Table 1-22 in the *Manual*, Part 1 lists tolerances for camber and sweep. The sections to which the limited tolerances apply are given at the bottom of the table. If straightness is critical for sections not included in the "limited tolerance" group, the steel detailer will need to note the shop drawing for maximum sweep and camber permitted.
- Welded column flange plates are ordered to the thickness specified on the design with $1\frac{1}{4}$-in. allowance on the length for trim or finish.
- Web plates for welded columns are ordered with an allowance for trim on the width and with a $1\frac{1}{4}$-in.

allowance on the detailed length for trim or finish. Both edges of the web plate must be trimmed. Therefore, web plates are to be ordered detailed (net trimmed) with allowances for mill camber. Although mill camber may vary from 8 in. to $^1/_4$ in. for each 5 ft of plate length, or fraction thereof, for ordering purposes generally only 50% of this amount is to be used. Allowances should be determined to permit trimming at least $^1/_4$ in. from each edge.

- Avoid making multi-story building columns in one-story lengths and, thus, creating unnecessary splices. The customary multi-story building columns are fabricated in two-story lengths. Generally, this column length falls within the restrictions of OSHA, which limit the height above which workers require permanent or temporary deck installed on a lower level in order for erection of steel on higher levels to proceed. Columns for three-story buildings often are fabricated in one piece. However, the erector must lay deck on at least one of the first two floors before erecting steel at the roof level.

Welded Girders

- Web and flange plates should be ordered to detailed length with $1^1/_4$-in. allowance added for trim or finish.
- Web plates must have both edges trimmed. They are to be ordered detailed (net trimmed) width for mill camber plus camber required, if any, in the finished member. Although mill camber may vary from $^1/_8$ in. to $^1/_4$ in. for each 5 ft of plate length, or fraction thereof, for ordering purposes generally only 50% of this amount is to be used. Allowances should be determined to permit trimming at least $^1/_4$ in. from each edge.
- When ordering web plates for cambered girders, increase width and length to account for the manner in which the plate girder will be produced.

Trusses

- For welded chord sections, web and flange plates should be ordered to detailed length with a $1^1/_4$-in. allowance added for trim or finish.
- Web plates must have both edges trimmed. They are to be ordered detailed (net trimmed) with allowances for mill camber. Although mill camber may vary from 8 in. to $^1/_4$ in. for each 5 ft of plate length, or fraction thereof, for ordering purposes generally only 50% of this amount is to be used. Allowances should be determined to permit trimming at least $^1/_4$ in. from each edge.
- Members for the truss web should be ordered to detailed length using joint layouts.
- From a layout of the truss joints, the steel detailer can determine with a fair degree of accuracy the sizes of the gusset plates to order by scaling the dimensions of the plates.

Beams, Purlins and Girts

- Order lengths of structural shapes that do not require finishing to the nearest $^1/_4$ in.
- Depending upon the practices and preferences of the fabricator, lengths of beams and channels having shop-attached framing angles and framing to beams or channels are determined by deducting from the center-to-center of supports:
  (a) $^3/_4$ in. for ends framing to webs $^5/_8$ in. or less in thickness,
  (b) 1 in. for ends framing to webs over $^5/_8$ in. in thickness.
- Depending upon the practices and preferences of the fabricator, beams and channels framing between columns or welded girders having shop-attached framing angles are ordered $^7/_8$ in. to $1^1/_{16}$ in. less than the distance face-to-face of supporting steel, providing such framing can be erected. Lengths so obtained are rounded off to the nearest $^1/_4$ in.
- Beams and channels beveled at ends should be ordered with $^1/_2$-in. trim for each beveled end (Figure 5-3a).
- Short beams and channels with beveled ends may be ordered in multiple lengths and flame cut or sawed to length in the shop. The steel detailer should consult with the fabricator. Members that are ordered in multiple lengths should have allowances added for the width of the kerf (Figure 5-3b).
- When design permits, continuous lines of beams or channels, such as purlins resting on top chords of trusses or crane runway beams resting on brackets or tops of crane columns, should be ordered 1 in. less than the distance center-to-center of trusses or columns.
- Provided building joints are flashed, eave struts and girts forming window headers and sills should be ordered 1 in. less than the center-to-center of the columns. Where the joints are not flashed, the exposed material should be detailed and ordered with not more than $^1/_8$-in. joint opening. Provide $^3/_4$-in. trim when ordering.
- Trolley and monorail beams, where wheel or trolley rests directly on the flange of the beams, should be ordered center-to-center of support with $^3/_4$ in. allowed for finishing. Finished lengths should provide $^1/_{16}$-in. clearance at splices.
- Generally, angles and tees used as purlins or girts should be ordered in the same manner as beams and channels.
- At moment connections where beam flanges are to be field welded to the column, the steel detailer should consult with the fabricator for the amount of trim required to fabricate the prepared ends of the flanges.
- Beams with ends made square for field weld preparation or for the attachment of shear end- plates must be ordered similar to columns to allow for the finishing of the beam ends.

Detail Material

- If reasonably correct lengths of angles, bars, plates or rolled shapes cannot be determined prior to preparing the advance bill, estimate the total requirement, including trim and kerf allowances. On the advance bill list sufficient stock lengths to make up the total required. Based on what material is available in shop stock, the fabricator will order what is required to complete the project. To use some of the shop stock available, the fabricator may request the steel detailer to verify if specific detail material substitutions are permitted. This is one type of situation in which the steel detailer would issue an RFI.
- If worthwhile savings are possible, material having one square end and one diagonal end cut should be nested in pairs, each such pair to be considered as one detail piece for ordering. On the advance bill show a sketch with the note "1 cuts 2" (Figure 5-3c). Detail pieces having both ends cut diagonally should be treated similarly.

Pipe

- Pipe for structural uses is ordered to nominal diameter specified as "standard," "extra strong" or "double extra strong" and to ASTM Specification A53.
- Advance bills for pipe must show the specifications to which it is to be ordered.
- When requirements will permit, order pipe in linear feet and specify "random lengths." Single random lengths will range between 16 and 22 ft, but manufacturers retain the right to furnish a small percentage 6 to 12 ft long.
- With regard to ends, standard pipe should be specified as follows: threaded ends with couplings, threaded ends without couplings, plain ends or plain ends beveled for welding.
- *Manual* Part 1 contains tables of selected sizes of pipe—those most often encountered in the structural steel fabricating industry.
- Pipes used as columns with finished ends are ordered similar to wide flange columns.

HSS Products

HSS products come in round, square and rectangular shapes. Round HSS products differ from pipe in that the nominal size is virtually the same as the actual outside diameter. Because of the great variety of available sizes, shapes, surface finishes, material specifications and tolerances, the steel detailer should consult the *Manual*, page 1-5, before ordering. Finishing allowances must be used similar to wide flange columns.

Rails and Accessories

Rails for crane runways are illustrated and discussed with their accessories (splice bars, fastenings and clamps) in AISC *Manual* Parts 1 and 15. To order these products, the steel detailer should consult the catalogs of manufacturers.

Miscellaneous Items

Fasteners, such as studs, adhesive anchors, expansion bolts and other similar anchoring devices, are usually ordered on an advance bill listing a vendor's catalog number, quantities and other pertinent information required. Similarly, the steel detailer must order on an advance bill chains, clevises, turnbuckles, rods, special bolts and washers, slide bearings and any other items included in the fabricated steel contract. As with fasteners, the steel detailer will require vendors' catalogs for use in properly ordering these items. Where necessary for clarification, the steel detailer will be expected to furnish sketches of such items to facilitate their purchase.

Rolling and Bending

When ordering material to be bent during fabrication, the steel detailer will usually have to add an allowance, called a crop, to facilitate rolling. This will give the shop some latitude in locating the bend and will permit final trimming to secure the proper length or shape of the piece after rolling. Consult with the fabricator to ascertain how much allowance is preferred. The ordered lengths of rolled material are to be based on center-of-gravity lengths except angles, which are ordered using dimensions on their backs.

If at all possible, bent plates should be ordered such that bend lines will be perpendicular to the direction of rolling. The width of the plate written on the advance bill will be considered the dimension perpendicular to rolling.

When providing framing connections for skewed beams, fabricators generally can bend angles to a skew up to $A=3$ in Figure 5-3d. However, rolled angles will tend to bend about the root of the fillet, which produces a significant jog in the leg alignment. Therefore, the recommended practice is to bend angles to a skew up to $A=1$. For greater skews, consider using bent plates or a shear end-plate connection described in Chapter 3. If bent plates are used, every effort should be taken to ascertain that the bend line is perpendicular to the direction of mill rolling.

Architecturally Exposed Structural Steel (AESS)

All material to be fabricated as Architecturally Exposed Structural Steel must be identified on the advance bills of material as AESS. This alerts all material suppliers that the steel must be produced, handled and shipped according to special rules to protect its appearance.

As far as practicable in the time allowed for ordering material, the lengths required and the allowances for trim and finish should appear on the advance bills. This is especially important for material critical to the project schedule (i.e., columns, beams, trusses, purlins, girts) and for work that is duplicated extensively. However, on rush jobs and when advance bills must be released before exact sizes of material less critical to the project schedule can be determined by layouts or final details, such material must be ordered to sufficient length or size to cover probable requirements. For example, bracing angles may be ordered center-to-center of work points with a deduction to account for probable connection details. As soon as actual sizes are determined, the steel detailer should ascertain the status of such material ordered and issue a change order if possible.

REFERENCES

Lists of the various construction industry organizations for materials and contractor groups may be found on the AISC Internet web site www.aisc.org.

DETAILING KICK-OFF MEETING SAMPLE AGENDA

At the beginning of a project, it is advantageous to hold a meeting with various members of the construction team. The purpose of this meeting is to establish guidelines and rules for the development of the project throughout detailing, fabrication and erection. The detailer must take direction from the customer (which is typically a fabricator). However, the detailer will need to take into consideration the work of other trades and incorporate certain details into the drawings with the permission of the detailer's customer. There are often several issues to resolve at the beginning of a project, and it is not necessarily the detailer's responsibility to decide upon a course of action for each particular issue. Moreover, the detailer will provide suggestions and comments to aid in the development of the project and the fabricator, erector, contractor or owner will provide direction. Here is a generic outline of the topics that may be discussed at such a meeting.

I. CONTRACT DOCUMENT REVIEW AND GENERAL PROJECT OVERVIEW
   A. Establish the most current set of contract documents and confirm that all parties are working with this set.
   B. Clarify scope if required.
      1. Delineate between structural steel and miscellaneous steel.
      2. Confirm who is responsible for interface items such as embedment plates that support elevator beams, bolts that connect joists to structural trusses, connection angles that support metal pan stair stringers, and so on.
   C. Identify areas of concern such as critical tolerances (AISC and ACI), construction phasing, coordination with other trades, etc.
   D. Provide and discuss preliminary layouts showing the sequencing system to be used. This should include the lines of demarcation between sequences and the piece-marking system. The sequencing needs to be reviewed by the general contractor and erector for access and coordination with other trades.
   E. Confirm which codes and specifications and design methodology are applicable (ASD, LRFD, IBC, NFPA, city codes, etc.).
   F. Discuss any special concerns such as material grades, anchor rod grades, shop and field welding requirements, painting, bolt tensioning, connections for light gage girts and so on.
   G. Identify which areas of the structure are to be redesigned, if applicable.
   H. Identify delegated design items such as handrails, metal stairs, elevator machine room framing, etc.
   I. Review OSHA requirements and incorporate details required for OSHA conformance, such as perimeter cable supports and guy line connections.
   J. In projects involving remodeling work, identify the schedule for field measuring and exploratory demolition, and the responsible party.

II. DETAILING PROGRAM AND COORDINATION ISSUES
   A. Discuss detailing methodology to be used.
      1. Discuss fabrication and bolting method to be used.
      2. The detailer should be provided with the fabricator's and erector's standard details.
      3. Bolt diameters and grades and hole size should be provided in the contract documents. However, the detailer and fabricator should review this matter for a number of reasons.
         a. Select bolt diameters and grades to suit the loads when connections are to be designed by the fabricator.
         b. Maintain a $^1/_4$-in. difference in diameters to avoid installing an undersized bolt in a joint, where possible.

c. Try to maintain one bolt grade if feasible. In any event, do not use two different grades of bolts with the same diameter.

4. The entering clearances for tension control bolts and regular bolts are different and must be considered by the detailer.
5. Confirm which joints are to be designed by the fabricator and which methods of design are acceptable (i.e., elastic method or uniform force method).
6. Simple framed connections may be presented in table format.
7. Oversized and slotted holes may be used with the permission of the engineer. Although using such holes provides adjustability on the structure, having too much adjustability can cause difficulties for the erector.

B. Propose alternate details where applicable.
1. Review connection details that may be problematic for erection, such as framing angles welded to a beam web and connects to column webs at both ends, and explore alternate details.
2. Some shops prefer to bolt detail pieces to main members whereas other shops prefer to weld. The engineer must approve switching from one method of attachment to the other.

C. Discuss any information that may be missing or ambiguous in the contract documents. Refer to Part 3 of the AISC *Code of Standard Practice*. And CASE Document 962D, *A Guideline Addressing Coordination and Completeness of Structural Construction Documents*.
1. Member loads, dimensions and elevations.
2. The location and extent of section cuts and details.
3. Areas that are to be painted and the finish that is required.
4. Areas that are to be fireproofed in the field and are to be left, therefore, unpainted.

D. Review the location of member splices for constructability. Splices in trusses must be located such that the shop and site cranes are able to manage the weights of the pieces. Columns splices need to be located such that bolt access is possible from the floor level and so that a safety cable may be installed on the perimeter column being spliced (Ref. OSHA, Subpart R).

E. Determine if erection aids are required such as lifting lugs and temporary connections. Establish the size and location of lifting holes.

F. Confirm what sorts of details are required to accommodate other trades such as beam penetrations and supporting frames for roof equipment. If this information is not immediately available at the time:
1. Provide the owner with dates for the release of the required information such that detailing is not affected.
2. Ask if it is possible to assume "safe" locations with available dimensions and details. Proceeding on this basis would be carried out at the risk of the general contractor/owner in the event that the final details are different than the as directed details.

G. Discuss the advance bill of material (ABM) with the fabricator.
1. Develop a schedule for submitting the ABM's for material procurement.
2. Confirm what information needs to be provided on the ABM other than the material size and length (i.e., mill certificate requirement, Charpy testing, domestic material requirements, etc.)

III. COMMUNICATION

A. Set up an RFI system that complies with any RFI requirements outlined in the general contract provisions. It is helpful to include a numbering scheme that identifies the trade and company and a distribution list. Identifying the company can be accomplished by including the initials of the company name in the RFI number. The detailer must keep an accurate log of RFI status.

B. Request that direct contact be permitted between the fabricator/detailer and the owner's designated representative for design. Official documentation should follow any such communications for record purposes.

C. Request that an advance courtesy copy of the approval drawings may be sent to the owner's designated representative for design to help speed up the approval process.

D. Establish a regular schedule for meetings and/or conference calls, and determine who must participate.

E. Identify a person at the general contractor's office who will receive calls from serve as the primary contact with the steel subcontractor. An alternate contact should be established in case first contact person cannot be reached.

IV. DETAILING SUBMITTAL SCHEDULE

A. Determine how many prints and reproducible drawings are required for approval and for field use.

B. Connection designs are to be submitted well in advance of the detail drawings so that the detailer can produce detail drawings with approved connection designs.

C. Provide a detailing submittal schedule.
   1. Submittals should contain complete sequences.
   2. The AISC *Code of Standard Practice* stipulates that the fabricator is to allow 14 calendar days for the return of drawing submittals.

D. If schedule-critical areas exist on the project, the contractor should identify such areas for the owner's designated representative for design by the contractor. An accelerated turn around should be requested for the applicable drawings. It is impractical to expect a quick return of all submittals, so good judgment must be used when requesting such.

V. CHANGES TO THE CONTRACT

A. Determine how changes are to be dealt with
   1. The owner must decide whether changes are to be incorporated immediately or if cost and schedule impacts are to be provided to the owner for review before proceeding.
   2. In the event that the owner wants to review cost and schedule impacts before deciding whether to proceed with a change or not, supply dates by which a decision must be made so that the work is not affected further.
   3. Review procedures for notifying the fabricator when design revisions are made on approval of shop drawings.
   4. Some contracts contain unit prices for various changes. Determine if the changes in hand fit into a unit price category or not. (Changes to the scope that are based on unit prices must be made in a timely manner on the part of the owner.)
   5. In any event, the detailer shall take direction from the customer regarding the incorporation of changes.

B. Depending on the number of changes, it may be necessary for the detailer to maintain and distribute a "hold list." This list will include the piece mark, quantity of pieces, current status of the piece (not drawn, at approval, in the shop), the date the pieces were put on hold, why the pieces were put on hold, the erection sequence, and a release date.

C. Shop drawings that are revised or produced in response to a change should contain information that provides the source of the change (e.g., RFI number, drawing release number, customer request, etc.). This can be accomplished by providing either a brief description or a code in the revision box on the applicable drawing. The detailer should also keep a log of these drawings with summary information for record and management purposes.

VI. QUALITY CONTROL/NONCONFORMANCE ISSUES

A. Review the shop and field inspection requirements. Any alternate details should be selected in light of the inspection requirements so as not to add extra inspection work.

B. Destructive testing of bolts is sometimes required. Generally the number of bolts tested is small. However, in the event that the destructive testing becomes exhaustive, the number of bolts to be tested must be incorporated into the bolt lists provided by the detailer. This is normally handled by adding a certain percentage to the bolt quantity.

C. Charpy V-notch testing, when required, should be noted on the advance bill of material (ABM).

D. Corrective modifications made to pieces either in the shop or in the field will require a drawing.
   1. Shop rework can be handled by revising a shop drawing and issuing it to the shop.
   2. Field modifications require a fieldwork drawing, which is normally labeled "FW-XXX." This drawing should provide complete and clear instructions to the erector on the modifications to be made
   3. Both types of drawings should provide a reference to the source of the rework (contract change, nonconformance number, customer request, etc.). This can be accomplished by providing either a brief description or a code in the revision box on the applicable drawing. The detailer should also keep a log of these drawings with summary information for record and management purposes. These drawings are also required in order to furnish the owner with a complete set of as-built drawings.
   4. Revise the hold list to incorporate nonconformances.

E. A "punch list" will be developed and maintained by the inspector and will be distributed to the affected parties. Items contained in the punch list may address misfits, un-tensioned bolts, mislocated holes, nonconforming welds and so on. Accordingly, certain items will require action on the part of the detailer, while other items will not.

Upon the conclusion of the detailing kick-off meeting, a set of meeting minutes shall be prepared, usually by the general contractor or construction manager, and distributed to all parties.

CHAPTER 6
ERECTION DRAWINGS

Guidance for the preparation of erection drawings.

ERECTION DRAWINGS

The location of every shipping piece (individual member or subassembly of members shipped as a unit) must be shown on a drawing so that the steel frame can be erected quickly and accurately. The purpose of an erection drawing (defined by the AISC *Code of Standard Practice* as "Field-installation or member-placement drawings that are prepared by the Fabricator to show the location and attachment of individual shipping pieces") is to allow the erector to locate the size and length of a member, its piece mark and its position within the erection sequence. In addition, computer models or computer-generated isometric drawings are sometimes provided to give the erector a three-dimensional representation of the erected structure. For this purpose a set of erection drawings is created, such as is illustrated in Figures 6-1a and 6-1b. These drawings consist of line diagrams representing framing in plan, elevation, section, etc., to which principal dimensions, erection marks, notes and, when required, enlarged details, bolt installation requirements and field welding requirements are added.

The erection drawings show each separate piece or subassembly of pieces with their assigned shipping or erection mark to identify and locate them in the framework. The erection drawings show the sizes of members and give sufficient information so that the erector can assemble the structure easily and will not have to study the shop drawings to any great extent to determine how members frame together. Usually, the numerical identification of erection drawings is prefaced with E, such as E1, E2, E101, E302, etc. The drawings prepared by steel detailers for use by the erector are not intended to show the erection scheme, schedule, rigging devices, temporary supports, safety devices and such other material and equipment required to erect the structure.

Included among the several erection drawings on a project are an anchor rod plan and, when necessary, a grillage plan and an embedment plan. The AISC *Code of Standard Practice* refers to these drawings as "embedment drawings" and defines them as "Drawings that show the location and placement of items that are installed to receive structural steel." The three types of drawings may be combined into one drawing. These drawings show steelwork that must be set in concrete and masonry prior to the erection of the steel frame. Often the anchor rods and embedded items are "deliver only" items for the fabricator. The concrete and masonry contractors require these drawings early in the project. To differentiate these drawings from those for the structural frame, some fabricators preface the drawing numbers with AR, such as AR1, AR2, etc. Other fabricators may refer to these drawings as embedment plans and preface the drawing number with EB. Still other fabricators have different yet similar practices.

Each plan is described as follows:

- The anchor rod plan locates all cast-in-place anchor rods, which will hold the steel frame to the foundations, and loose base plates.
- The grillage plan locates the position of grillages embedded in concrete foundations for steel columns. A grillage is a tier of closely spaced beams connected to each other to support a heavily loaded column. When more than one tier is required to support a column, additional tiers are laid across and connected to the tier below.
- The embedment plan locates steel plates and shapes in cast-in-place concrete and masonry walls to which structural steel eventually will be attached. Also, it may include the location of embedded plates or shapes to support metal deck, joists, grating, etc., depending on the fabricator's scope of work.

To save time, reproductions of the drawings made by the owner's designated representative for design are sometimes used as erection drawings, but only when permission to do so has been obtained in accordance with the requirements in AISC *Code of Standard Practice* Section 4.3. However, these drawings are generally not used for anchor rod, grillage or other embedment drawings. The practice of using reproductions should be limited to those drawings that print clearly, are to an adequate scale, are free of confusing and extraneous information, show the extent of individual shipping pieces by line breaks, and do not identify the originating design organization. Information not required for the fabrication or erection of the structural steel must be removed from the drawings.

The preparation of erection drawings should precede the detailing work. This helps to ensure that all required information is available. When a set of erection drawings is available at the start of a job, subsequent detailing group operations are made easier. When several members of the detailing group are involved in the preparation of shop drawings, a set of erection drawings can be used by the detailing group as a "mark-off" set. Such a set requires steel detailers to highlight each piece on the drawings as the corresponding detail is

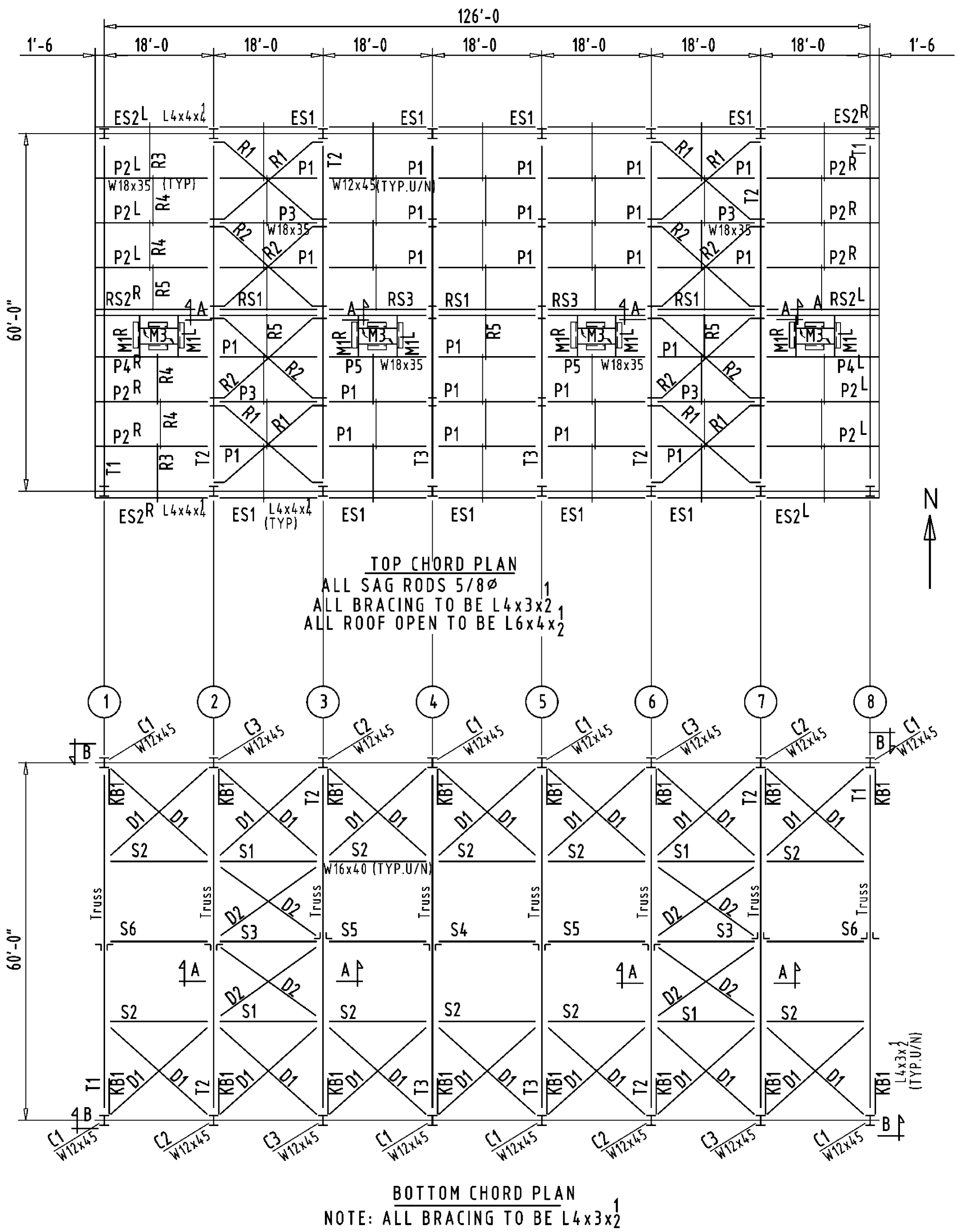


Figure 6-1a. Erection drawing.

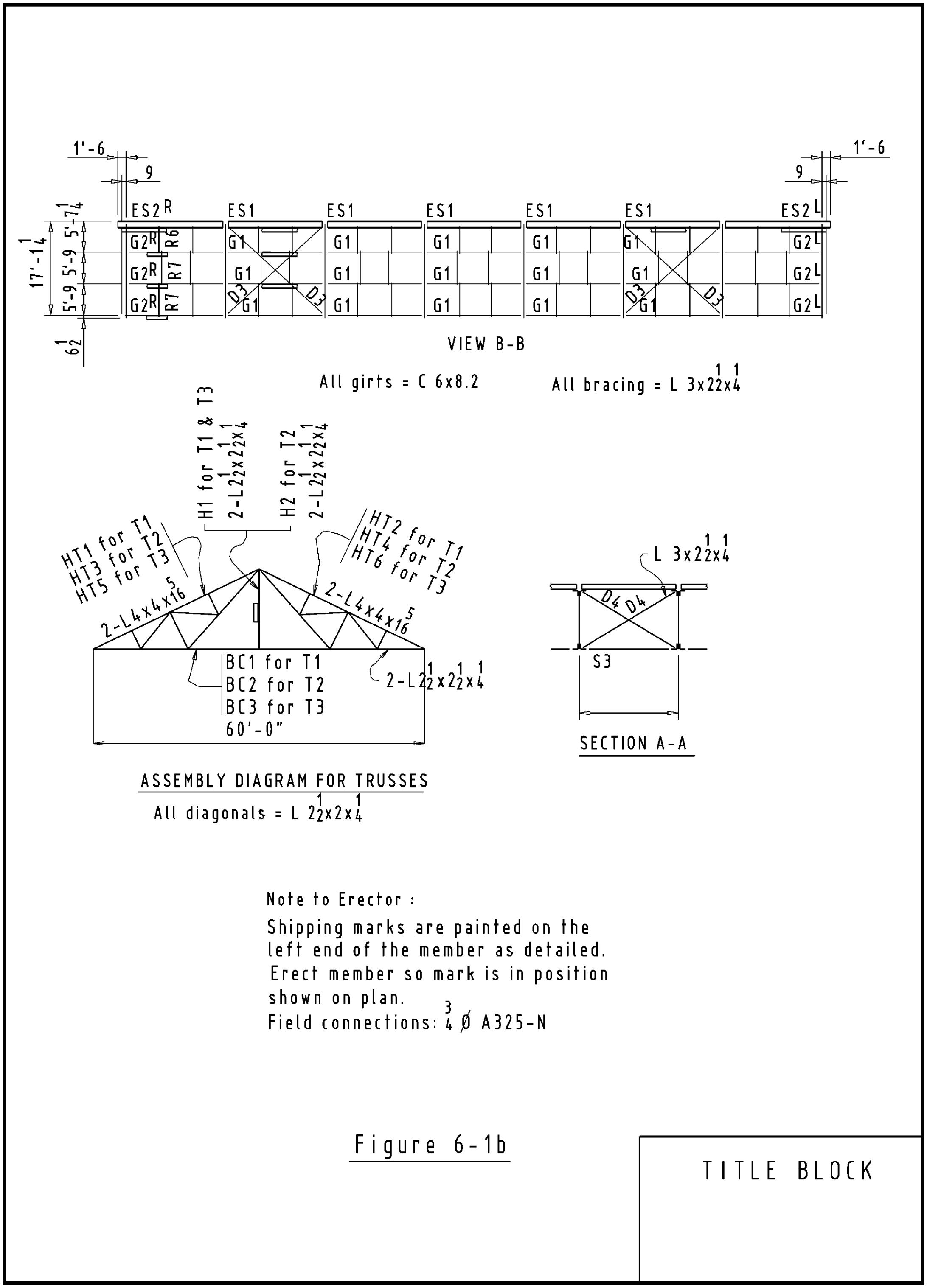


Figure 6-1b. Erection drawing.

completed. In this way the drafting project leader is able to monitor and report the progress of the detailing work to the fabricator.

When using a 3-D modeling program, a different system of preparing erection drawings is required as these drawings are an output of the database. Preliminary drawings must be plotted and checked for database accuracy. Erection drawings are produced after the 3-D model is complete, and the computer will often produce erection drawings with final marks. Field details are embellished sections and enlarged drawings of the database that give direction to the erector.

Generally, material provided by others that is to be installed on or attached to the steel frame is not shown on the erection drawings. Holes and cuts required by other trades are not provided unless required by the contract drawings and specifications (see AISC *Code of Standard Practice* Section 7.15).

The erection drawings show the areas into which the structure is divided for shipping and erection purposes (divisions or sequences). This permits scheduling delivery of the pieces required for each section to predetermined locations at the building site without unnecessary re-handling. Such advance planning ensures that raw material orders and shop details will be prepared in a sequence that will fit into an overall fabrication program planned to meet the required job delivery and erection schedules.

Some fabricators require that a general arrangement sheet, numbered A1, be prepared, especially for large projects. Initially, it will depict the general configuration of the structure and show column grid lines and column splices. This sheet will then be used by interested parties for divisioning, or sequencing, the structure. Copies of the completed sheet are issued to the shop for reference and issued to all concerned as a work guide.

Guidelines

The steel detailer may find the following “rules of thumb” useful in preparing erection drawings:

- Anchor rod, grillage and embedment plans and erection drawings usually are made to 1/8-in. scale. Details and sketches on the drawings are made to a larger scale suitable for clearly showing the required information.
- When anchor rod plans must show other cast-in-concrete members such as sill angles, pipe sleeves, trench angles, machinery foundation steel, etc., the information must be kept clear and uncomplicated for the concrete workers.
- Various types of erection drawings for the structural frame include floor plans, framing plans, roof plans, side and end elevations, bottom chord bracing plans, and crane runway plans and should include drawings of required sections and details.
- The compass north and project north should be shown on all plans. Preferably, the north arrow should be oriented in the same direction as shown on the design drawings.
- On an elevation view or building section, note the direction viewed (such as “Elevation Looking East”). Where sections cut along specific column lines or rows, these lines of rows should be stated (such as “Elevation along Line B”)
- When the shop drawings or clearances require a certain sequence of erection, the erection drawings must indicate the procedure required.
- The erector should be able to erect structural steel members without temporarily shifting supporting members from their final positions in the structure. If this is impossible, place a note on the erection drawing calling the erector’s attention to this situation.
- When the finished appearance of a structure (or portion of a structure) is under consideration, placing the field bolt heads on the exposed side may be required. In this case place a conspicuous note on the erection drawing to describe the locations of such bolts.
- In situations requiring field adjustment of members, on the erection drawings clearly show the method of adjustment and final location of the member.
- Members that are fabricated as Architecturally Exposed Structural Steel (AESS) must be identified on the erection drawings.
- Line work representing framing should be shown with heavy lines, the weight of which should contrast markedly with center and dimension lines. Leave a small space between the end of a beam and the member to which it frames to indicate they are separate members.
- When placing marks on the erection drawings, the mark should be placed on the end of the member corresponding to the marked end on the shop drawing.
- Instructions for making field connections, either by bolting or welding, must be given.
- If required, the starting point of erection should be shown.
- Members that are asymmetrical, such as angles and channels, should be shown or noted as to which way they are oriented. See Figure 6-8a.
- When locating channel members on an erection drawing, the dimensions should be given to the back of the channel, not to the centerline of the channel web.
- Erection drawings should be complete enough for the erector to assemble the structure without undue pondering or calls for assistance.
- To the greatest extent possible, sections and line elevations should be cut looking in the same directions.
- General Notes should indicate elevation of top of steel, grade of steel, type of field welding and field bolting and washer criteria.

Special Instructions for Mill (Industrial) Buildings

On mill building work, the following information should also be shown on the erection drawings:

- Typical cross-section of the building with general dimensions including height to top of crane rails, height to bottom of roof trusses, crane clearances, etc. Include the cross-section of crane runway girders indicating position of walk plates with respect to girder flanges.
- Cross-section of roof showing purlin spacing.
- Girt spacing including locations and sizes of wall openings for windows, louvers, exhaust fans, etc.
- In-to-in of door jambs and height to headers. Locate door openings with respect to columns.
- Show which way bracing shapes turn.
- On stairs and platforms, show which way stringers turn, if shipped knocked down. Locate stairs and platforms in plan with respect to columns.

Special Instructions for Tier Buildings

On tier building work, the following information should also be shown on the erection drawings:

- Except for isolated members, the elevation of the top of steel (TOS) of each floor is given by a general note such as: ELEVATION TOS = 62'-3 UNLESS NOTED. Exceptions are indicated by giving the top-of-steel distance above (+) or below (−) the stated elevation such as: W18×50 (+2) or W12×40 (−3½). If these beams are members in the floor at elevation 62'-3, the elevations of their tops are 62'-5 and 61'-11½, respectively.
- No ditto signs should be used on floor plans; all sizes (and elevations, if necessary) must be repeated. This eliminates potential problems in the event a design revision causes a ditto-noted member to be changed or eliminated, or a differently sized member to be inserted between two ditto-noted members.

Method of Giving Field Instructions

The following is information for the steel detailer to apply to erection drawings so that all parties involved in the project will know how the connections are made. This information appears under the "General Notes" section of the first erection drawing (E1, E101). Succeeding erection drawings may either refer to the notes on the initial drawing or repeat the notes. If the drawing represents a different phase of construction, the notes should be repeated.

Bolting

When field connections are to be bolted, the instructions that the detailing group must furnish to the erector are relatively simple. The number and location of fasteners required for each connection is obvious from the open holes provided to receive them.

High-strength bolts must be identified by size, specification and type such as: ¾"ϕ A325-N. If exceptions to the bolt identification are to be found on the project, the notation UN (Unless Noted) is placed behind the type designation. Then, locations on the project where the General Notes do not apply are called out to show the identification of the bolts to be used there. For instance, the General Note may read: "Bolts to be ¾"ϕ A325-N, in snug-tightened connections, UN." Perhaps, certain connections on the project are required to be slip-critical with ⅞"ϕ A325 bolts. On the erection drawings, where they are required, the slip-critical connections would then be noted: "⅞"ϕ A325-SC bolts." Other exceptions to the general bolt note may include connections using A490 bolts. Remember: if different grades of bolts are used on the same structure, they should be of different diameters. Otherwise, clearly and boldly mark all exceptions.

If the project requires the use of high-strength bolts in bearing with threads excluded from the shear plane, a detail such as shown in Figure 7-1 of the *Manual* must be given on the erection drawings. The steel detailer is referred to Chapter 3 of this text for additional information regarding "X-type" connections.

A situation that should be avoided to the greatest extent possible is one in which bolts must be installed in a particular sequence in a connection. When necessary, the sequence of installation must be given on the erection drawing.

In addition to the foregoing, the bolting instructions must address the use of galvanized bolts, nuts and washers, if any are required on the project. Also, the instructions must note the method of installation for high-strength bolts (e.g., snug tightened or pretensioned); the required installation method for pretensioned, if applicable (e.g., turn-of-nut, calibrated wrench, TC bolt, or DTI); and the requirements for washers (e.g., hardened, plate, or beveled), if required.

Welding

Instructions on the field weld size and location are conveyed to the field by means of weld symbols, notes and details showing the required work. By systemizing the manner of presenting this information, the number and complexity of details can be kept to a minimum, yet provide complete information.

The information on field welds must be made available to the owner's designated representative for design at the same time that drawings are submitted for approval. Hence, preparing field weld details in advance is necessary. These details should be on erection drawings at the time they are submitted for approval with the shop drawings. For example, if a shop drawing shows a beam end prepared for field welding, the approver needs to know what kind of weld is intended.

To cover the field welding required for a beam seated in a column web, a drawing similar to that shown in Figure 6-2 would be sufficient. In this case, the detail and welding symbols show the size, type and extent of field welding of the top clip angle. Note that the length of weld is not given because the full length of the top angle is to be welded. However, if this is not the case, information must be given concerning the length of field weld required. The classification of the welding electrode is shown in the General Notes on the first erection drawing.

Figure A6-3 in Appendix A is a shop detail drawing of a tier building column D4(0-2) with all fittings shop welded. These details are representative of a commonly used moment connection whereby the web of the beam is field bolted with high-strength bolts to a single-plate on the column and the flanges are field welded to the column. The *Manual*, Part 12 provides an illustration of this type of moment connection. A variation of this type of connection is one in which the single plate is field welded to the beam web. To hold the beam in position while welding, two bolts are used to connect the beam web to the plate.

To facilitate erecting the beam and field welding the beam flanges to the column web, a stub is fabricated between the column flanges. The stub consists of a pair of stiffener plates between the flanges and tied together by a single plate. The stub is extended approximately 1 in. beyond the toe of the flanges to eliminate the effects of triaxial stresses. Often the lower stiffener is made 1/4 in. thicker than the upper stiffener to compensate for beam overrun and underrun. The *Manual*, Part 12 illustrates this type of moment connection.

Another type of field-welded moment connection a steel detailer may encounter is illustrated in Figures 6-4a and b.

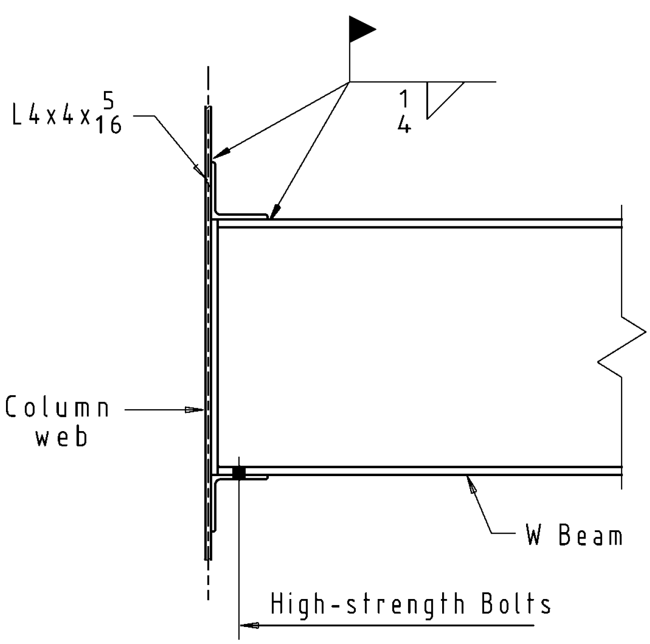


Figure 6-2. Seated connection.

Moment plates are shop-welded to the column flange. Prior to welding these plates to the beam, the erector will clamp and draw each plate to the beam flange. Top plates M21 and M22, shown detailed in Figure 6-5, are received at the site as loose pieces. Note in this example that in field welding the top plates to the beam flanges, some space along the edges remains unwelded. This is done purposely to permit the plates to lengthen or compress slightly as the beam deflects under load or wind moment. The detail shown represents a Type PR (partially restrained moment) connection.

Note that plates M21 and M22, Figure 6-5, are detailed with a width that is 1/8 in. less than the inside dimension of the column flanges for erection clearance. The curved transition is started about 1 in. from the flange toe to avoid a potential stress riser, which can develop at an abrupt change in section such as this. Although placing details and instructions on the erection drawings to which they refer is preferred, lack of space may require their location on separate sheets. This is true, particularly, on large jobs or on those having many special connections. The examples shown in Figures 6-6 and 6-7 illustrate this practice.

In Chapter 3, information is provided on how to handle weld shrinkage in field-welded moment connections. When this condition is present on a project and the beams or girders are fabricated 1/8 in. longer than theoretically required, the erection drawing must have a note to this effect to alert the erector.

The steel detailer should refer to Chapter 4 for a discussion on methods of detailing economical field-welded joints. Particular attention must be paid to detailing the preparation of edges of material for overhead field welding of complete-joint-penetration groove welds.

Locating Marks

The shop places erection marks on the left end of pieces detailed in horizontal or diagonal positions and at the bottom of pieces detailed in the vertical position. Therefore, placement of these marks on the erection drawings must follow the same system. This marking system, along with the fact that the marks are placed on steel to read right side up, enables the erector to position most of the members in a structure by referring to the location of marks on the drawings.

Some fabricators prefer to use variations of this system. For example, the compass direction is noted on some members, notably columns. Thus: "Mark Face A North." Likewise, members such as long girders or trusses, which cannot be turned at a job site, will require a compass direction on the appropriate end so it will be shipped that way (i.e., with the end pointed in the proper direction upon its arrival at the job site).

Referring to Figure 6-8a from the location of marks, members 101B1, 102B3, 103B1, 104D1, 104D2, 104D3 were detailed looking north. Similarly, member 101B3 was detailed

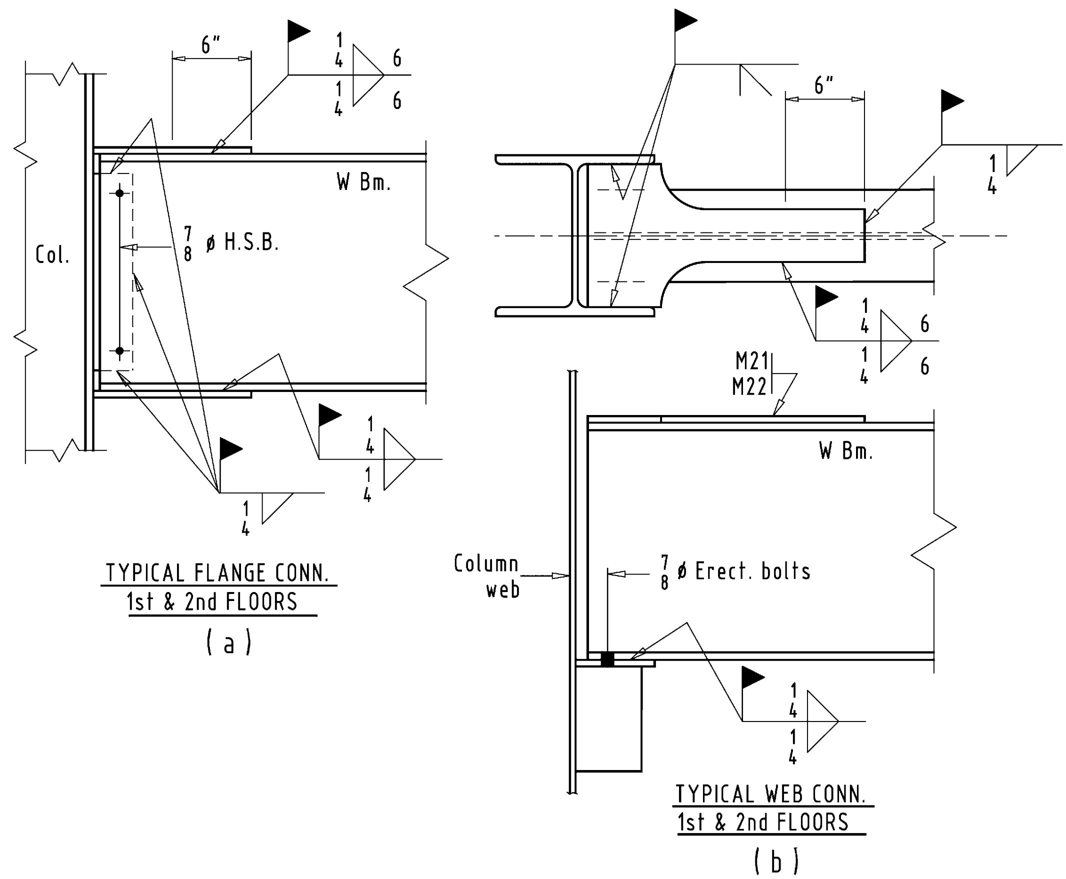


Figure 6-4. Field-welded moment connection.

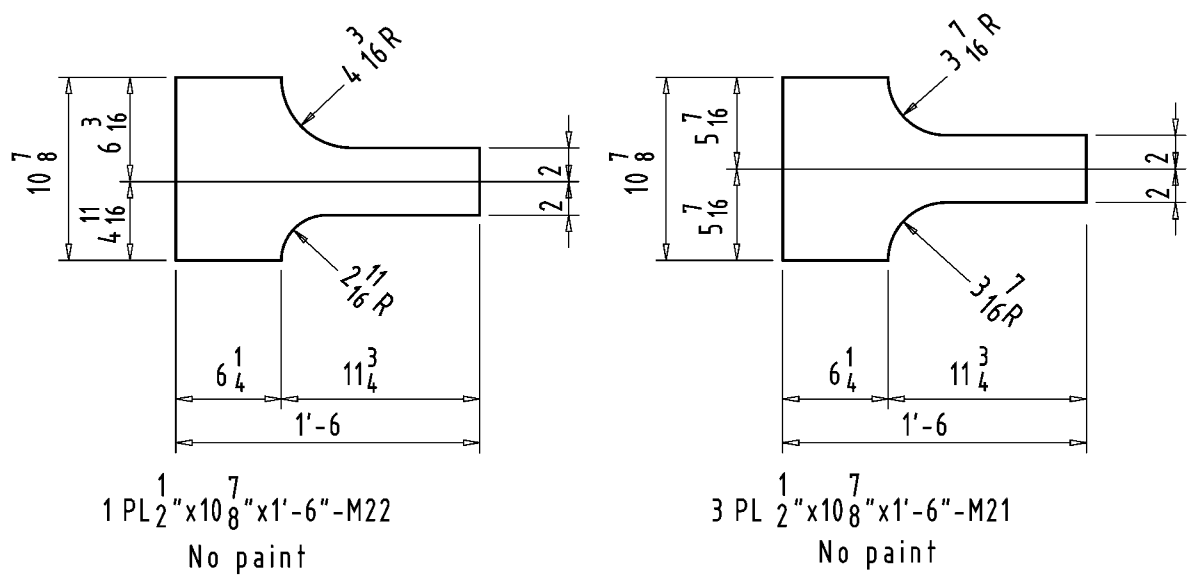


Figure 6-5. Plates M21 and M22.

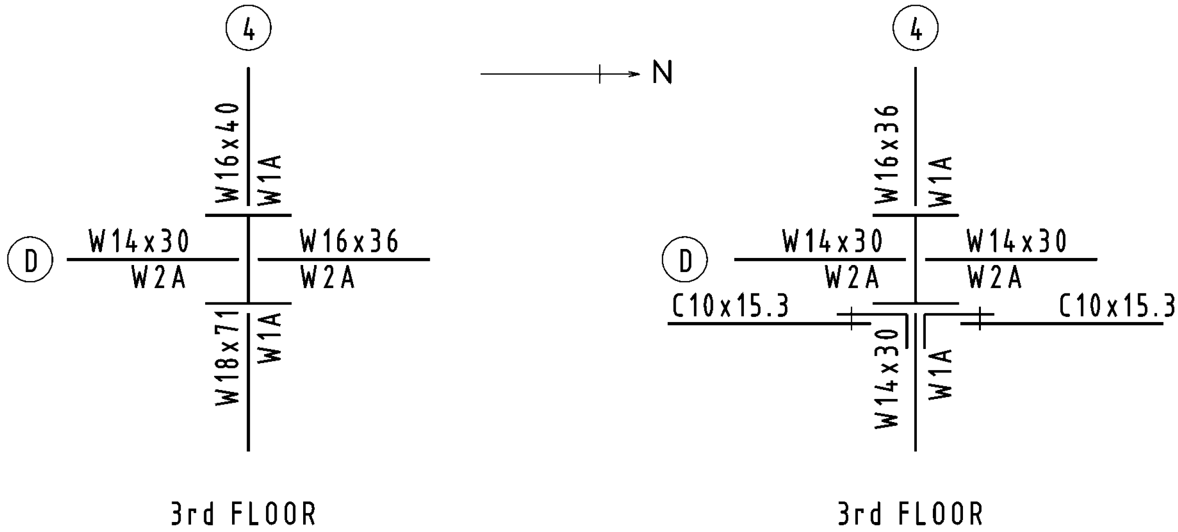


Figure 6-6. Special connections, plan view.

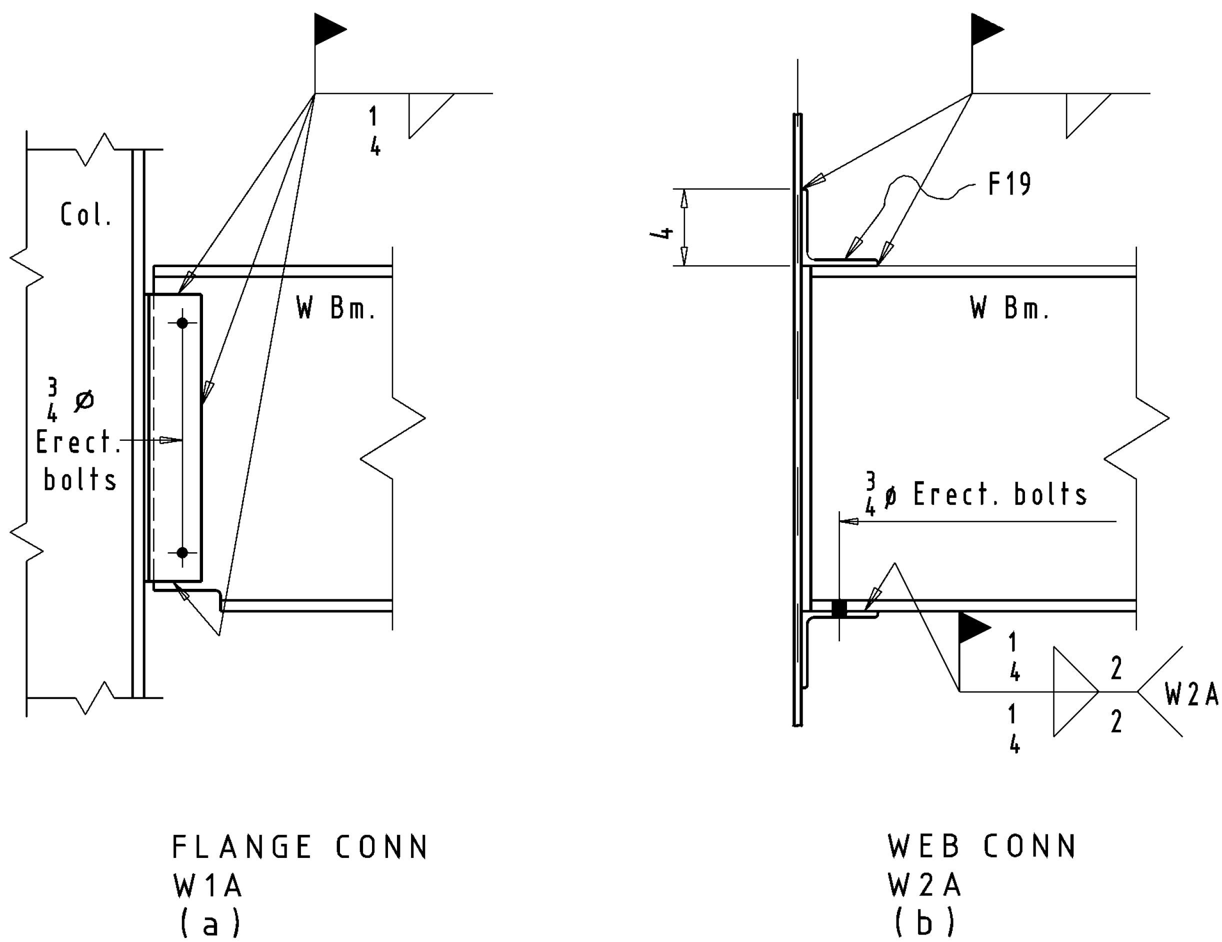


Figure 6-7. Special connections, details.

looking south while members 101B2 and 102B1 were detailed looking west.

Referring to Figure 6-8b, the steel detailer can see from the location of marks that all members in this elevation were detailed looking from the near side.

Erection marks should be shown on the drawings in bold lettering.

The steel detailer will note that, for purposes of clarity of Figure 6-8, the size and section identification of the members were omitted.

Field Alterations

Sometimes, an existing structure requires field modification or attachment of new connection material to receive new or revised framing. Therefore, in addition to erection drawings, field work drawings must be prepared. These drawings show sketches and details of connections to the existing structure. Also, they may be used as preliminary drawings to list field measurements, which determine the length and connection details of new members.

Field work may be done concurrently with the erection of material in a new structure when a revision is necessary on material already shipped from the fabricator's shop. Some guidelines for preparing such field work drawings are detailed here:

- Sketches need show only enough detail of the existing member to locate and show the new work.
- Location of an existing member must be described by reference to grid lines or some other easily accessible structure features. The original shipping mark on the member in all likelihood will not be visible.
- Details should show the member in its normal position in the structure and views described as "PLAN" or "ELEVATION LOOKING NORTH," etc.
- The recommended conventional practice is to draw present material with dashed lines and to show new work (such as cuts or holes) in heavy lines. These lines should be heavier than dimension and centerlines.
- To differentiate field work drawings from the erection drawings, sometimes the field work drawing numbers are prefixed with FW, such as FW1, FW2, etc. If the job is sequenced, the numbers may be FW101, FW102, FW201, etc. Another system ties the original shop drawing number to the field work drawing. In this case field work on members on original shop drawing 104 would be detailed on field work drawing FW104.

TEMPORARY SUPPORT OF STRUCTURAL STEEL FRAMES

The AISC *Code of Standard Practice*, Section 7.10 states that the contract documents shall identify:

- The lateral-load-resisting system and connecting diaphragm elements that provide for lateral strength and stability in the completed structure; and,
- Any special erection conditions or other considerations that are required by the design concept, such as the use of shores, jacks or loads that must be adjusted as

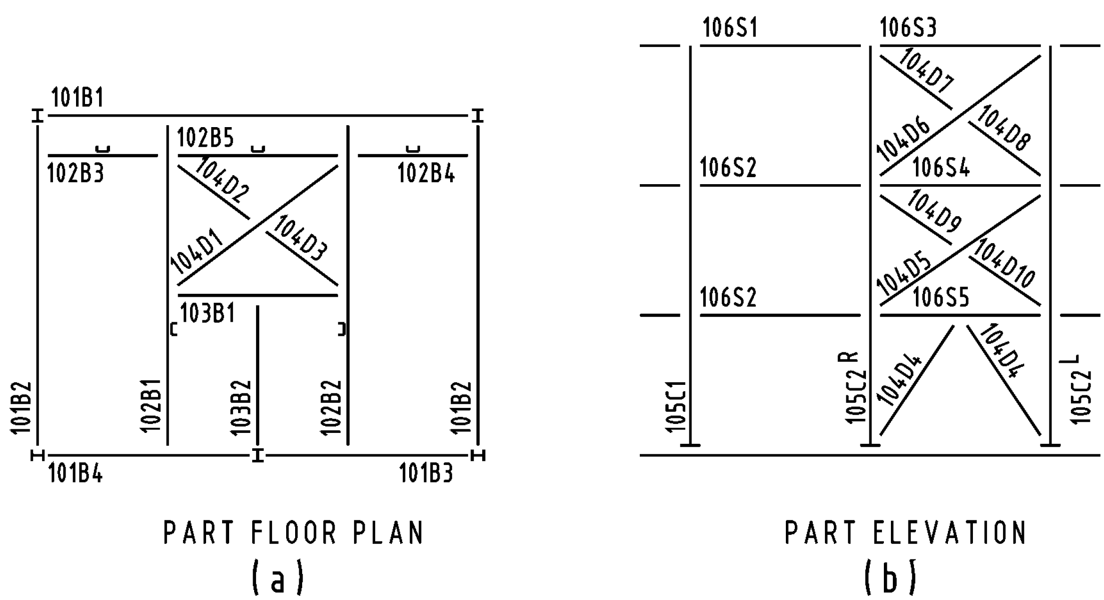


Figure 6-8. Erection drawing.

erection progresses to set or maintain camber, position within specified tolerances or prestress.

Having this information, the erector can determine, furnish and install all temporary supports required for the erection operation. These supports must secure the bare structural steel framing against loads expected to be encountered during erection.

If the design requires nonstructural steel elements to provide lateral-load resistance, the owner's designated representative for construction is required to indicate when these elements will be in place. Such elements may include, but not be limited to, roof and floor diaphragms of metal deck with or without concrete, concrete or masonry shear walls and precast spandrels. The owner's designated representative for construction must specify, prior to bidding, the sequence and schedule of placement of such elements and the effects of the loads imposed on the structural steel frame by partially or completely installed interacting elements. The erector furnishes and installs temporary support as necessary in accordance with this information, but does not, thereby, assume responsibility for the appropriateness of the sequence.

ERECTION AIDS

Erection Seats

When beams or girders with framing angles connect opposite each other and take the same open holes in the web of a column, as in Figure 6-9, consideration must be given to the problem of supporting one member while erecting the other to its final position. An erection seat, usually an angle, sometimes is provided in webs of columns. This erection seat is normally detailed to clear the bottom flange of the member it is to support by 1/8 in. to 1/4 in. to accommodate beam depth overrun and location tolerances. If the erection sequence is known, this erection seat only needs to be provided on the side requiring the support (i.e., the first side erected). If the erection sequence is not known or is doubtful, the seat can be provided on both sides of the supporting web. The erection seat should be sized and attached with sufficient bolts or weld to support the load on the seat.

By prior agreement between fabricator and erector, erection seats may be shipped loose. The erector is expected to weld or bolt these seats to the supporting column as required by the erection scheme. Erection seats are not generally required to be removed unless they create an interference or are unacceptable visually.

When beams are to sit permanently on seats in column webs, bolts or an equivalent attachment must be placed through the beam flange and the seat immediately after landing the beam. This is a mandatory OSHA rule—refer to the section on OSHA requirements in Chapter 2 of this text. Some erectors request bolts through the top flange (or upper portion of the web) of the beam and the top angle (or side angle), in addition to those through the bottom flange.

If the situation permits, when connecting to opposite sides of column webs, staggering the connections as shown in Figure 6-10 may eliminate the need for temporary seat angles. Furthermore, as an alternative to the use of framing angles and temporary seat angles, permanent seats should be

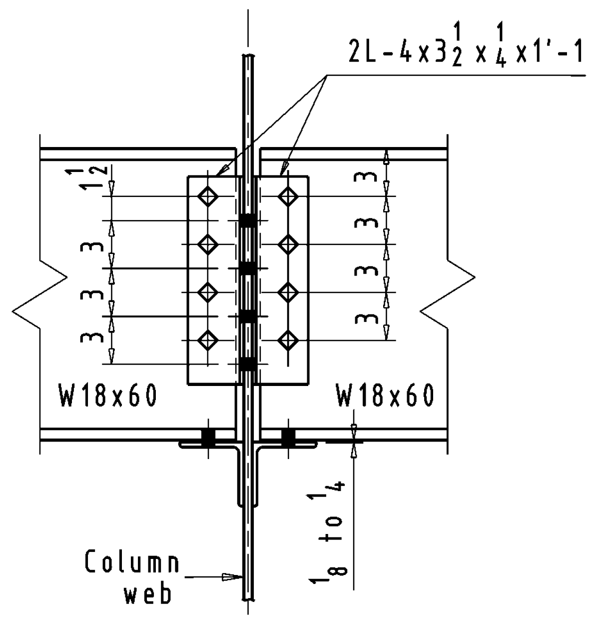


Figure 6-9. Beams with framing angles connecting opposite each other and taking the same open holes in the web.

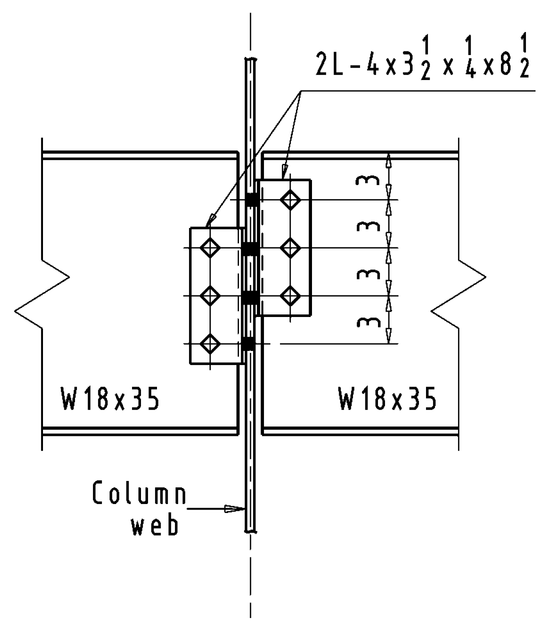


Figure 6-10. Staggered connections.

considered. See notes on Figure 6-10 regarding the sequence of erection.

Staggered connections to beam and girder webs may also be used to facilitate erection when double connection angles are used. See Figures 7-30 and 7-31.

Lifting Lugs

Lifting lugs may be required for members (columns, trusses and girders) exceeding a given weight, depending upon the capacity of the equipment of the erector. Also, lifting lugs may be required for members when their shape or erection conditions make the use of slings impractical. Under certain conditions, a lifting lug is used by the erector to facilitate handling columns at the site. Conventional splice plates on H-shape columns can be modified easily by adding holes to receive the pin of a lifting lug. H-shape columns spliced by groove welding may be provided with a lifting hole in the web, if permissible, or with holes in auxiliary plates that are bolted or welded in the shop so that they do not interfere with field welding. The erector furnishes to the fabricator the information for the lifting lugs to be used so that the steel detailer can provide any holes or weldments on the members needed to accommodate the lifting lugs.

Column Lifting Devices

In Figure 6-11 several types of devices are illustrated for lifting columns of various shapes. The lifting device and its attachment to the column must be of sufficient strength to

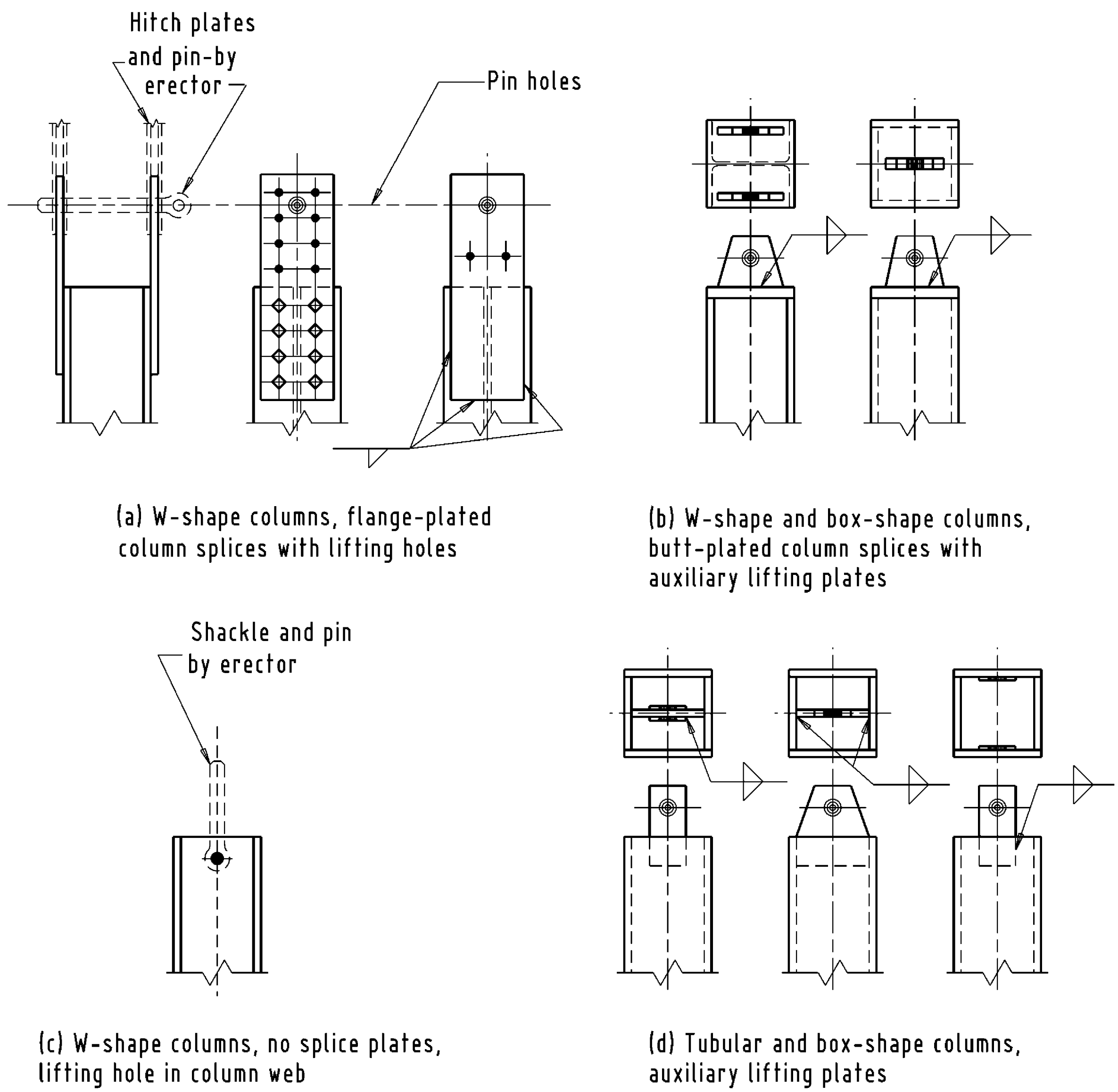


Figure 6-11. Column lifting devices.

support the weight of the column as it is brought from its delivered horizontal position to the erected vertical position. Tensile forces, shear forces and moments are induced in the device during handling and erecting. A shackle and pin are connected to the lifting device while the column is on the ground. The size and type of shackle and pin are determined by the erector, who passes the information to the fabricator prior to detailing. Often, a single pin and pinhole diameter are selected to accommodate all the steel members on a project requiring lifting devices. The size of the pin hole is based on the largest anticipated weight to be lifted. The pin is attached to the lifting hook, and a lanyard trails to the ground or floor level (see Figure 6-12). After the column is erected and connected, the pin is removed from the device by means of the lanyard. The shackle pin as assembled with the column must be free and clear so that it may be withdrawn laterally after the column has been landed and stabilized.

Column Stability and Alignment Devices

Figure 6-13 illustrates types of temporary lugs for field bolting columns to be spliced by welding. The material thickness, weld size and bolt diameter required are a function of the loading. The *Manual*, Part 14 recommends that the minimum plate or angle thickness is $1/2$ in. and the recommended minimum weld size is $5/16$ in. The available strengths of these minimums must be verified. High-strength bolts are used as fasteners. Usually, the temporary lugs are not used as lifting devices. Unless contract documents require their removal, these lugs may remain. Alignment of the columns is achieved by aligning the centerlines of the web and flanges of abutting column shafts. The centerlines are located on the columns in the shop by placing punch marks (centerpunching) on the centerline of the web and of each flange at both ends of each column shaft. See Note A in Figure 6-13. Note the $1/8$-in. gap between matching lugs shown in Figure 6-13.

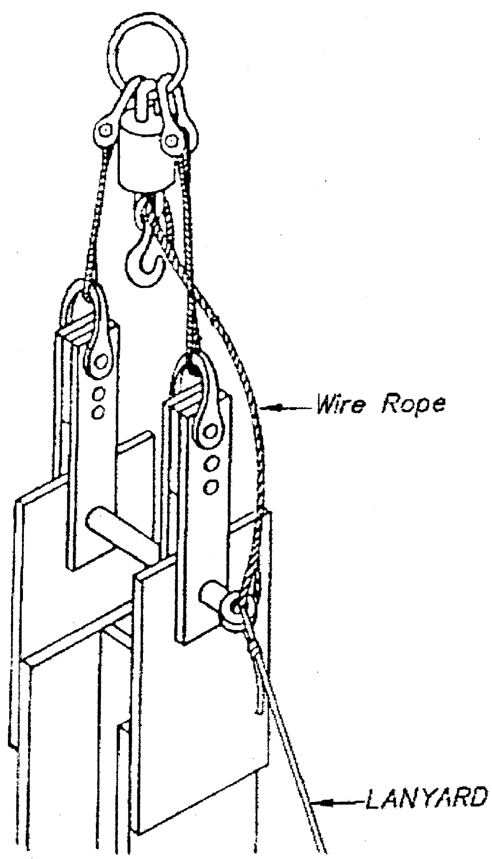


Figure 6-12. Lifting hook and lanyard.

SINGLE-PLATE, SINGLE-ANGLE AND TEE CONNECTIONS

If a framing member can be connected to either side of a single-plate, single-angle or tee, show clearly on the erection drawing which side of the single-plate, single-angle or tee the member is to be connected. This can be accomplished by either:

- Placing on the erection drawing a general note such as: "All beams bolted to east and north sides of single-plates, single-angles or tees unless shown otherwise"; or
- Indicating at each joint on the drawings the location of the single-plate, single-angle or tee with respect to each beam. This method is accomplished by placing a short dash mark (−), L, or T alongside the line representing the beam, the (−), L or T representing the single-plate, single-angle or tee, respectively. See Figure 6-14.

However, in lieu of this and wherever permitted, a paint stripe (striping) or paint spot (spotting) is placed on the surface of the connection that will be in contact (called the "faying" surface) when erected (Figure 6-15). When a member may be erected on either side of supporting framing, the surface to which the member frames is to be noted on the shop drawing of the connection: STRIPE NS or STRIPE FS. The shop will place a contrasting stripe of paint on the surface indicated. When striping or spotting is not permitted, erection drawings must be marked as noted earlier. Drawings must be noted clearly to guide the erector in correctly locating the member.

If the striping method is used in conjunction with slip-critical high-strength bolted connections, care must be taken that the stripe does not encroach on the clean zone surrounding the bolt hole group. The clean zone is illustrated in RCSC *Specification* Figure C-3.1.

MATCHMARKING

Occasionally, structural steel members that are too large or too heavy to ship are shipped in smaller sections (knocked

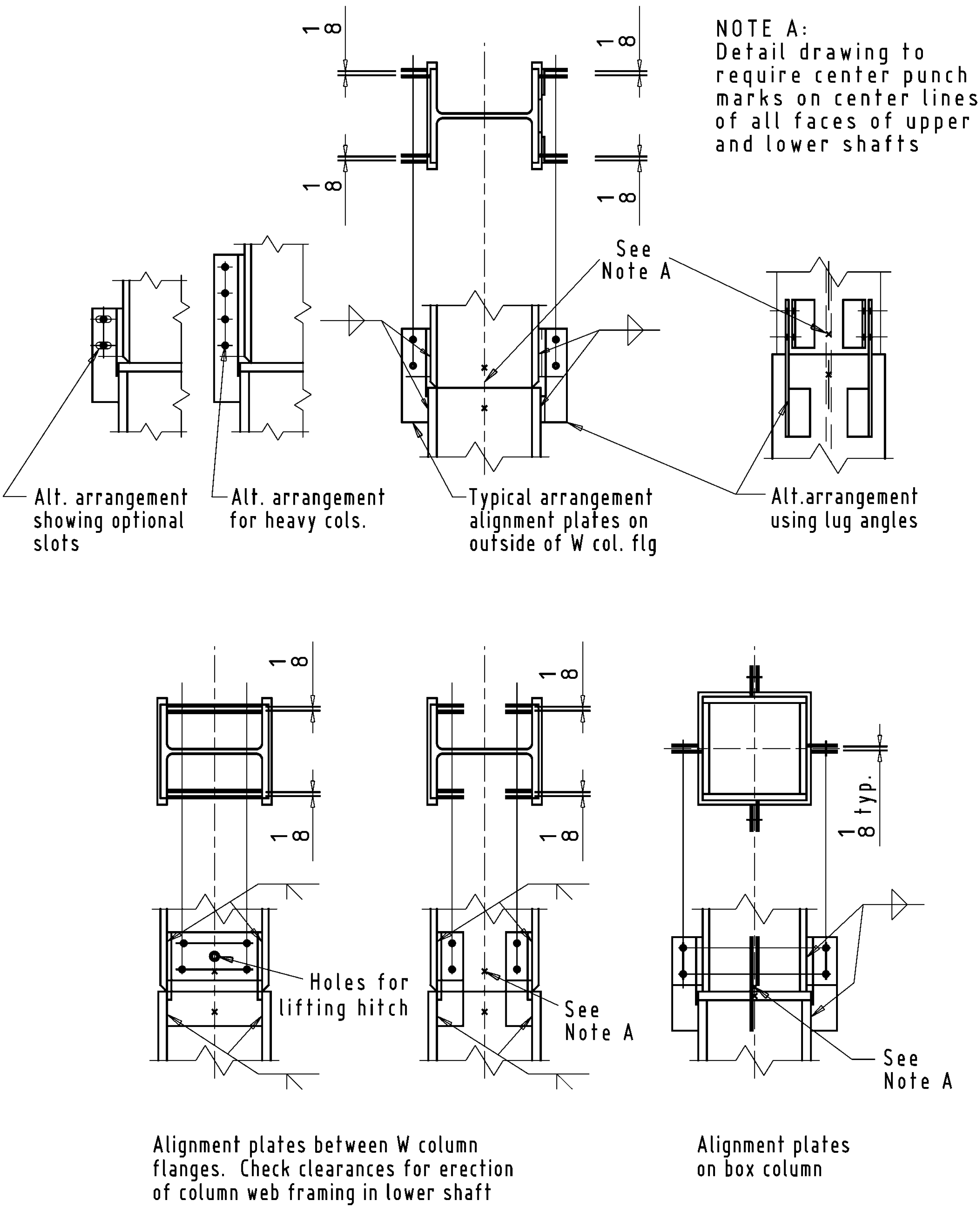


Figure 6-13. Types of temporary lugs for field bolting columns to be spliced by welding.

down). Prior to shipment these smaller sections may be assembled in the shop for fitting or reaming of field connections. Each field splice is matchmarked while assembled so that the sections can be reassembled in the field in the same manner as they were in the shop. The matchmarking system used is that preferred by the fabricator. Generally, the matchmark will be a paint mark with a numerical identification. However, if paint is not permitted, the marks may be stamped with steel dies. On the appropriate erection drawing, the steel detailer draws a diagram of the entire member, noting thereon the location of each splice and its matchmark identification. To assist the erector, a brief note should accompany the diagram to explain the system. A standard location of the stamped mark should be established for each job so the erector can minimize the time spent looking for the mark.

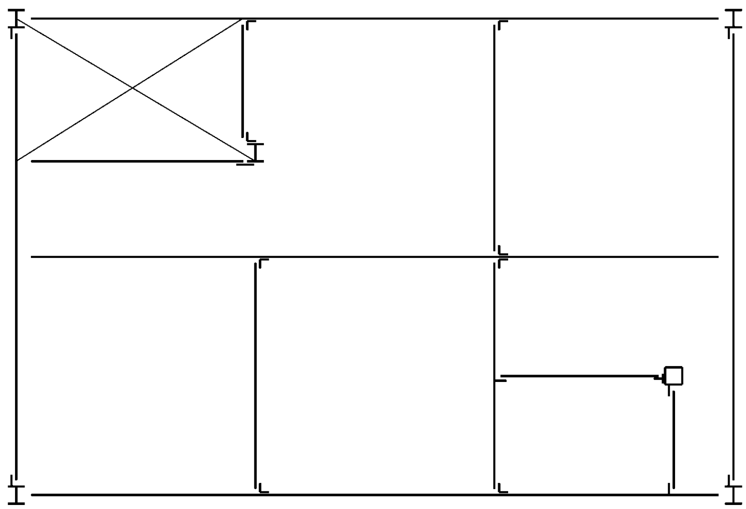


Figure 6-14. Indication of the location of the single-plate, single-angle or tee with respect to each beam.

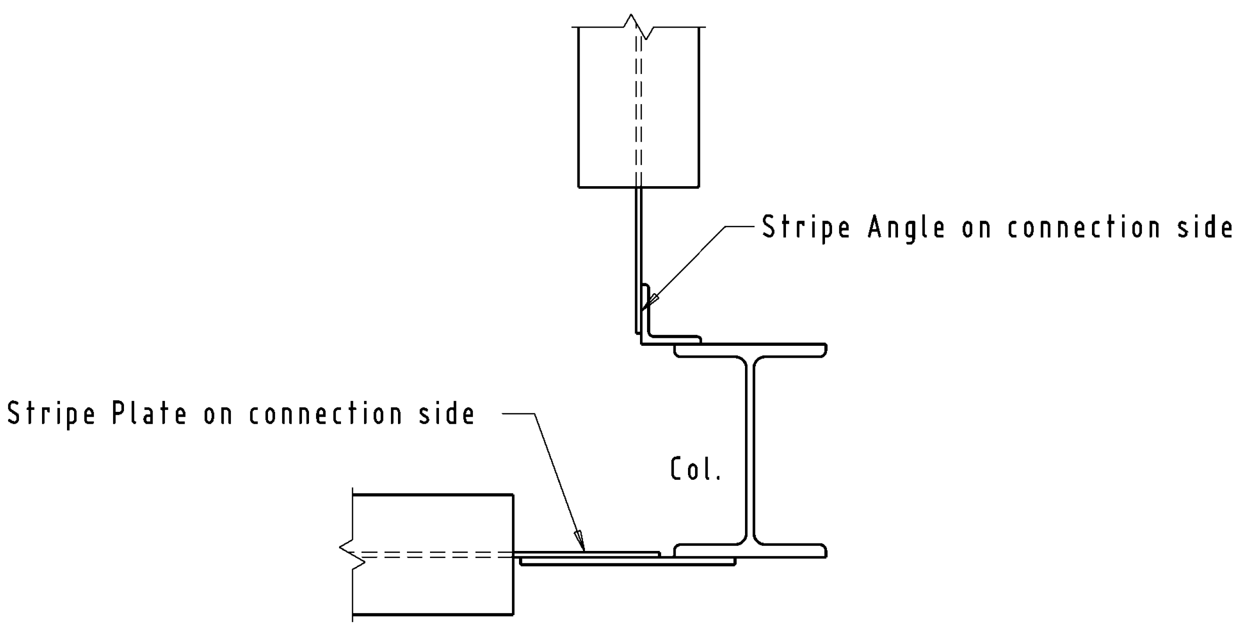


Figure 6-15. Striping or spotting.

CHAPTER 7
SHOP DRAWINGS AND BILLS OF MATERIALS

Guidance for the preparation of shop drawings and bills of materials.

Each of the many different types of steel structures requires many shapes and sizes of steel members. The task of detailing, fabricating and erecting this steel would be impossible without a well-established system to convey information from detailing group to shop to erector. The structural steel fabricating industry has developed appropriate practices and procedures in detailing that are commonly used throughout the industry.

In preparing shop details the steel detailer will make constant reference to the tables of "Dimensions" in the *Manual*, Part 1. These tables list the cross-sectional dimensions of rolled structural steel shapes, both in decimals and to the nearest $^1/_{16}$ in., the smallest unit of measurement to which a shop usually works. Following each page of "Dimensions" is a page of "Properties," which lists design properties with which the steel detailer normally will not be concerned, but with which the design engineer is vitally involved. Note that slight differences appear between the fractional dimensions and the decimal dimensions, resulting from "rounding off" the decimal dimension to the nearest $^1/_{16}$ in. The fractional dimensions are used for detailing and the decimal dimensions for designing.

The sketch at the top of each table identifies several clearance dimensions, which are tabulated under the heading "Distance." Dimensions T, k and k_1 are referred to as the "T-distance," the "k-distance" (both k_{des} and k_{det} are provided) and the "k_1-distance," respectively.

The examples of details shown in this text represent the generally acceptable methods of presenting information on shop drawings to fabricators.

ANCHOR ROD AND EMBEDMENT PLANS AND ASSOCIATED DETAILS

Although the construction of foundations is not normally a part of the fabricator's work, design foundation plans may show certain items that are in the fabricator's scope of work. These items may include anchor rods, leveling plates, base plates, grillages, machinery supports, lintels, curb angles and other embedments. Such items are ordinarily shipped in advance so the general contractor or concrete subcontractor can place them prior to erection of steel. They should be detailed on drawings separate from the remainder of the job.

The design anchor rod plan shows typical column base details. An example of such a plan is shown in Figure 7-1a; in this case it is small and not complicated and is included on the same drawing as the project design drawings. For a more extensive discussion on base plates and anchor rods, please reference AISC Steel Design Guide 1, *Column Base Plate and Anchor Rod Design,* 2nd Edition (Fisher and Kloiber, 2006).

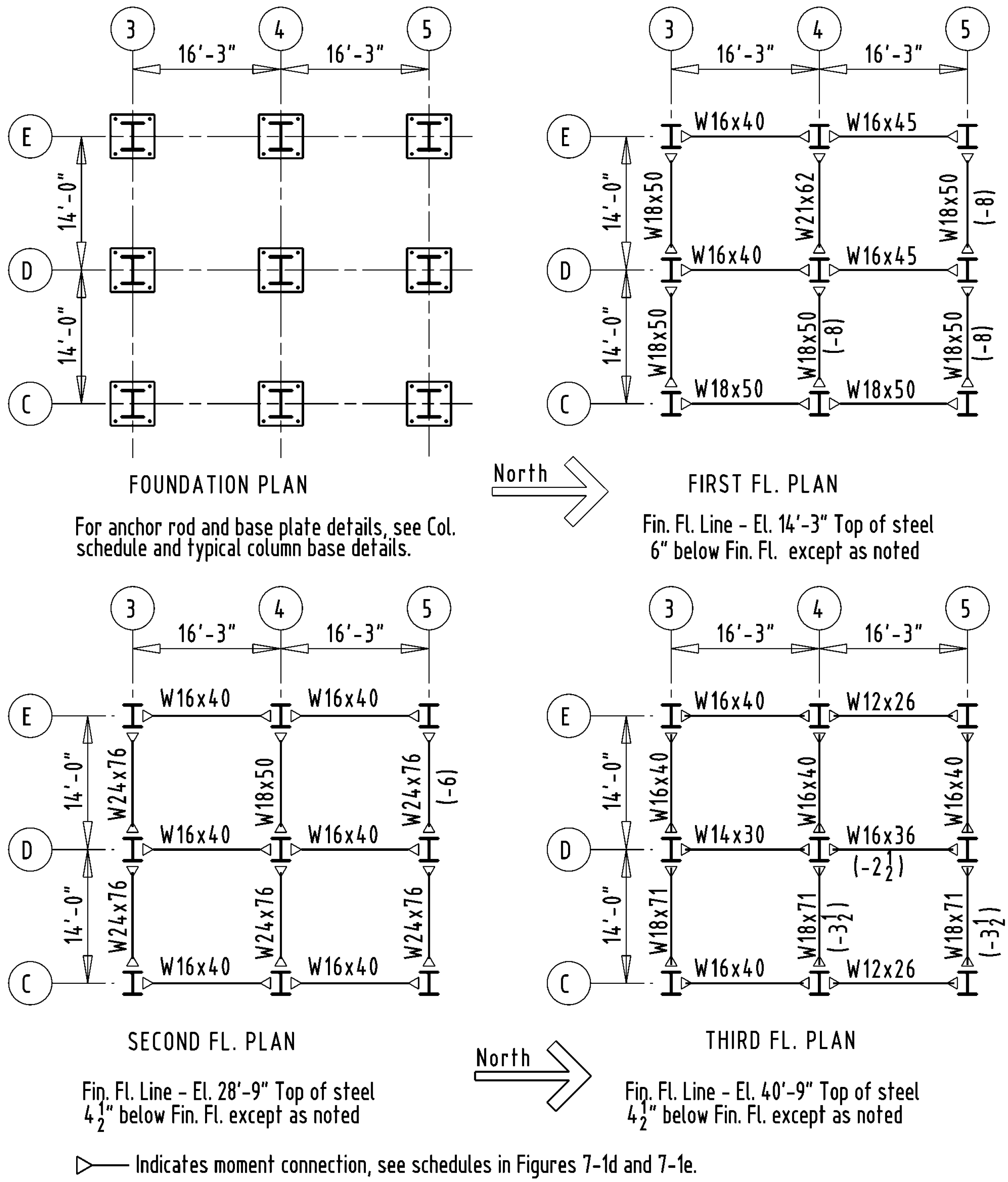
Anchor Rod Plans and Details

The steel detailer prepares an anchor rod plan concurrently with the details of advance material. This plan, which is similar in appearance to the design foundation plan, gives complete information for field placement of the advance material, such as anchor rods, leveling plates and loose base plates. It includes erection marks, elevations at the tops of base plates and leveling plates, grout thickness and the projection of anchor rods above the top of concrete. Although data such as the elevations of the tops of base plates and leveling plates can be taken from the foundation plan, they should be verified with information presented on the design drawings for the structural framing and with the elevations of the tops of footings. Also, the orientation and location of columns must agree throughout all tiers above the foundations. Strict attention paid to these details at the outset of a job will save much time and expense at a later date in the event an error is made.

Base plates, as shown in Figures 7-2a and 7-2b, are usually welded to the bottoms of columns in the shop. The preferred shop welding of the base plate to the column shaft illustrated in Figure 7-2b allows the welding to be completed without incurring an additional shop operation of turning the column over to weld the outside of the opposite flange. However, the weld must be adequate to resist the forces applied to the connection.

Typically, the top of the rough footing or pier is set 1 in. to 3 in. below the bottom of the base plate to provide for concrete tolerances and subsequent grouting. The difficulty of supporting such columns while leveling and grouting their bases requires that footings be finished to the proper elevation.

One approach is the use of a steel leveling plate, which is normally $^1/_4$ in. thick and the same size as the base plate it will support. Leveling plates lend themselves well to small- and medium-sized column bases up to about 24 in. If the leveling plate is also used as the setting template for the anchor rods, the holes are general standard size. If the leveling plate is set after placement of the anchor rods, the holes are sized in accordance with Table 14-2 of the *Manual* to enable them to fit over the installed anchor rods. Leveling plates are placed by



General Notes

Specification: AISC.
Material: A992 (W-shapes) & A36 (All other).
Shop paint: See project specifications.
Shop fasteners: 7/8" dia. Bolts, A325-N.

Field fasteners: 7/8" dia. Bolts, A325-N, except moment connections to be A325-SC.
Note: Use turn-of-nut method in tightening all bolts.

Figure 7-1a. Structural design drawing.

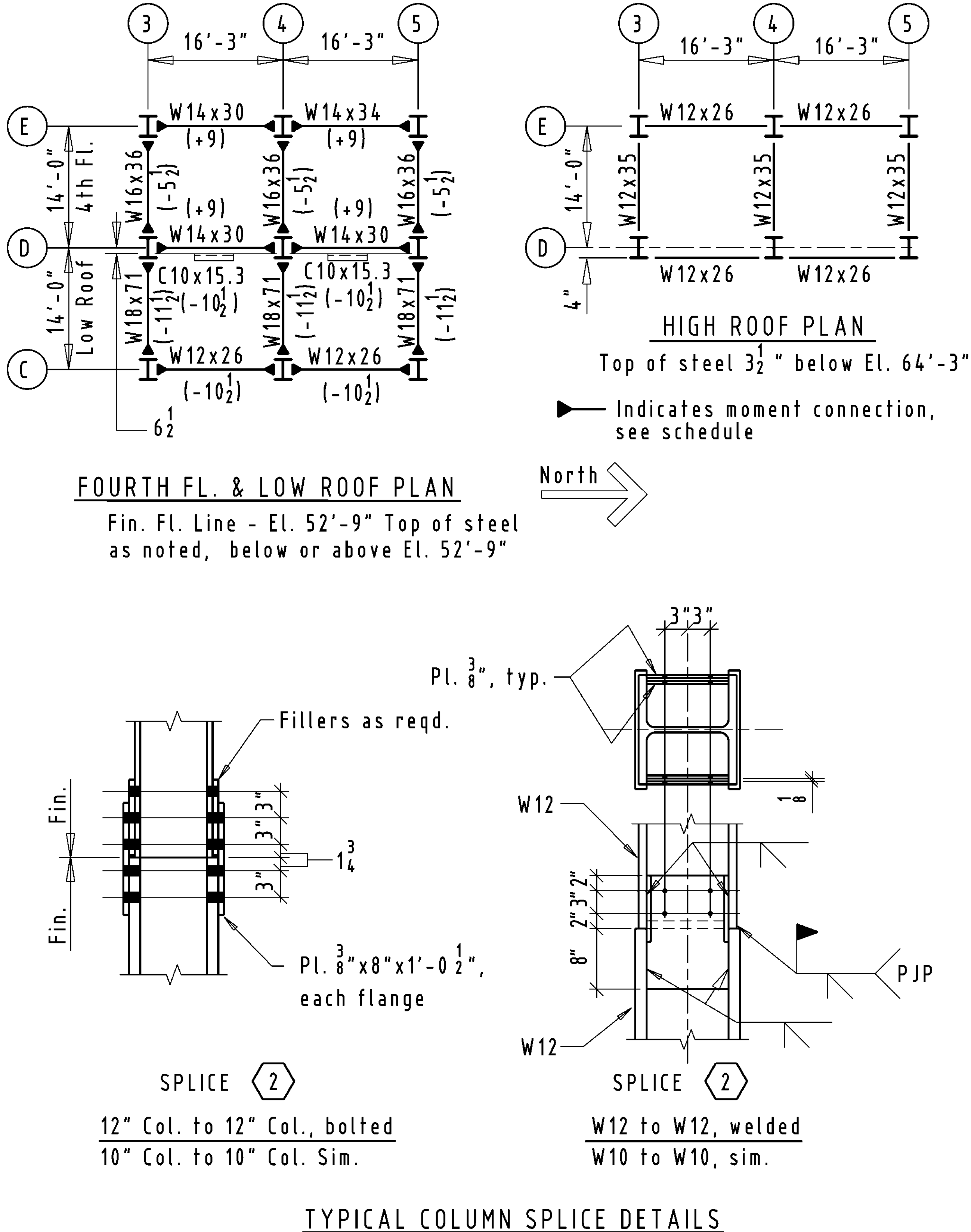


Figure 7-1b. Structural design drawing.

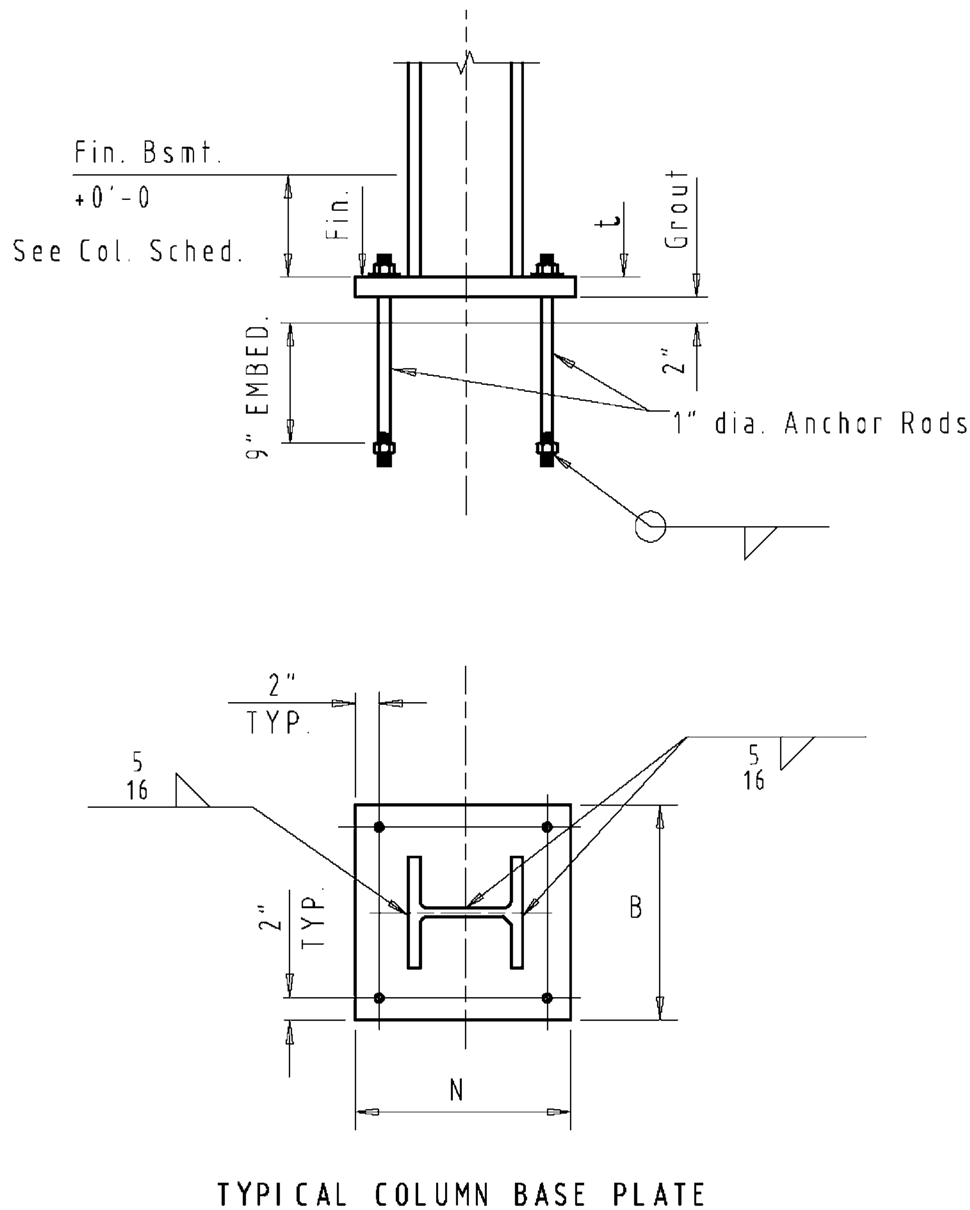


Figure 7-1c. Structural design drawing—column base plate details.

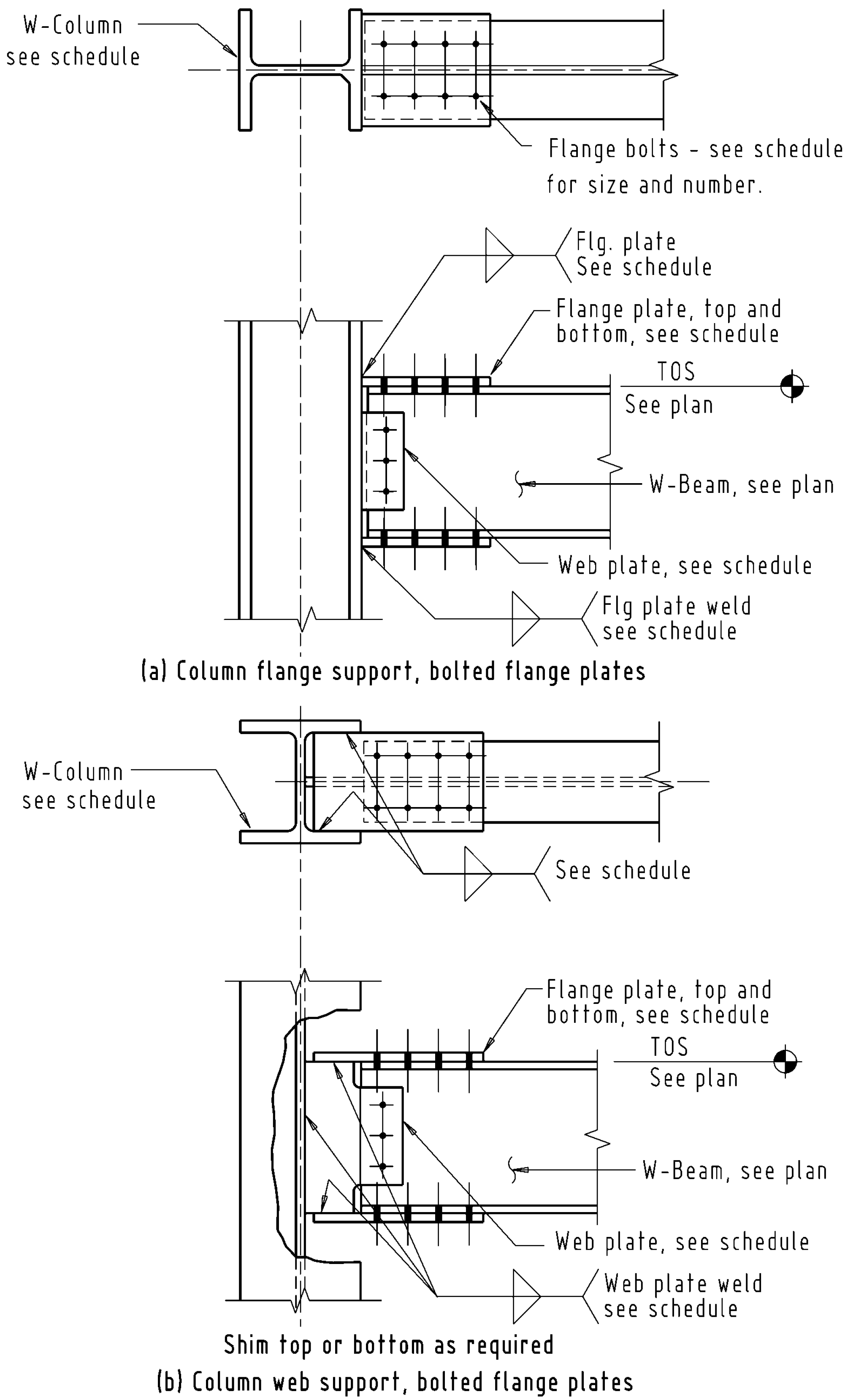
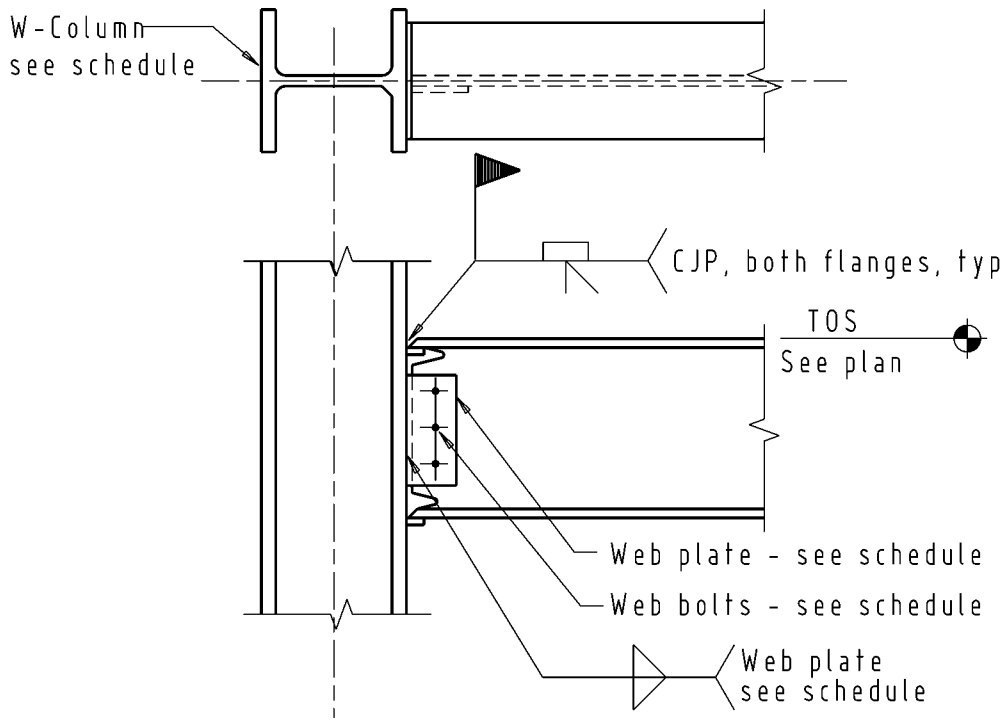
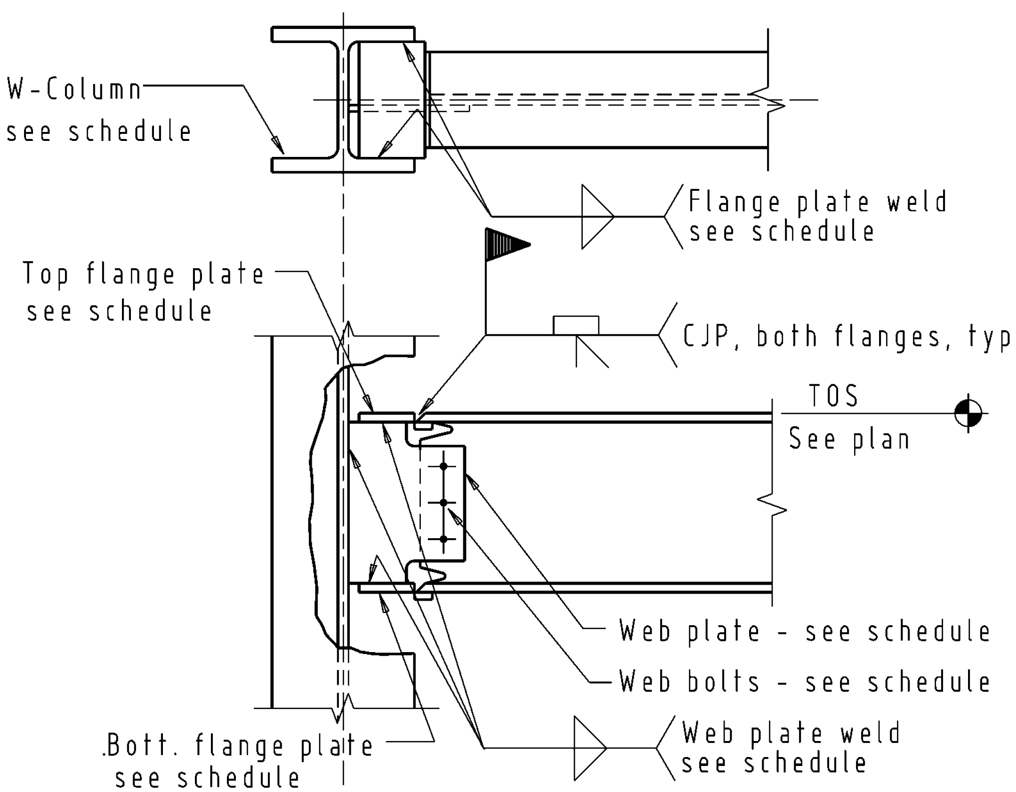


Figure 7-1d. Structural design drawing—bolted moment connection detail.



(a) Column flange support, welded flanges



(b) Column web support, welded flanges

Figure 7-1e. Structural design drawing—welded moment connection detail.

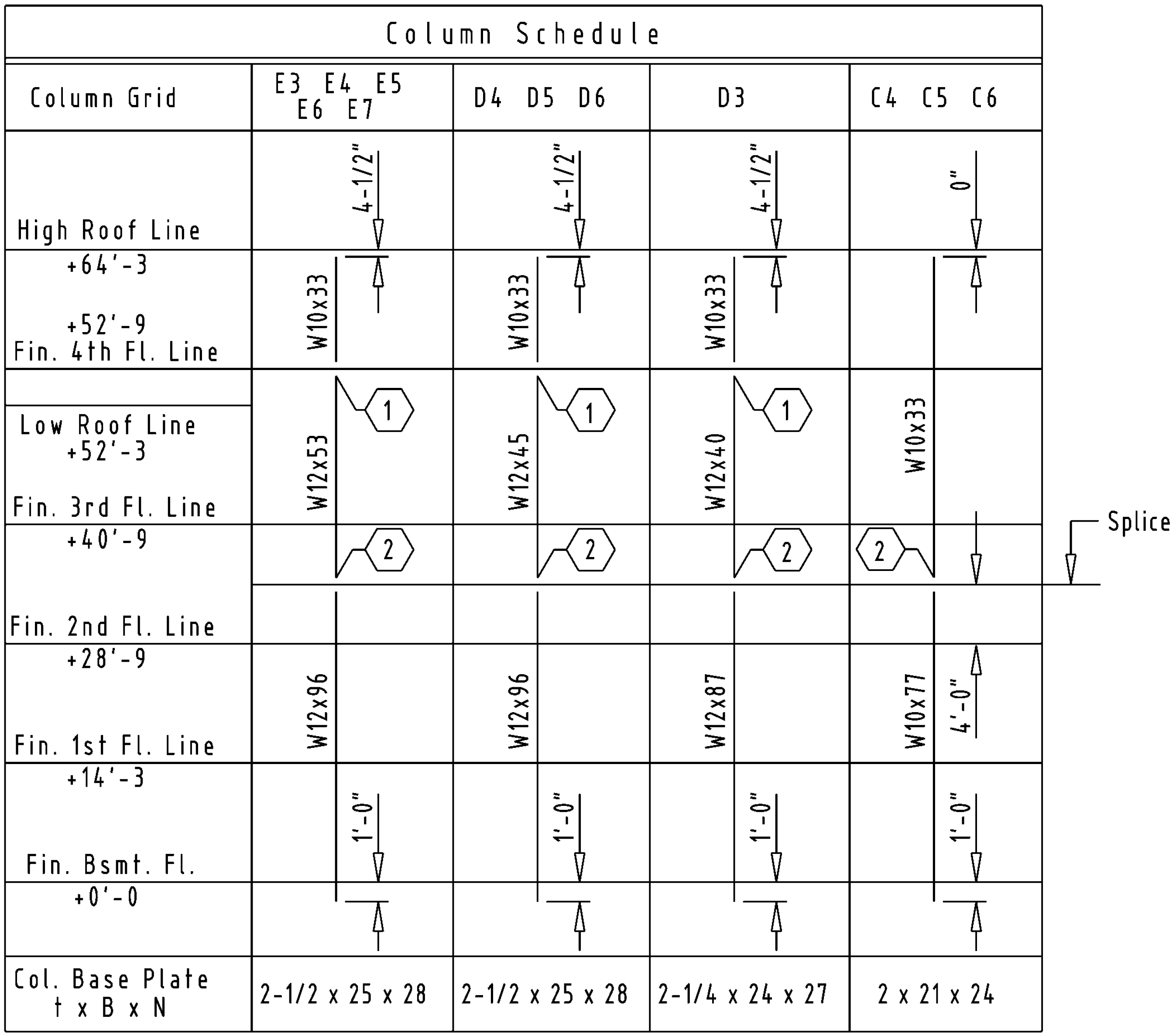


Figure 7-1f. Structural design drawing—column schedule.

Moment Connections (Welded)						
Floor Beam	Column Flange / Web	Web Plate			Flange Plate	
		Size	(No. Bolts) : Hole size	Weld	Size	Weld
W18x50	W12x96 F	3/8 x 4-1/2	(5):15/16x1-1/8	5/16	None	Field
W24x76	W12x96 F	3/8 x 4-1/2	(7):15/16x1-1/8	5/16	None	Field
W16x45	W12x96 W	3/8	(4):15/16x1-1/8	5/16	5/8 Top × 7/8 Bottom	5/16
W16x40	W12x96 W	3/8	(4):15/16x1-1/8	5/16	1/2 Top × 3/4 Bottom	5/16
W21x62	W12x96 F	3/8 x 4-1/2	(6):15/16x1-1/8	5/16	None	Field
W18x71	W12x45 F	3/8 x 4-1/2	(5):15/16x1-1/8	5/16	None	Field
W16x40	W12x45 F	3/8 x 4-1/2	(4):15/16x1-1/8	5/16	None	Field
W16x36	W12x45 F	3/8 x 4-1/2	(4):15/16x1-1/8	5/16	None	Field
W14x30	W12x45 W	3/8	(3):15/16x1-1/8	5/16	1/2 Top × 5/8 Bottom	5/16
W16x36	W12x45 W	3/8	(4):15/16x1-1/8	5/16	1/2 Top × 3/4 Bottom	5/16

W sections are ASTM - A992. Connection material is ASTM A36.

* Note: The thicker bottom plate is to allow for beam overrun/underrun of depth.

Locate "NO PAINT" areas where beam flanges will be field welded to column.

Figure 7-1g. Structural design drawing—welded moment connection schedule.

Moment Connections (Bolted)							
Floor Beam	Column	Web Plate			Flange Plate		
	Flange/Web	Size	(No. Bolts) : Hole size	Weld	Size	(No. Bolts) : Hole size	Weld
W18x50	W12x96 F	3/8 x 4-1/2	(5):15/16x1-1/8	5/16	1/2 x 9	(8):1-1/16 DIA.	5/16
W24x76	W12x96 F	3/8 x 4-1/2	(7):15/16x1-1/8	5/16	3/4 x 11	(12):1-1/16 DIA.	5/16
W16x45	W12x96 W	3/8	(4):15/16x1-1/8	5/16	1/2 x 11	(8):1-1/16 DIA.	5/16
W16x40	W12x96 W	3/8	(4):15/16x1-1/8	5/16	1/2 x 11	(8):1-1/16 DIA.	5/16
W21x62	W12x96 F	3/8 x 4-1/2	(6):15/16x1-1/8	5/16	3/4 x 10	(10):1-1/16 DIA.	5/16
W18x71	W12x45 F	3/8 x 4-1/2	(5):15/16x1-1/8	5/16	3/4 x 9	(10):1-1/16 DIA.	3/8
W16x40	W12x45 F	3/8 x 4-1/2	(4):15/16x1-1/8	5/16	1/2 x 7	(8):1-1/16 DIA.	5/16
W16x36	W12x45 F	3/8 x 4-1/2	(4):15/16x1-1/8	5/16	1/2 x 7	(8):1-1/16 DIA.	5/16
W14x30	W12x45 W	3/8	(3):15/16x1-1/8	5/16	1/2 x 10-7/8	(6):1-1/16 DIA.	5/16
W16x36	W12x45 W	3/8	(4):15/16x1-1/8	5/16	1/2 x 10-7/8	(8):1-1/16 DIA.	5/16

All W sections are ASTM A992. Connection material is ASTM A36.

Figure 7-1h. Structural design drawing—bolted moment connection schedule.

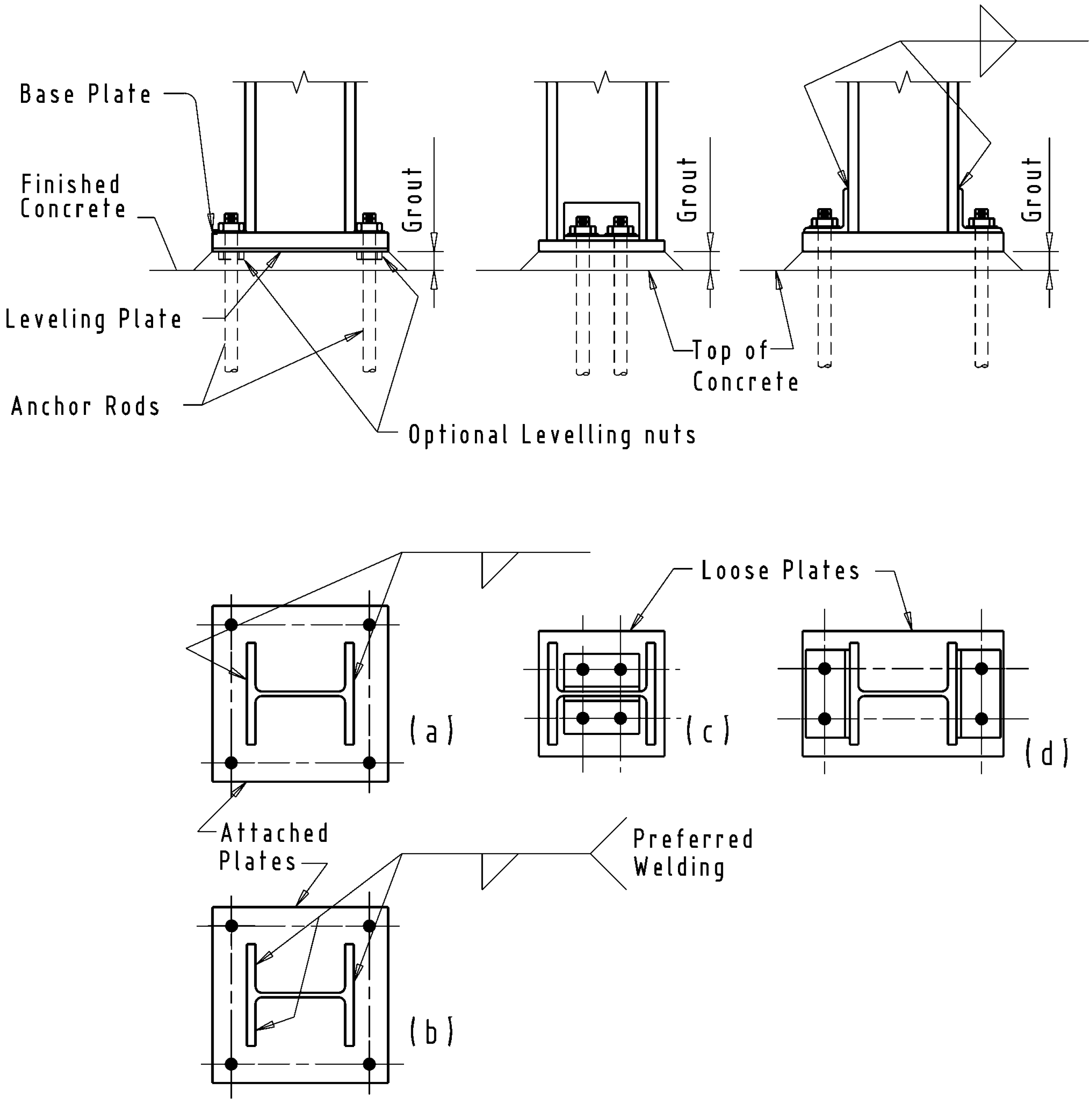


Figure 7-2. Column base plate details.

the general contractor or foundation sub-contractor (see Figures 7-2c and 7-2d).

When the use of a leveling plate is undesirable or impractical, four or more anchor rods can be utilized with each rod having two nuts and two heavy washers. The lower nut and heavy washer on each rod are set to the proper elevation; the heavy washer prevents the nut from pushing up into the hole in the base plate. At the time the column is erected, it can be lowered into place quickly and the upper washers and nuts installed. The base plate is to be grouted promptly after the structural steel frame or portion thereof has been plumbed.

As a third alternative, shim stacks can be used with four or more anchor rods. Properly sized and configured shim stacks [see AISC Steel Design Guide 10, *Erection Bracing of Low-Rise Structural Steel Frames* (Fisher and West, 1997)] are placed to the proper elevation to support the column. The individual shim stacks must be sized and arranged to properly support the imposed downward loads and the compression component of any column base moment induced prior to grouting. The individual shims should be graded in size to minimize the number required in each stack and the shims should be deburred so that they stack properly. Proper use of shims stacks may require tack welding the various plies of the shims to prevent relative movement during erection operations and when load is applied. Heavy washers and nuts are then installed on top of the base plate. The base plate is to be grouted promptly after the structural steel frame or portion thereof has been plumbed.

Generally, only very large base plates are shipped loose. Whether base plates are attached or loose depends on job conditions, as determined by the fabricator and erector. The loose plates must be lifted by a crane, set to elevation and leveled by the steel erector. This is accomplished by either using shims and wedges of varying thickness (see Figure 7-3a) or with three-point leveling bolts (see Figures 7-3b and 7-3c) (See also Base Plates, later). Proper use of shims and wedges may require tack welding the various plies of the shims to prevent relative movement during erection operations and when the base plate is applied. After the base plates are checked for line and grade, high-strength grout is worked under the base plate to ensure full bearing under the entire plate area. For large base plates (plates with a minimum dimension of approximately 36 in.) the design should call for one or more holes having a diameter of about 3 in. near the center of the plate through which grout is pumped under pressure to obtain an even distribution and prevent air pockets (see Figure 7-4). On the other hand, grouting holes may not be required when the grout is dry packed.

If the structural contract includes steel anchor rod setting templates, the fabricator customarily furnishes $^{1}/_{4}$-in.-thick plates. They are similar to leveling plates except that the overall size need be only large enough to include the bolt pattern and the diameter of the holes for the anchor rods is $^{1}/_{16}$ in. larger than the rod diameter.

Although a directly welded connection is more common today, for ordinary-size anchor rods, $1^{1}/_{4}$ in. diameter and less, heavy clip angles bolted or welded to the columns, as shown in Figures 7-2c and 7-2d, can generally be used to transfer overturning or uplift forces from the column shaft to the anchor rods. The clip angles shown in Figures 7-2c and 7-2d should preferably be set back from the column end about 8-in. to provide proper transfer of uplift forces from the structure to the foundation and to ensure that the column load is applied via the shaft and not the angles.

In lightly loaded structures, tall narrow framework, and mill buildings where crane loading is a factor, horizontal forces may tend to overturn columns or cause an uplift from the base. To resist these forces, anchor rods are used to tie the column to the foundation. Anchor rods also serve to locate and to prevent displacement or overturning of columns due to accidental collisions during erection. Observe that the plate stiffeners, capped with a plate, in the bases of Figures 7-3a and 7-3b are cut back about 1 in. from the base plate. This detail provides for proper transfer of uplift forces from the structure to the foundation. Also, it eliminates a pocket and permits drainage to protect the column base. These stiffeners are intended to resist uplift from an overturning moment and are not designed as part of the column area in bearing on the base plate. The column anchorage, including the anchor rods and the attachment of the column shaft to the base plate, must be capable of withstanding the base moment as set forth in OSHA safety regulations.

Shear forces can be resisted by friction between the base plate and the grout or leveling plate (induced by the compressive load in the column that is concurrent with the shear load, with proper consideration of the coefficient of friction between the materials), by shear through the anchor rods or through the attachment of a shear lug to the underside of the base plate (see AISC Design Guide 1, *Column Base Plate and Anchor Rod Design,* 2nd Edition, and AISC Design Guide 7, *Industrial Buildings,* 2nd Edition), which inserts into a groove or keyway in the concrete foundation (see Figure 7-5).

Table 14-2 in the *Manual*, Part 14 gives recommended hole sizes in steel base plates to accommodate anchor rods. The hole sizes given permit a reasonable tolerance for misalignment in setting the rods and permits more precision in adjustment of the base plate and column to their correct centerlines. The holes of the sizes indicated should be covered with a plate (structural) washer sized for the forces it must transfer. Minimum washer sizes are given in Table 14-2.

Insert sleeves sometimes are cast in the foundation. These holes are oversized and accommodate a threaded anchor rod,

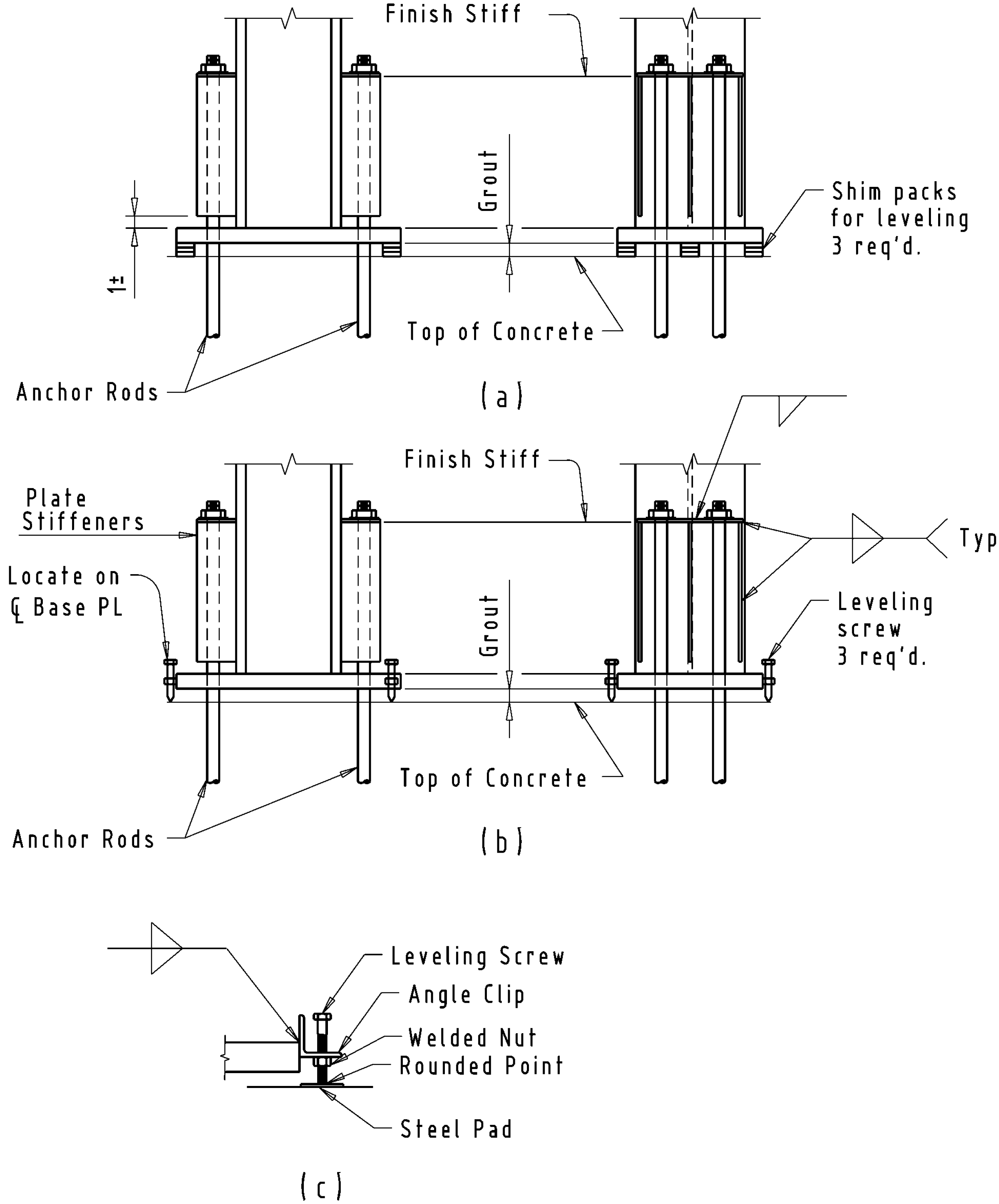


Figure 7-3. Leveling plate use.

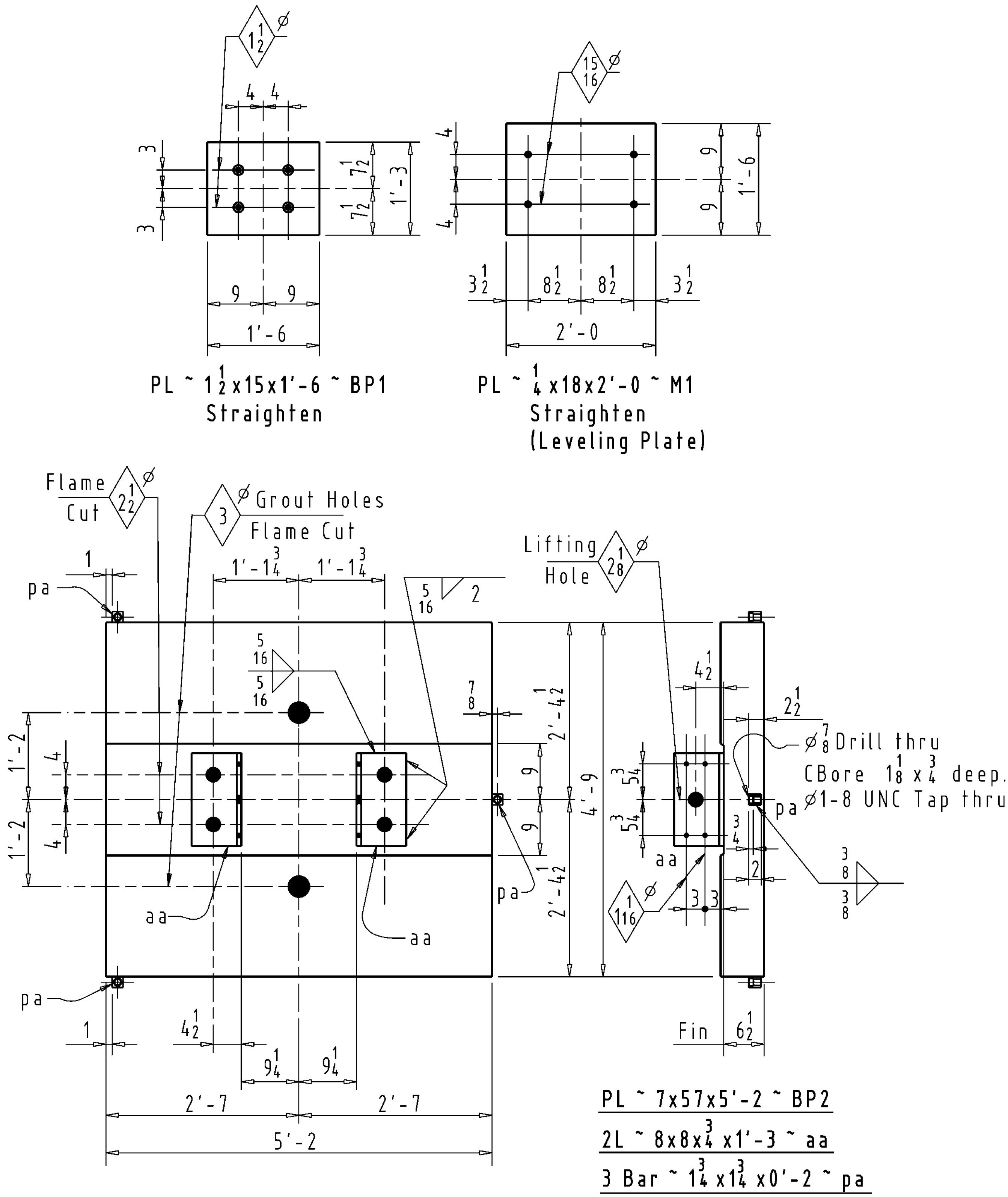


Figure 7-4. Base plate details.

which is grouted subsequently into the hole, usually with the piece installed in final position, see Figure 7-6c. Insert sleeves should be sealed prior to use in locations subject to freezing to avoid spalling of the foundation by the freeze-thaw cycle of water-filled holes.

Drilled in post-set anchors may also be used in foundations for column bases. These anchors are proprietary to their manufacturers and should be used and installed in a manner consistent with manufacturer's literature and instructions. Post-set anchors that rely upon torque or tension to develop anchorage by wedging action should not be used unless the stability of the column during erection is provided by means other than the post-set anchors.

Base Plates

In the absence of specific job requirements, the surface preparation of rolled steel base plates is governed by AISC *Specification* Section M2.8. This section stipulates that, if satisfactory contact in bearing is present in plates 2 in. or less in thickness, finishing is not necessary. Plates over 2 in. thick and not over 4 in. thick may be either straightened to obtain this contact or finished at the option of the fabricator. To ensure satisfactory flatness, unfinished base plates and leveling plates are noted "straighten" on shop detail drawings. Plates over 4 in. thick must be finished. However, finishing is not required on the underside of base plates when grout is used to ensure full contact on the foundations, nor on the top surfaces of bearing plates when complete-joint-penetration groove welds are provided between the column and bearing plate.

Figure 7-4 presents illustrations of typical details of base plates and leveling plates. Base plate thickness should be specified in multiples of 8 in. up to $1^1/_4$ in. thick, in multiples of $^1/_4$ in. thereafter up to 3 in. thick, and in multiples of $^1/_2$ in. over 3 in. thick. When finishing is required, as for BP2, the plate must be ordered thicker than the specified finished dimension to allow for the material that will be removed. Table 14-1 in the *Manual* Part 14 provides information on finish allowances for a variety of plate widths and thickness and for finishing one or both surfaces. These tabulated finish allowances are based on experience and have been proven satisfactory for structural work.

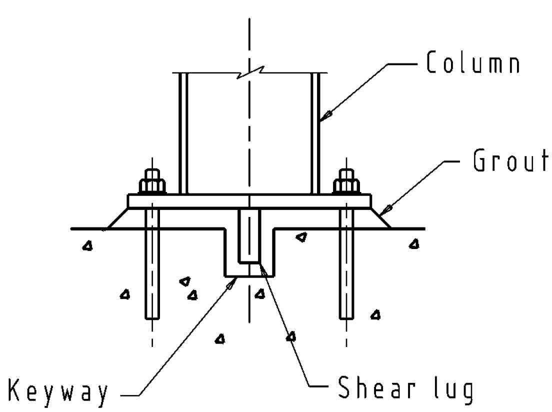


Figure 7-5. Keyway in concrete foundation.

As no useful purpose is served in finishing more than the area in contact with the finished end of the column, the shop detail is dimensioned to show the area on which finishing (Fin.) is required (see Figure 7-4). To reduce machine time, the cut should be made in the direction producing the least possible finished area. The finishing is usually carried across the full width of the plate to avoid interrupted machining operations, although it is not required from a design standpoint.

Holes are punched in base plates of a thickness that is within the machine capacity of the fabricator. Thicker plates must be either drilled or flame cut to provide the holes. Most fabricators are limited to a maximum drilling capacity of $1^1/_2$-in.-diameter holes. From a practical and economic standpoint the design should permit flame-cut holes when they are over 1 in. diameter in any thickness of base plate. The flame cutting operation produces holes with slightly tapered walls, and the holes should be inspected to ensure proper clearances for anchor rods. Grout holes do not require the same accuracy of size and location and generally are flame cut. Heavy loose base plates should be provided with some means of handling at the erection site. On BP2 in Figure 7-4, lifting holes are provided in the vertical legs of the connection angles.

Various means have been developed for use in leveling heavy (usually weighing 400 lb or more) base plates at erection. Three-point leveling is used, normally, because it is fast and sensitive to adjustment. Figure 7-3b indicates three leveling screws on the plate. A threaded attachment is welded to the base plate and may consist of a nut, a threaded bar as in Figure 7-4 or an angle and nut as shown in Figure 7-3c. The leveling screw must be long enough to compensate for the grout space and preferably should have the point slightly rounded to prevent it from "walking" as it is turned down. A small steel pad under the point reduces friction.

Leveling screws or shims are not intended to support the weight of the column. If grouting is to be delayed until after the base plates are checked for line and grade, adequate shim packs must be installed to distribute loads into the foundation without overloading either the base plate or the foundation.

Anchor Rods

The preferred material specification for anchor rods is ASTM F1554, which is available in grades 36, 55 and 105. In the *Manual*, Part 2, Table 2-5 lists a variety of other ASTM Specification materials that can also be used as anchor rods if availability is confirmed. Though clear with ASTM F1554, distinction should be made for other grades based upon what product can or cannot be obtained. For example, does the specification cover headed rod or threaded rod only? Are hooking or threading covered at all? It should also be noted that headed bolts generally are stocked in lengths up to about 8 in., depending on material specification. This length is too short for most column anchorage. Considerable delay and expense can be expected when nonstandard lengths and sizes are specified. Suitable nuts can be selected from ASTM Specification A563.

Threads of anchor rods can be produced either by rolling or cutting.

Anchor rods for structural work may take any of the forms shown in Figure 7-6. Perhaps the most commonly used are hooked rods, which are illustrated in Figure 7-6a. Note however that hooked anchor rods should not be used to resist calculated uplift forces. They are primarily for axially loaded compression members subject to compression only, to locate and prevent displacement and overturning of columns during erection due to wind loads or erection operations. High-strength steels are not recommended for use in hooked rods, since bending with heat may affect the strength of the rod material.

The nut shown in Figure 7-6d is acceptable, normally, in lieu of a bolt head. The nut may be attached either to a threaded rod or welded to a plain rod. If used with a threaded rod, the nut must be welded to prevent it from unwinding when the upper nut is tightened. However, welding should not be used if it is inappropriate for the particular steel specified. Generally, in the lengths and diameters required for anchor rods, headed rods are not readily available. Many shops do not have facilities for threading rods and, as a result, purchase the required anchor rods from suppliers.

Figure 7-7 shows typical details of anchor rods. An alternate method of detailing a rod, which is acceptable to most fabricators, is to show the rod as a single, heavy line. Note that no attempt is made to show conventional thread symbols as the shop will understand what is required by reading the notes. Because of possible inaccuracies in the setting of anchor rods, the distance H, shown in Figure 7-6c, should be sufficient to permit the rod to project a positive distance E, as much as

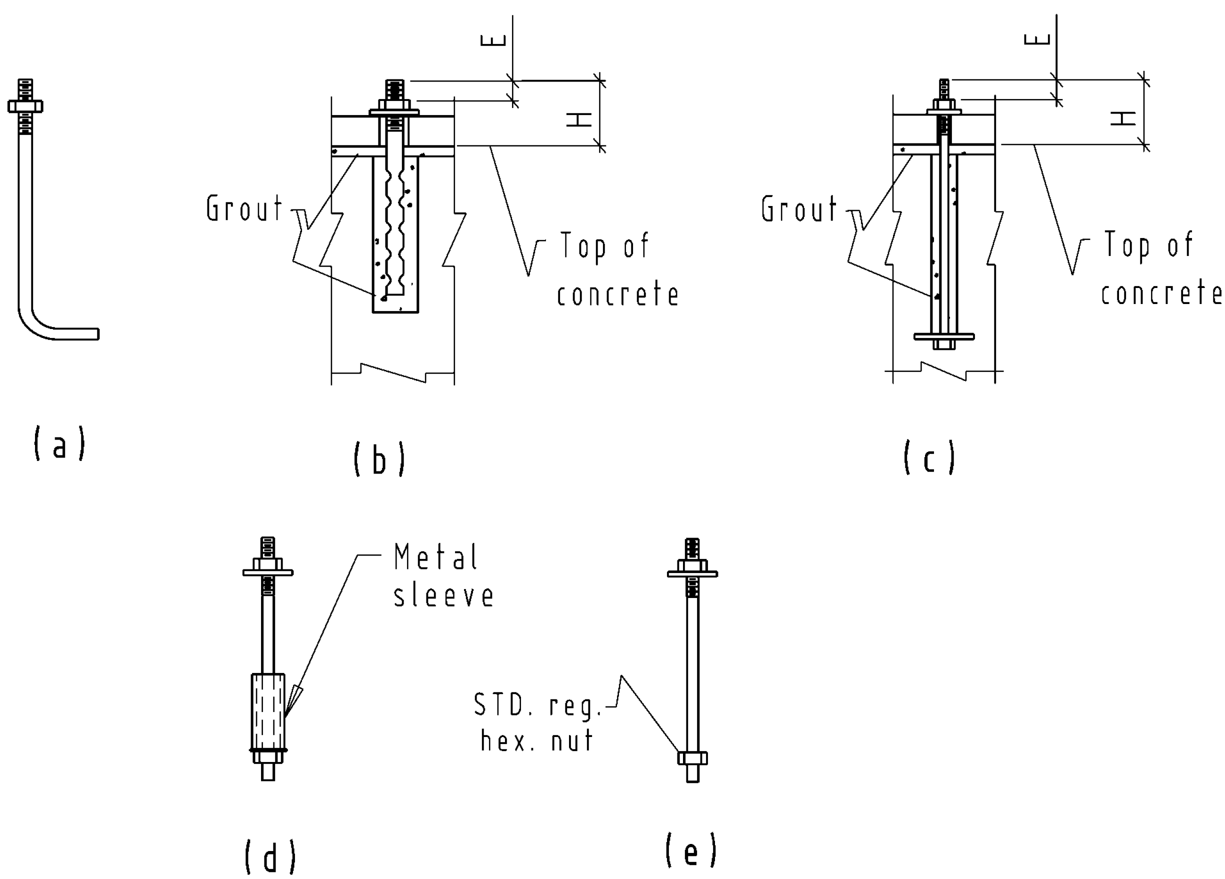


Figure 7-6. Anchor rods.

3 in., above the nut. Thread lengths, therefore, will be somewhat longer than the standard lengths furnished on regular bolts. Washers, which may be either round or square, will have holes 1/16 in. larger than the bolt diameter and will be furnished from ASTM A36 steel plates for most applications. Their use is required because of the large bolt holes provided in the base plate and column details. The thickness of such washers must be appropriate for the force to be transmitted.

Figure 7-6c shows an anchor rod set in a metal sleeve. Though rarely required, its advantage lies in the opportunity for some horizontal adjustment at the time the base plate is set in place.

Grillage

For extremely heavy loads in major structures, or where subsoil conditions are poor, a grillage as shown on Figure A7-8 (see Appendix A for all A7 figures) may be used. A grillage consists of one or more layers of closely spaced beams (usually S-shapes because of the thicker webs relative to the flange width) encased in the concrete foundation. When more than one layer is required to support a column, additional layers are laid across and field welded to the layer below. Note that the total grillage shipping piece includes the beams, separators, base plate and attachments for the column shaft.

Embedded Material

Embedded pieces, or "embeds," consist of a steel plate or shape to which headed concrete anchors are shop attached to anchor the piece into the side or top of a concrete or masonry wall. They come in many different configurations depending upon the type of support they provide (shear, tension/compression or bearing) and the loads applied to them.

Figure 7-9 shows some common types of embeds. Those illustrated in (a) and (b) normally would be set into the side of a wall to support a beam. The slots compensate for variations in the alignment of the wall surface. For types (a) and (b) the embedded plate is located vertically by giving the elevation of the top of the embed. It is located along the length of the wall by dimensioning to the vertical centerline of the plate. The steel detailer should locate the top or bottom hole in the single plate in type (a) by its elevation or by dimensioning the hole to a basic elevation in the structure. Along the length of the wall one face of the single plate should be located by reference to a working line (WL) in the structure. A working line is a reference line, usually a grid line in the structure, from which structural members are located.

The seat angle in type (b) will be located vertically by giving the elevation of the top of the angle. The piece will be located along the length of the wall by dimensioning the centerline of the seat to a working line.

Although type (a) is shown with field-bolted connections, it is suitable for field welding. However, OSHA requires erection bolts for field-welded connections. Type (b) requires standard or slotted holes whether the permanent connection is field bolted or welded.

Types (c), (d) and (e) are used for beams bearing on tops of walls. In type (c) the beam is set on the plate and its bottom flange field welded to the plate. The centerlines of the plate are located on the plan with respect to the centerline of the wall and a working line.

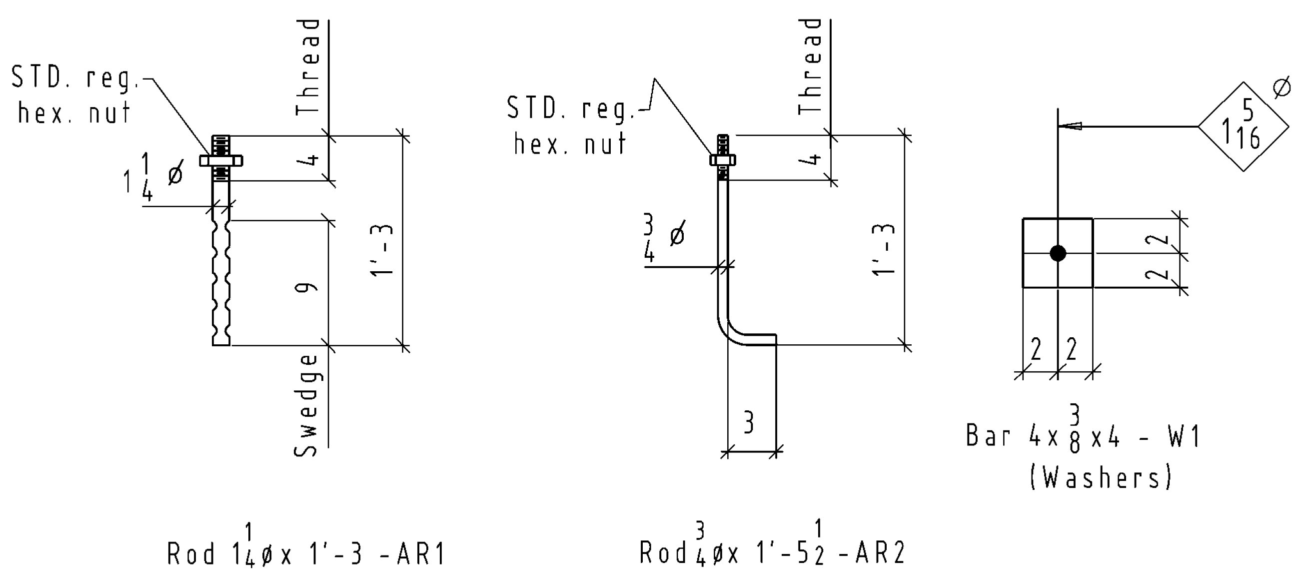


Figure 7-7. Typical anchor rod details.

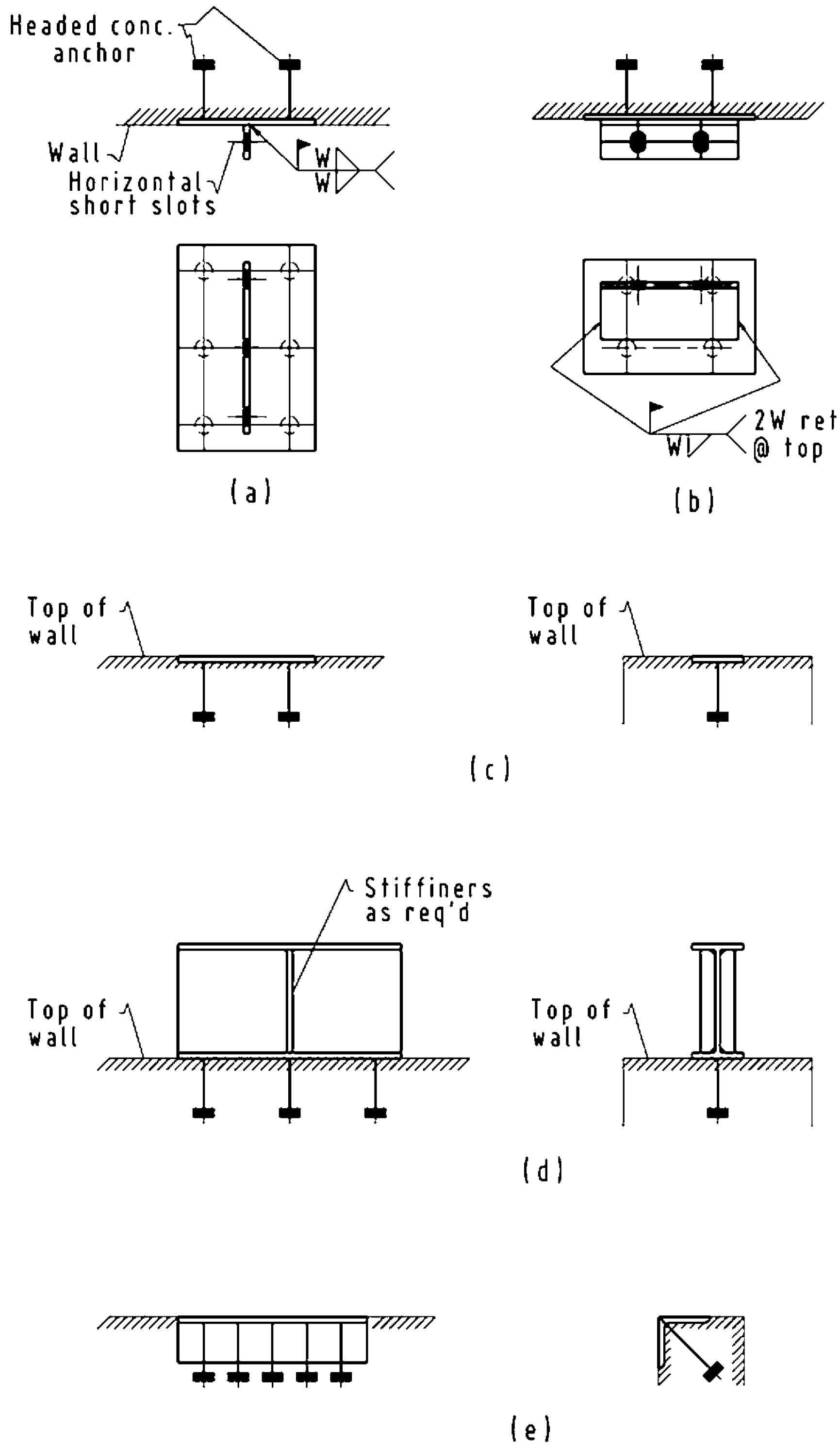


Figure 7-9. Common types of embeds.

Type (d) is used when the supported beam is elevated above the top of the wall. The embed may be a W-, S- or HP-shape. Depending upon the load on the embed and its shape, a pair of web stiffeners may be required. The supported member may be field bolted or welded to the top of the embed. Type (d) embeds are located on the plan by referring its centerlines to the wall centerline and to a working line.

The embed shown as type (e) is used to support several closely spaced beams or steel joists along the edge of the top of a wall. The bottom flanges of the supported beams and the joist bearings are field welded to the embedded angle. On the plan the angle is located so that its legs are flush with the top and side of the wall and one end is dimensioned from a working line.

Although the foregoing discussion has emphasized the use of embeds to support beams, they may be used, also, to support machinery. When used as such, they would be embedded in concrete piers or the floors on which the machinery supports sit.

COLUMNS

Before starting shop drawings of columns and beams, the detailing group, erector and shop management must determine the fabrication method that will be used for both shop and field connections. The erector's preference of the method for connecting is an important consideration as it reflects the erector's ability to apply manpower and equipment to the best advantage. The following methods are available:

1. Shop welded and field bolted
2. Shop and field bolted
3. Shop and field welded
4. Shop bolted and field welded

Systems 1 and 2 are the most common methods.

The shop preference for a particular system varies with the available equipment and shop experience. Fabricating plants that have equipment and layout adapted to punched or drilled work may prefer the use of bolts. Other shops may be better suited for welding and prefer that all shop connections be welded. Many can handle either type of fabrication, but to balance workloads between shop areas prefer to select the connection on a job-to-job basis. The steel detailer is responsible for furnishing shop drawings that will permit the fabricator to produce a job in an economical and efficient manner.

The owner's designated representative for design may list on the contract documents the types of connections that are to be used. However, the owner's designated representative for design may be receptive to a request for a change to alternative connections that might better suit the capabilities of the fabricator's shop. A change in types of connection should be approached with caution. For example, a member sized for gross area in a welded design may not have adequate net area in a bolted design. The effect on erection procedures and field connections should also be considered. It is important to consider the safety requirements set forth by OSHA, as outlined in Chapter 2, when considering any changes.

A parallel but equally important decision is to determine on which members the detail material will be assembled (i.e., the columns or the beams). For instance, columns may have considerable detail materials, like fittings, splice plates, etc., attached to them. If all of this detail material can be assembled to the columns in the shop, the plain beams (no detail material) can bypass the shop assembly area and go directly to the inspection, painting (if required) and shipping areas—an efficient procedure. Special connections and conditions may require some compromise. Any conflicts are resolved as drafting proceeds.

In tier building work, the greater part of the connection material may appear on the columns. Therefore, preliminary planning is helpful even before detailing is started. In the case of large tier buildings, this includes an advance preparation of details covering job standards for wind bracing connections, column splices and other features that repeat throughout the structure. Column and beam gages are selected, and layouts of bracing connections and standards for framed and seated connections are made. Job standards involving connection material are drawn on sheets separate from the shop drawings for reference by the steel detailer and for use in the shop.

One steel detailer working alone seldom produces the required shop details, even for a relatively small project. The tight time schedules of most contracts may require several steel detailers working on a single level of floor framing. Early development of complete column details speeds the work and minimizes discussions that sometimes arise when several steel detailers attempt to work up connections to the same column. A well-developed set of job standards goes even farther in this direction by providing common standards for both column and beam detailers. Often, job standards are kept on file and used on subsequent jobs.

Drawing Arrangement

Shop details of columns may be shown either in a horizontal position with the bottom of the shaft to the left, as shown in Figure A7-10, or in an upright position with the bottom of the shaft at the bottom of the sheet, as shown in Figure A7-11. Furthermore, when a column requires finishing at one end only, that end should be shown at the bottom of the sketch if the column is drawn in an upright position (see Figure A7-12) or at the left end if the column is drawn in the horizontal position. The steel detailer is guided in this choice by the amount of space needed to produce a clear and uncrowded detail. More columns can be shown on a sheet when they are detailed upright, but where complicated fittings, bracing connections and sectional views are required, horizontal placement is advantageous. The space between views must be sufficient to contain all dimensioning as well as all details of gusset plates or brackets that may extend from the column shafts. The column details shown elsewhere in this manual demonstrate the location and spacing of views. Many fabricators maintain files of typical drawings that may be consulted for the approved treatment of specific applications.

In addition to the title block, preprinted drawings generally include shop bills for listing the material required (see Figure A7-11). These usually appear at the right side of the sheet above the title block or across the bottom. Separate shop bill forms, preferred by some fabricators, allow use of the entire sheet area for detail sketches.

General notes applying to the entire sheet are placed in the lower right corner of the sheet, near the title block. Special notes may appear anywhere on the sheet, preferably near the detail concerned.

Column Faces

In tier building work a common practice is to assign a letter to each of the four faces of a column (see Figures 7-13 and 7-14). This identification by letter is helpful to the shop in laying out the work and reduces the probability of shop errors. Looking down on top of the column, the lettering progresses alphabetically in a counterclockwise direction around the shaft as shown in Figure 7-13. In the case of W-shape columns (see Figure 7-13a), faces A and C are always flange faces and faces B and D are always web faces. Seldom is a separate view of face D for W-shape columns shown. Any fittings on face D that differ from those on face B can be shown by dashed lines, indicating a far-side location. The punching or drilling is, of course, common to both faces. The letter D on a combined B and D face view is shown dashed to denote the far side of the web. For a box section if a separate view of face D is required, it follows face C in sequence. Many fabricators do not show the letter D on the column detail except in the case of box columns because the far side of the B face of an H-type column is obviously the D face. If this system of identification is used, material on the web face is noted "N.S." or "F.S.," indicating near side or far side, instead of noting it for face B or D.

The same system is used on box and HSS columns, with the flange-web relationship as shown in Figure 7-13b. The views of faces follow the same alphabetical sequence on the shop drawing, beginning with A and reading from left to right for columns drawn upright and from top to bottom for those drawn horizontally.

The designation of face A as it appears on the framing plan is optional with the steel detailer. As a view of face A is drawn usually to show splice details, either select the flange face that contains the most detail (fittings and fabrication), labeling it A, or arrange the marking sequence to produce a face B web view that will contain the most detail fittings and will be shown as a near side view with full lines.

Faces that contain no detail or fabrication of any kind need not be shown. However, their presence should be indicated by a centerline in the proper relation to the other faces, labeled with a note such as "Face C Plain." In the event that all material and fabrication on face C is identical with A, it may be combined with face A by adding a note such as "Face C same as face A" along the centerline of face C.

Sections

Transverse sections taken through a column shaft are projected always from the web view and are shown looking toward the bottom of the column (see Figures 7-15b and 7-15c). When such sections must be displaced from the B face

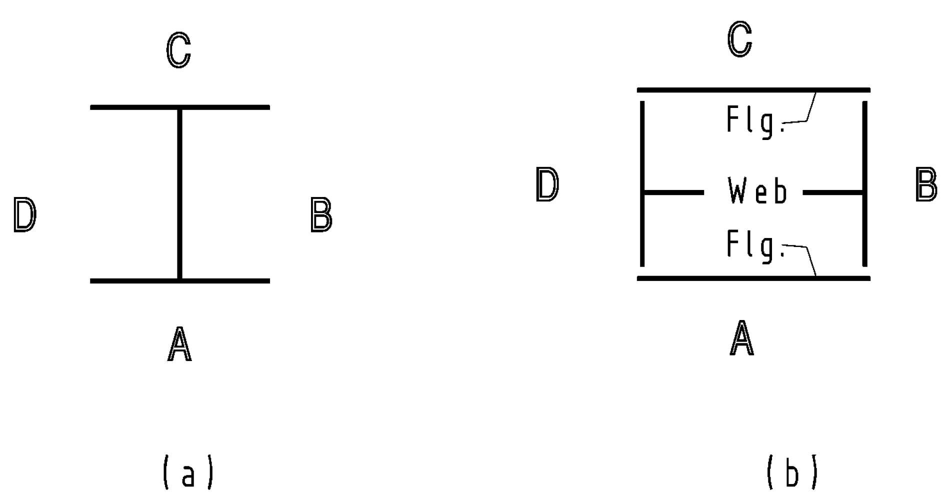


Figure 7-13. Typical letters assigned to each of the four faces of a column.

centerline, as shown in Figure 7-14c, they should retain this same orientation. Unless a box or other section that requires dimensioning is involved, the main material outlines need not be shown in transverse sectional views. When such sections are taken at floor levels, the cutting symbol generally is omitted and the section is labeled from the floor number. Sections showing isolated details or the inner details of a box section are provided with cutting plane symbols and section identification. Where possible, sectional views should be projected directly from the cut sections for viewing in the direction of the cutting plane arrows.

Combined Details

Columns in the same or different tiers that are alike except for minor differences sometimes are shown on the same sketch. While this is an accepted practice, the steel detailer is cautioned against combining details for too many columns on one sketch, regardless of how minor the differences may seem to be. The large number of notes that may be necessary can render the shop drawing difficult to read and cause shop errors. If exceptions cannot be shown without undue complications, the columns should be detailed on separate shop drawings. To reduce the time and effort such repetition requires if preparing shop drawings manually, consideration should be given to making reproductions of the original sheets and altering the copies as necessary to show the differences among the various columns.

Column Marking

Like other framing, columns must be given shipping marks. The mark appearing on the shop drawing is painted on the column shaft for identification in the shop and at the building site. As indicated in Chapter 4, fabricators differ in the marking systems they favor; steel detailers must use the standards adopted by the fabricator.

When the direction a column must face in a structure may not be apparent to the erector, particularly in tier building work, a compass or direction mark should be provided on one of the column faces. To accomplish this, a note such as “Face A East” on the shop drawing following the column mark instructs the shop to paint “East” on the A face (see Figure A7-12). When the axis of a structure varies so widely from a North – South line that some question as to the actual placement arises, the erector will use the plan North direction as shown by the compass arrow on the erection drawing. The steel detailer is cautioned to observe the orientation of each column when applying compass marks. As shown in Figure 7-15, even identical columns will require different compass marks if their webs are not parallel.

Column Details—Bolted Construction

Because much of the work required to fabricate the framing for a tier building is shown on the column details, careful planning is necessary to ensure legibility of the completed shop drawing. An experienced steel detailer will be able to visualize the connections and allow sufficient room for all necessary views and dimensions. Preplanning will result in a clear, uncrowded shop detail drawing, and preparation of a few free-hand sketches will be helpful in consolidating all the information and in visualizing the relationships that exist between the framing on the several column faces at any level. With the complexities thus pictured, space requirements can be foreseen in addition to possible points of interference that must be investigated as detailing proceeds.

Examples of column shop details are given in Figures A7-11, A7-12 and A7-16. These drawings show, respectively, the bottom, middle and top sections of column D4 called for on the partial design drawing shown in Figure 7-1. In the general notes, reference is made to the AISC *Specification* and, for the purpose of illustration, shop connections shall be made

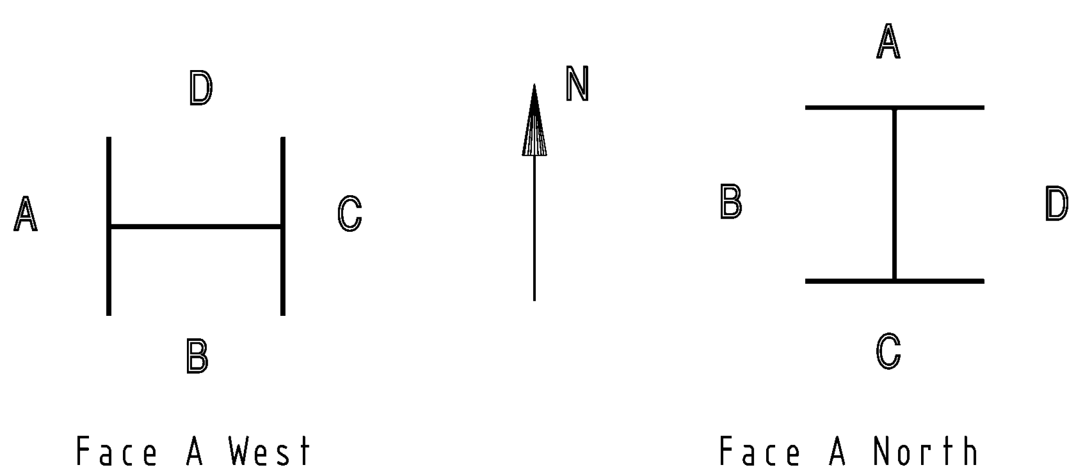


Figure 7-14. Column orientation.

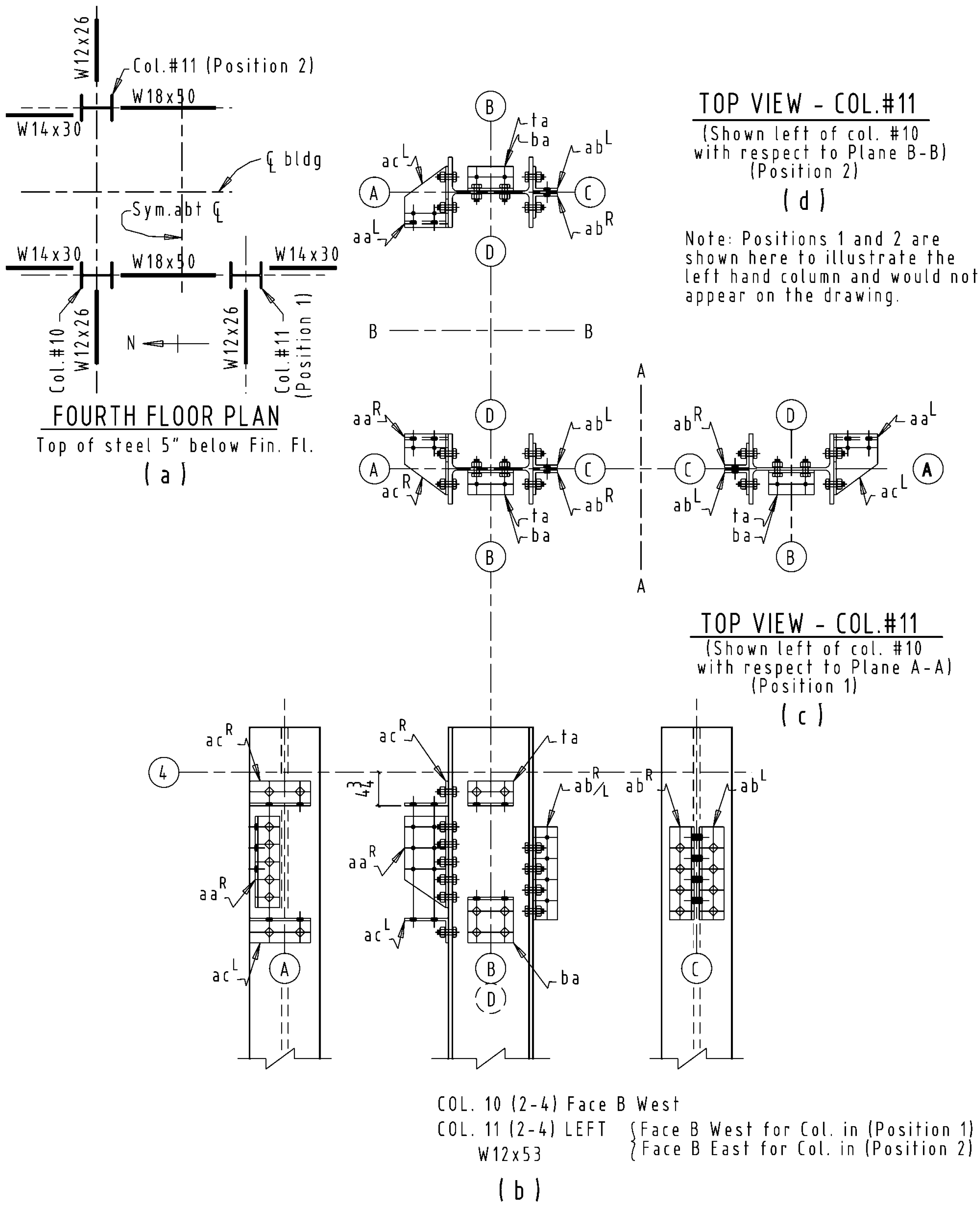


Figure 7-15. Section views of columns.

with high-strength bolts and weld. All field connections shall be made with high-strength bolts. The design also requires that all first and second floor beams be provided with moment connections.

Column D4(0-2) in Figure A7-11 contains these combined shear and moment connections. These details show moment connections in which plates shop welded to the column flanges are field high-strength bolted to the beam web and flanges. To connect the beam to the column web, a stub is fabricated between the column flanges. The stub consists of a pair of stiffener plates between the flanges, tied together by a single plate that projects beyond the toes of the column flanges to connect to the beam web. The stiffener plates are made long enough to provide bolted connections to the beam flanges. Beams are swung horizontally into these connections on the column flanges and web. These types of moment connections are illustrated in the *Manual* Part 12. The steel detailer should note that the distance between lines on which bolts and open holes are placed is called a gage; the lines are known as gage lines.

Base angles and splice plates are in accordance with design requirements. The $1^1/_8$-in. holes in the splice plates are for the use of the erector in handling the column. Although it is doubtful if this column, weighing less than 2 tons, would require these holes, they are shown here to illustrate detailing practice. Heavy columns, which must be plumb as they are swung into place, are generally provided with some such means of attaching a lifting lug (see Chapter 6).

Column D4(4-R) in Figure A7-16 has a shear connection that was selected from the *Manual*, Part 10. A special connection at the fourth floor on face A of column D4(2-4) consists of two $4\times4\times{}^1/_2$ angles designed to support 10-in. channels framed to either side (see Fourth Fl. & Low-Roof Plan, Figure 7-1). The legs extend beyond the column flange to connect the channel webs.

In the connections on columns D4(0-2) and D4(2-4) in Figures A7-11 and A7-12, respectively, erection clearances are provided only at the top of the beam, similar to that shown in Figure 7-17c. The $^1/_4$-in. to $^3/_8$-in. clearance shown in Figure 7-17c is suggested when the top and bottom connections are either shop bolted or shop welded to column flanges. Figures 7-17a, b and d illustrate other methods of handling erection clearances.

The two W12×26 beams shown 4 in. off center on the High Roof Plan (Figure 7-1b) will frame most easily inside the column flange. Fills attached by welds to the A face of column D4(4-R), fill the space between the beam web and the inner face of the column flange.

The dimensioning shown in Figures A7-11, A7-12 and A7-16 represents fairly universal practice in column work. Overall dimensions and dimensions locating floor levels are placed prominently, farther away from the views. Detail dimensions showing spacing of punching, gages, etc., are placed closest to the sketch. Checker's or shop inspector's figures, such as beam depths and extension dimensions, are placed in between or wherever they best will serve their purpose.

Extension dimensions, used by the shop to establish and check the location of open holes or the tops of seats, are measured to the finished bottom of the column shaft. This is true even when such dimension lines are not shown full length. In the absence of seated details, extension dimensions are given either to the top hole or to the bottom hole of any group of holes to which framing attaches, depending on the fabricator's standards.

All dimensions relating to a connection are tied to the top of steel at which the connected beam appears on the plan. Note that dimensions that space punching or drilling in the column shaft are continuous within each group and are not interrupted by the insertion of edge distances or gages.

Fittings that have been detailed once need not be dimensioned completely where they appear again on the same sheet. In Figure A7-12, note that flange face gages on the beam connection angles appear only once for each different fitting and that both the gages and hole spacing for repeating angles are omitted in other views. However, the spacing of punching through the column shaft is always given, even though the fittings connected may be duplicates of one previously dimensioned.

Edge or end distances of fittings are not shown unless the punching on the piece is not symmetrical. Otherwise, equal ends or edges resulting from the billed size of the fitting will be provided. As a case in point plates (pb) in Figure A7-11 will be punched with equal end distances of:

$$\frac{15\text{ in.} - 12\text{ in.}}{2} = 1\tfrac{1}{2}\text{ in.}$$

Dimensions used to instruct the shop on required field clearances are shown between the outstanding legs of beam web connections, between splice plates at the tops of columns and out-to-out of fillers and butt plates at the bottoms of columns when they must enter between splice plates. The presence of these dimensions alerts the shop to clearance requirements that must be met. Critical or "tight" clearance dimensions sometimes are noted "Not more" or "Not less" where nominal allowances are permitted in one direction only. A more positive method is to note a tolerance dimension that stipulates upper and lower limits, such as "1'-$6^1/_4 \pm {}^1/_{16}$." In this case any measured distance between 1'-$6^3/_{16}$ and 1'-$6^5/_{16}$ will be acceptable.

Dimensions should be given for the opening between seats and top angles with tolerances shown, if required. Where beam web connections and seats are used in combination, tie the bottom hole of the web detail to the top of the seat.

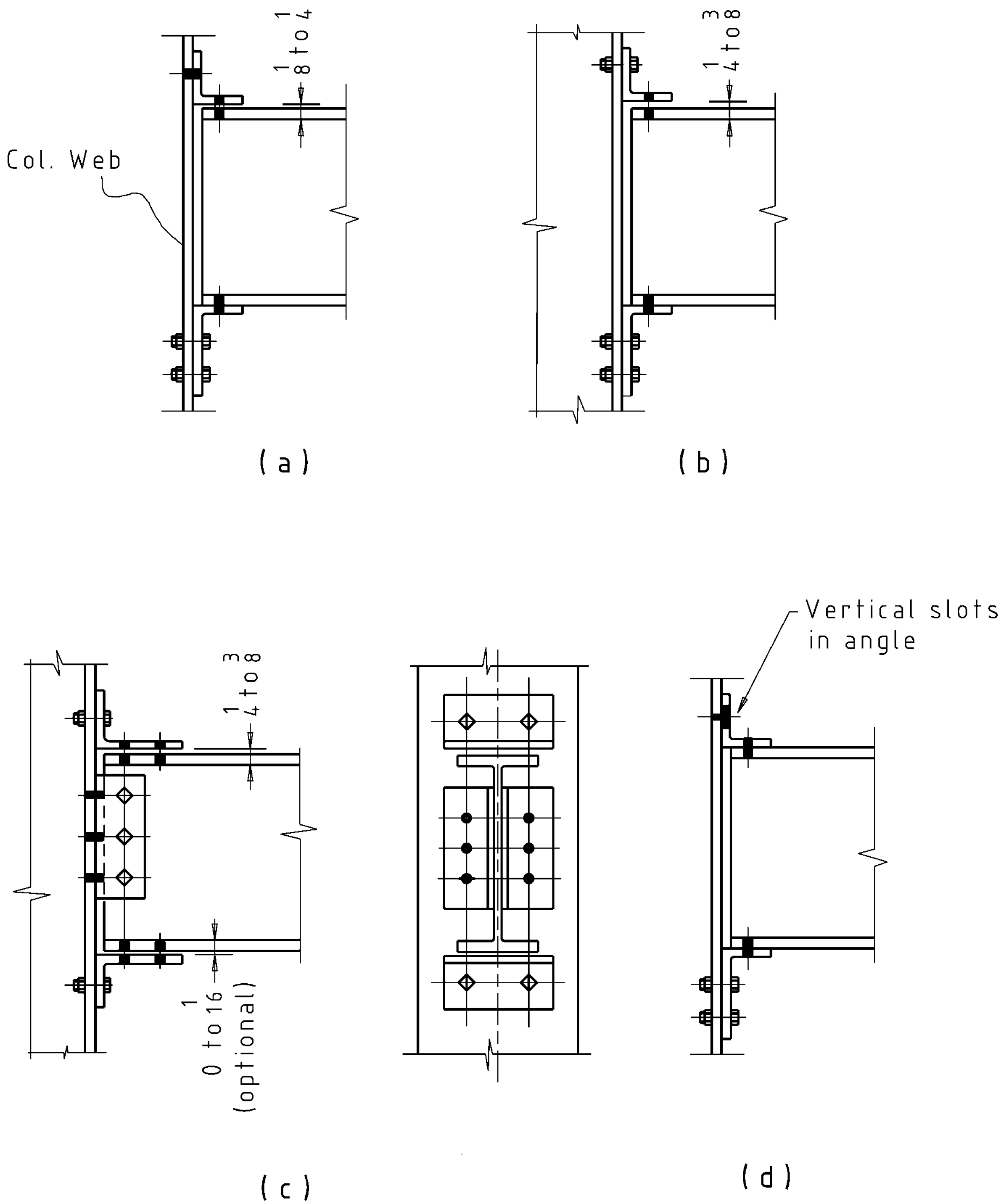


Figure 7-17. Methods of handling erection clearances.

Customarily, the dimensions for the lines of holes (flange gage) are given at the bottom of the column on face A. Similarly, at the bottom of the column on face B, dimensions are given for the flange thickness and centers of web holes (web gage).

Below the detail sketch of a rolled column, place an identification block to contain the following information, provided it is consistent with the fabricator's practice (see Figures A7-11, A7-12 and A7-16):

- The basic column mark (including tier designation) with "Left" designation, if required
- Section description
- Published depth, flange width and web thickness
- Direction mark
- Lifting weight, if required

Figures A7-11, A7-12 and A7-16 illustrate one method of billing material. It utilizes a preprinted shop bill form as part of the shop drawing. Detail material is indicated on the shop drawing by assembly marks and is billed completely in summary form in the shop bill. A second method uses billing directly on the shop drawing. This latter method requires a separate shop bill, similar to the form printed on Figure A7-11, on which again the material is billed in summary form.

The billing of shape descriptions follows the system recommended in Figure 1-1 in Chapter 1. Notes for finish requirements follow the billing in the shop bill. Allowances for finish on detail fittings are covered in the ordering procedure and seldom are shown on either the shop drawing or bill.

These three column detail sheets also illustrate two systems of assembly marks. Figures A7-12 and A7-16 use an alphabetic system in which each letter identifies a different fitting. Identical fittings on the same sheet carry the same letter. In the two-letter system used in Figure A7-16 the prefix letter describes a particular type of fitting. Thus, a = angle, p = plate, etc. The second letter denotes in alphabetical sequence the first, second, third and subsequent appearances of this type of fitting. In the two-letter system, different angles on the same sheet would be designated as aa, ab and ac and different plates as pa, pb and pc. In either system, certain letters, such as i, l, o and others, are omitted as being too easily confused with numerals. Other methods in use are numeric and alphanumeric. The full development of these alternate methods will not be treated here, as the steel detailer must use the system adopted by the fabricator.

Although the general and special notes appearing on Figures A7-11, A7-12 and A7-16 are relatively simple, some explanation is in order. The note "CUT SQUARE" at the bottom of columns D4(0-2) and D4(4-R) directs the shop to finish the column prior to the assembly of detail material. The letters B and D opposite certain dimensions and detail fittings designate the web faces to which the dimensions refer or upon which the fittings assemble. The drawings should give the detailing dimension, as shown in the lower left-hand corner of Figure A7-11. This includes the total depth of the section, web thickness, and web half thickness, with the flange width and flange thickness below. This information makes referring to the dimension tables unnecessary when any of these principal figures must be known during checking, determining fastener grips and providing clearances. The instructions in the general notes for painting were taken from the job specifications.[1] The Society for Protective Coatings (SSPC) *Guide for Selecting One-Coat Shop Painting Systems* (SSPC-PS Guide 7.00) (SSPC, 1982) will be applied in accordance with AISC *Specification* Section M3.1. The notes require (1) complete fitting and shop bolting before any painting is done, in accordance with AISC *Specification* Sect. M3.3, and (2) omission of paint on faying surfaces of connections assembled by high-strength field bolts to provide the slip-critical connections called for on the design drawing (Figure 7-1).

Column Details—Welded Construction

Figures A7-18, A7-19 and A7-20 are details of tier building column D4 utilizing welded construction to support the framing shown on the plans in Figure 7-1. In the arrangements of views, the steel detailer can see methods of dimensioning, billing and notes that these details follow closely the bolted types shown in Figures A7-11, A7-12 and A7-16. The principal difference lies in the presentation of connection details.

Although this section discusses welded details of W-shape columns, a similar discussion applies to box and HSS columns. One significant difference concerns box and HSS columns with shop-welded base plates and open tops. If these columns are exposed to rain or snow, water may accumulate in the bottom of the column to the extent that, if it freezes, the column bursts. To avoid this situation, the steel detailer should provide a drain hole at the base of the column shaft.

Figures A7-18 and A7-19 are shop detail drawings of columns D4(0-2) and D4(2-4), respectively, with all fittings shop welded. These details are commonly used moment connections in which the web of the beam is field bolted with high-strength bolts to a single plate on the column and the flanges are field welded to the column. The *Manual*, Part 12 provides an illustration of this type of moment connection. A variation of this type of connection is one in which the

[1] The steel detailer should note that, when permitted in the contract documents, AISC *Specification* Section M3.1 provides for omission of all shop paint on certain types of work. Although the columns detailed here might have been in this class of work, shop painting was specified to illustrate the noting incident to high-strength bolting and field welding.

single plate is field welded to the beam web. To hold the beam in position while welding, at least two bolts are used to connect the beam web tightly to the plate.

To facilitate erecting the beam and field welding its flanges to the column web, a stub is fabricated between the column flanges. The stub consists of a pair of stiffener plates between the flanges and tied together by a single plate. The stiffeners are extended approximately 1 in. beyond the toe of the flanges to eliminate the effects of triaxial stresses. Often the lower stiffener is made thicker than the upper stiffener to compensate for beam overrun and underrun. The single plate is extended past the toe of the flanges to connect to the beam web, which is swung horizontally into position. The *Manual*, Part 12 illustrates this type of moment connection.

One feature of the beam-to-column connections D4(0-2) and D4(2-4) requires special treatment on the erection drawing. In order to ensure placement of beams on the correct side of the welded single-plate connections, the erection drawing will show the column as illustrated in Figure 7-21.

The base plate on column D4(0-2), Figure A7-18, is welded in such a manner that all welds can be made in the horizontal position without turning the column. These welds must be strong enough to withstand toppling loads during the erection phase and to comply with OSHA requirements.

Connections to column D4(4-R), Figure A7-20, include the use of an unstiffened seat on a flange face and a plate connection for off-center beams. Design of the seat is taken from Table 10-6 in the *Manual*, Part 10. Although a 6-in.-long angle would have satisfied design strength requirements, it would have required blocking the bottom flange of the W12×35 to permit horizontal welding to the seat. A 6×4×3/4 seat with a length of 8 in. was selected to eliminate this blocking. However, the seat is detailed with a 9-in. length, as billed, in order to permit placement of the vertical welds along the edges of the column flange.

The high-strength bolts attaching the two W12×26 beams to the 5/16-in.-thick plates are assumed to take vertical shear only. Welds attaching the plates to the column are proportioned for vertical shear and moment using Table 8-8 (angle = 0°), in the *Manual*, Part 8. Erection data showing the field welding at the top of column D4(4-R) is shown in Figure 7-22.

Unstiffened Seat Details—Bolted

Figure 7-23b illustrates details of the connection of a W18×71 beam to a W8×35 column. Several points that are characteristic of seated connections when used to connect beams to column webs are listed:

- The published depth of the beam, rather than its nominal depth, is used in determining the vertical distance between the seat angle and the top angle.
- The top angle is marked "Bolt to Ship," meaning that it is to be bolted so that it will not be misplaced during shipment. Common practice on shop detail drawings

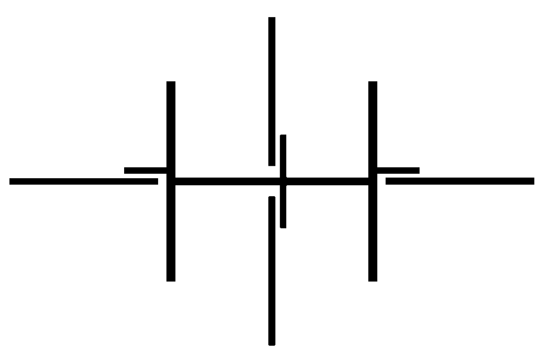


Figure 7-21. Column erection drawing to ensure placement of beams on the correct side of the welded single-plate connections.

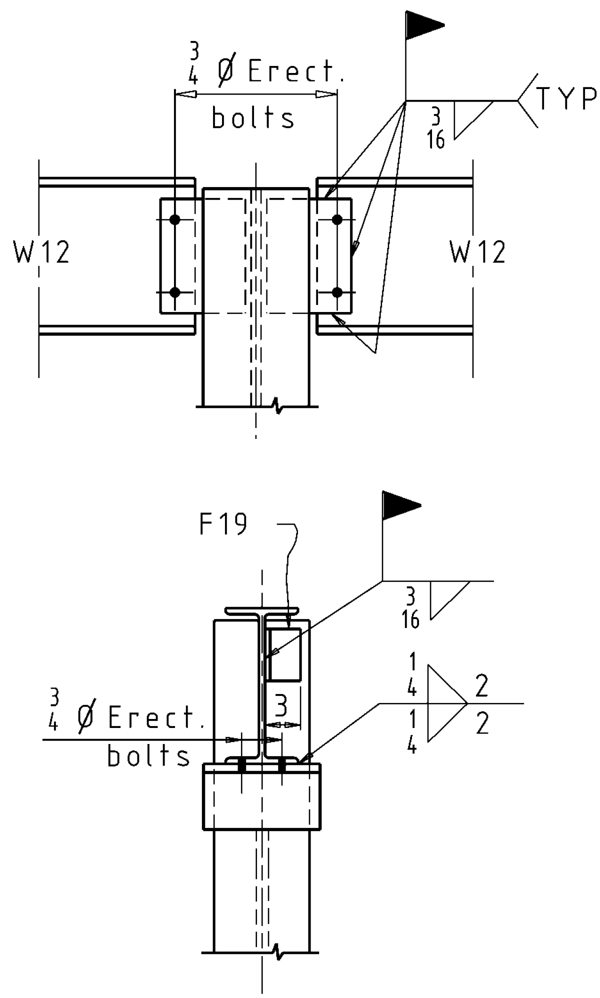


Figure 7-22. Part plan taken from a design plan and connection details.

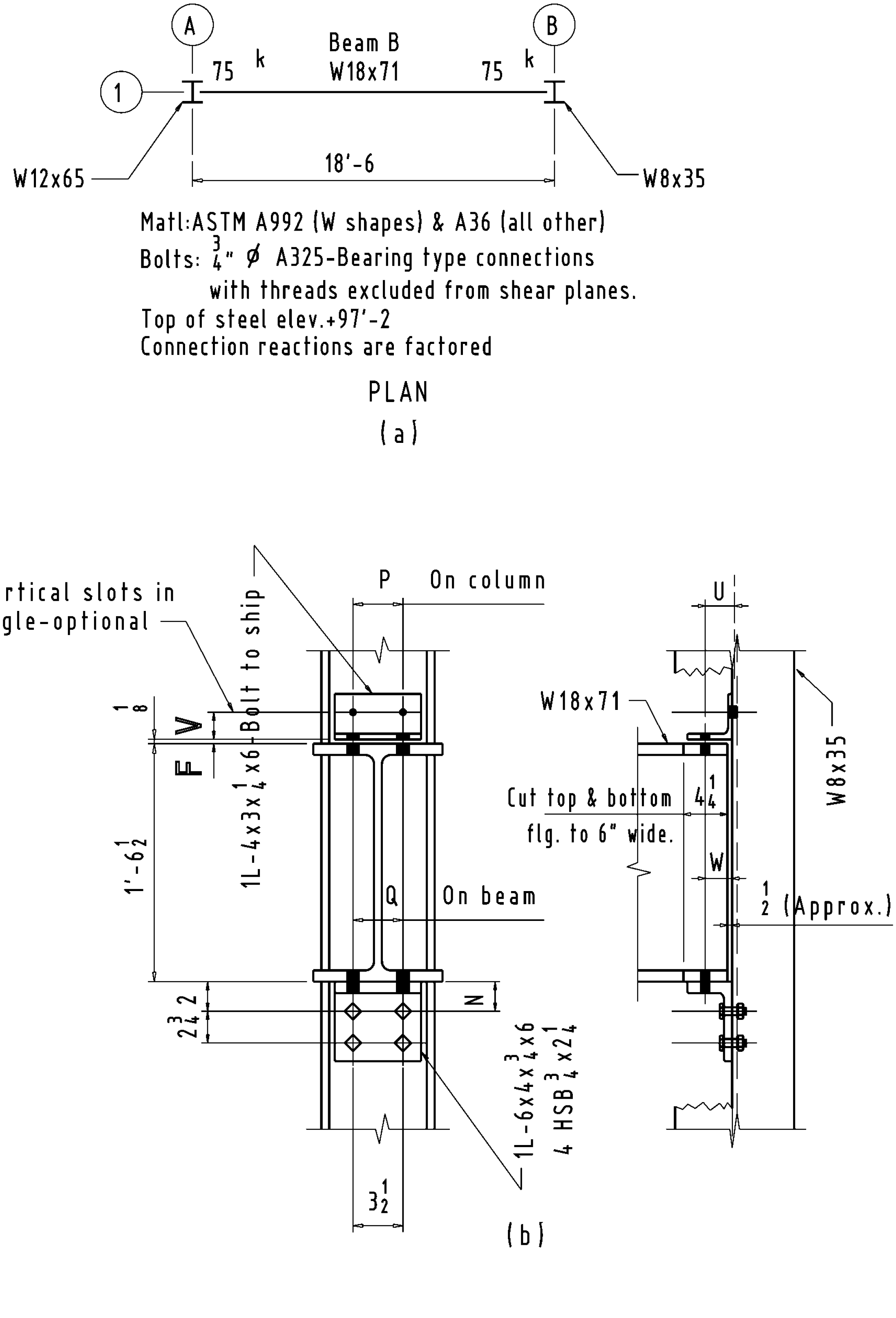


Figure 7-23. Part plan taken from a design plan and connection details.

is to show open holes, even though the shipping piece will have bolts in these holes. Before the beam can be lowered onto the seat in the field, the top angle will be removed temporarily. However, on column flanges the top angle usually can be shop bolted to the column, as the ends of the supported beams can be entered horizontally between the top and seat angles.

- The top angle is located to provide an adjustment distance, F, between the angle and the top of the beam to compensate for mill tolerance variations in the published depth of the supported beam. Some fabricators detail it as 8 in., requiring no fills. Other fabricators provide a larger dimension, requiring fills to suit the gap, while still others would prefer to use vertical slots in the top angle and avoid fills altogether (see Figure 7-17). Another option worth considering may be to ship plain top angles for field welding.
- Gage N is kept as small as possible to reduce the deflection of the seat angle under the beam load.

Gages P and V are the usual gages except as noted here:

- When A325 bolts are required, the usual field procedure is to place all bolts in a joint before tightening each bolt.[2] Sufficient clearance must be provided for the impact wrench. Gages W and V must be arranged to provide this clearance. When a beam frames to only one side of a column web, tightening of the field bolts on the V gage line can be performed on the opposite side of the column web. However, when beams frame to both sides, gage V must be sufficient to provide clearance over the bolts through the beam flange. Note that for connections which are suitable for tightening to the "snug tight" condition, tightening may be accomplished either by the use of an impact wrench or by the full effort of an individual using an ordinary spud wrench.
- Gages P and Q are the same ($3^1/_2$-in.) The distance between column flanges limits the length of the top and seat angles and, thus, determines both column and beam gages.
- Gage W is the same for both the top and seat angles. It may be the usual gage for the given size of angle leg or may be adjusted for one-half the thickness of the column web to afford a convenient figure for distance U. (For example, 3 in. $-$ $^3/_{16}$ in. $=$ $2^{13}/_{16}$ in., as shown in Figure 7-24 for this case.) This latter practice permits the duplication of beam details when otherwise identical beams are supported between columns having varying web thickness.
- The ends of the punched beams are kept approximately $^1/_2$ in. back from the face of the column web, except where it is necessary to increase this dimension for erection clearance.
- The top and bottom flanges of the beam are cut at the ends, if necessary, to provide at least $^1/_2$-in. erection clearance between the column flanges.
- Referring to Figure 7-24, the steel detailer will note that cutting the flanges on the right end of the beam to 6 in. wide results in $^9/_{16}$-in. clearance between edge of beam flange and inside of column flange. This is adequate if no bolt heads or nut and stick-throughs are inside of the column flange above the beam connection. In all cases whether to clear column flanges or bolts, where members must be erected by dropping, allow clearance of between $^1/_2$ in. and $^3/_4$ in.

Stiffened Seat Details

For an example of a detailed stiffened seat, see Figure 7-25. To tighten the $^3/_4$-in.-diameter A325 field bolts with an impact wrench, the holes in the outstanding leg of top angle a and seat plate b and, also, the holes in the vertical leg of the top angle should be located to provide a minimum $1^1/_4$-in. clearance for the wrench (see Tables 7-16 and 7-17, *Manual* Part 7). A gage of 3 in. in the seats and 3 in. in the vertical leg of the top angle furnish the required clearance.

The gage is calculated by adding dimensions H2 and C1 to the angle thickness. (Clearances C1, C2, H1 and H2 are given in Tables 7-16 and 7-17.)

Vertical and outstanding legs:

$$1^3/_8 + 1^1/_4 + {}^5/_{16} = 2^{15}/_{16}\text{ in.} \rightarrow \text{use 3 in.}$$

The preferable shop practice is to maintain the same gage in the top and bottom beam flanges. This keeps the spacing of the open holes the same in the top angle and seat plate. The practice of maintaining the same spacing for open holes in both pieces is so general that unless specifically shown otherwise, the shop will take for granted that the dimensions shown in the top sectional view (Figure 7-25) apply to both the top angle and the seat plate.

To ensure good contact of the stiffeners where they bear against the underside of the seat plate, the top ends are fitted carefully in the shop by grinding, milling or saw cutting. The notation F1E instructs the shop to "fit one end" of the stiffener angles to produce a full bearing surface (see Figure 7-25). Some shops require the note BE (bearing end) to be shown on the shop drawing at all points where fitted stiffeners must take loads in bearing.

The fitting material for the connection on the web view of the column (Figure 7-25) is billed to show that two connections

[2] Although this method is preferred, limited clearance in some "tight" connections may require final tightening of certain bolts before entry of others. In such an event the sequence of bolt assembly and tightening must be covered on the erection drawing. (See "Entering and Tightening Clearances," the Manual, Part 7).

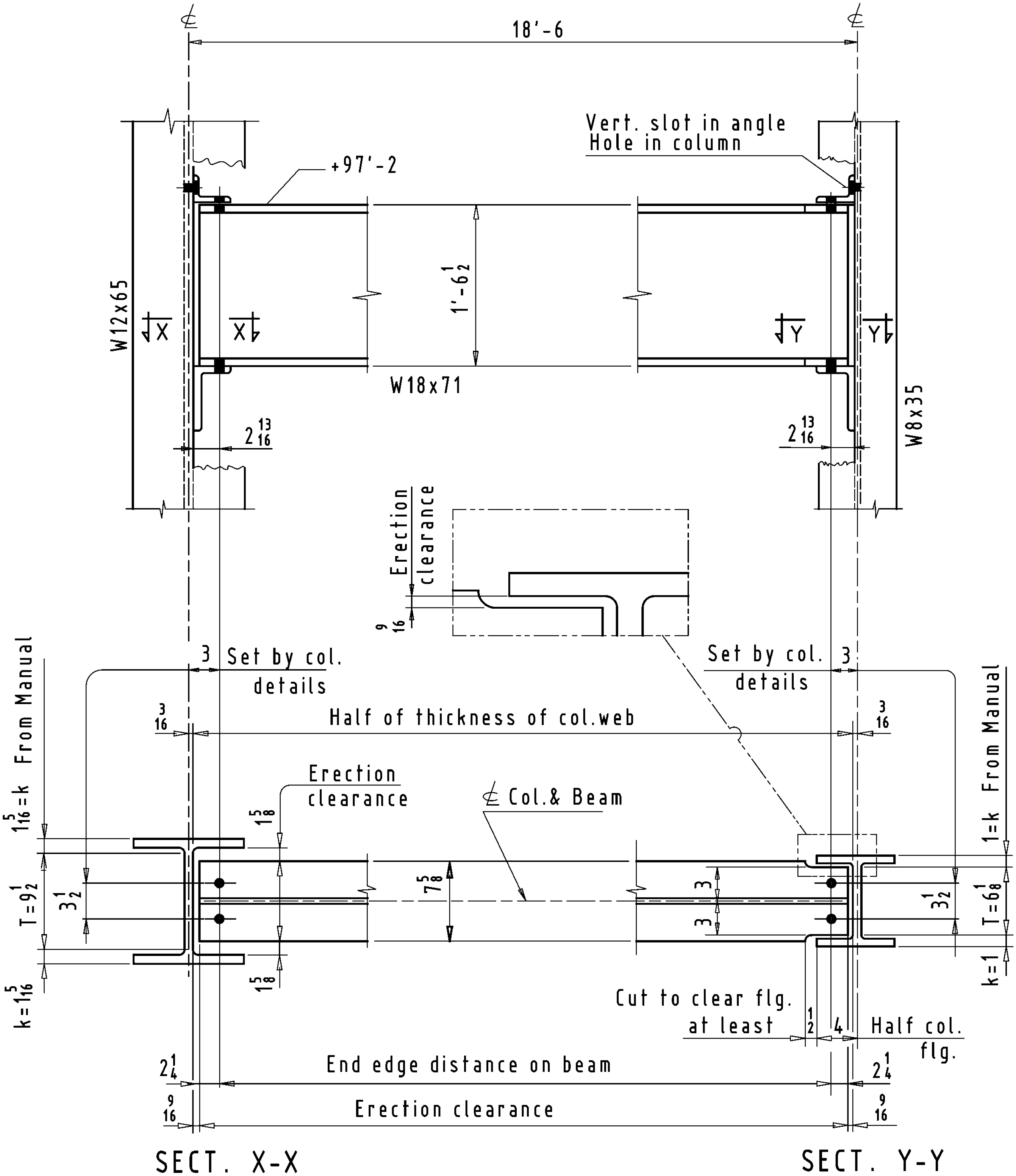


Figure 7-24. Erection clearances.

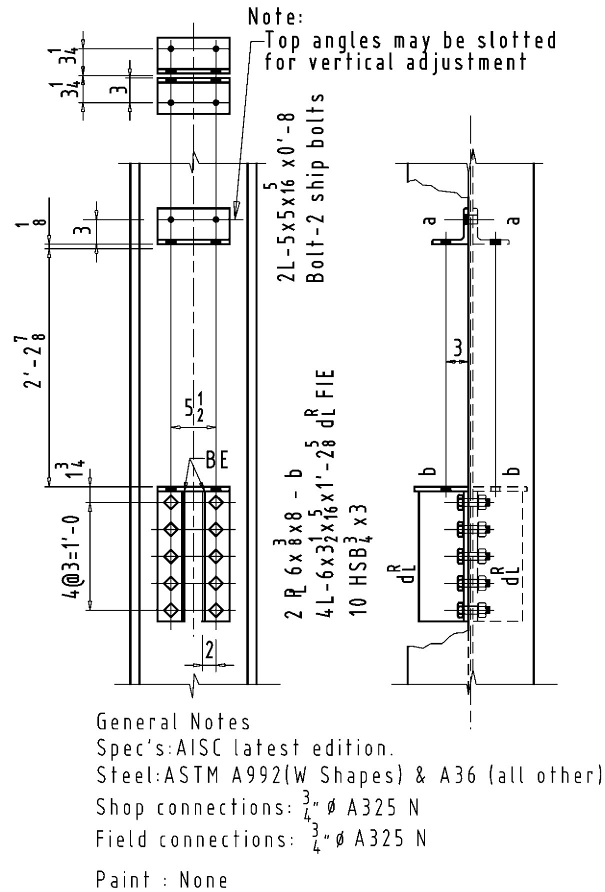


Figure 7-25. Detailed stiffened seat.

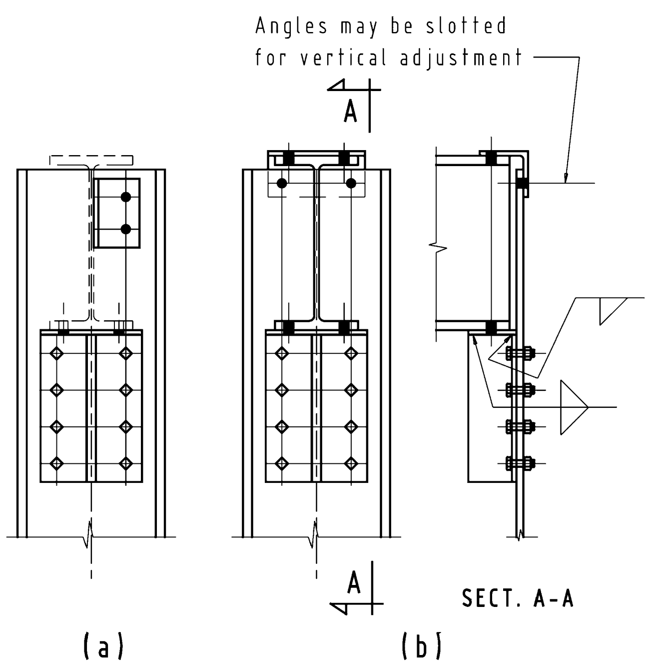


Figure 7-26. Methods of providing lateral support.

(one on each side of the column web) are required. Such billing and the use of sectional views usually eliminate the need to show the side view of a column web connection. The cutaway flange view in Figure 7-25 is shown only for illustrative purposes.

Figure 7-26a shows a method of providing lateral support at the top of a column where no room is available to place the top angle in its usual position. The arrangement shown in Figure 7-26b can be used when no beam is framing to the opposite side of the column web.

BEAMS AND GIRDERS

As a rule, each beam in a system of floor or roof framing makes a convenient shipping and erection unit. The shop drawing seldom pictures any of the adjacent members to which the beam later will be connected in the field. However, in preparing a detail drawing, all features that have a bearing on the erection of the beam must be investigated.

The locations of open holes in the beam connections must match the location of similar holes in the supporting members. Proper erection clearances must be provided and possible interference must be eliminated, so that the beam can be swung or lowered into position for connecting to its supporting members.

Detailing practices call for the most economical use of material, with due regard to reducing shop fabrication costs. Methods for the calculation of structural steel weights are published in the AISC *Code of Standard Practice*. The steel detailer is responsible for determining multiples of non-rectangular shapes, when they can be cut economically from either a larger or longer piece to effect an overall saving.

Suppose that three identical, skewed 12 in. channels (C12×20.7) each require a piece of material 17 ft 2 in. long.

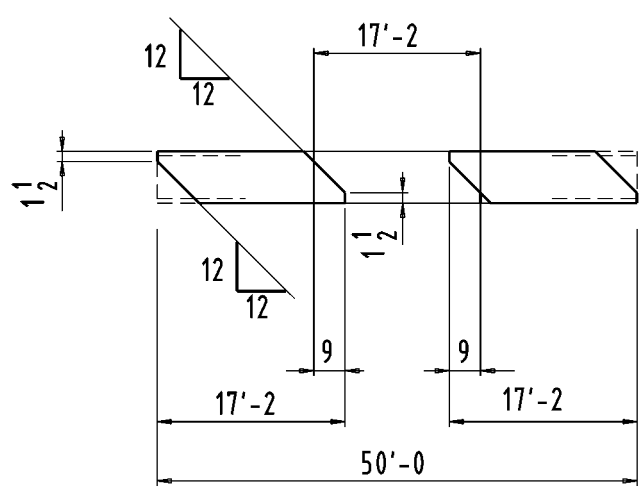


Figure. 7-27. Nested or multipled pieces.

Because the skew cuts at the ends must be made in the shop, a single piece sufficient in length to make the three pieces is ordered from the mill. Figure 7-27 indicates how these pieces can be nested or multipled to effect a saving of 1 ft-6 in. in the overall ordered length. Instead of a piece 51 ft-6 in. long [3×(17 ft-2 in.)], one piece 50 ft-0 in. will suffice.

Care must be taken in dealing with unsymmetrical shapes, such as channels and unequal leg angles, to be certain that the appropriate length is ordered based on the nested or multipled pieces. For example, if one of the three skewed channels just discussed must have its sloping cuts made opposite to the cuts shown in Figure 7-27, a total length of 50 ft-9 in. instead of 50 ft-0 in. is required.

Connection Angle Details

Figure 7-29 is a shop detail for the W18×60 beam in the Part Plan shown in Figure 7-28. At the left end of beam B1 are shown the details of the connection angles selected. The shop details of these angles follow long-established practices among most fabricators. Following are pertinent facts on detailing these connection angles:

- For every shop bolt through the web, shown as hexagons, two open holes for field bolts are required (shown by blackening the angle thickness of the bolt pitch).
- The web leg is 3 1/2 in. and its gage is 2 1/4 in. The distance from gage lines to backs of angle channels are called gages.
- The minimum edge distance from the center of a standard hole to any rolled or cut edge is given in Table J3.4 of the AISC *Specification*. With a 2 1/4-in. gage, the edge distance to the rolled edge of the connection angles is 1 1/4 in., well within the AISC *Specification* requirements. This gage also provides sufficient clearance to install and tighten the shop bolts (see "Entering and Tightening Clearances" in Tables 7-16 and 7-17 in the *Manual*, Part 7). The incremental amounts to be added to accommodate oversize and slotted holes are shown in Table J3.5 of the AISC *Specification*.

The open holes for connection to the webs of the supporting W24 and W21 are shown spaced 5 1/2 in. apart center-to-center by the note following the angle size (GOL = 2 1/2; 5 1/2 c-c). This is a very common dimension and permits the use of nominal leg size connection angles. This dimension is

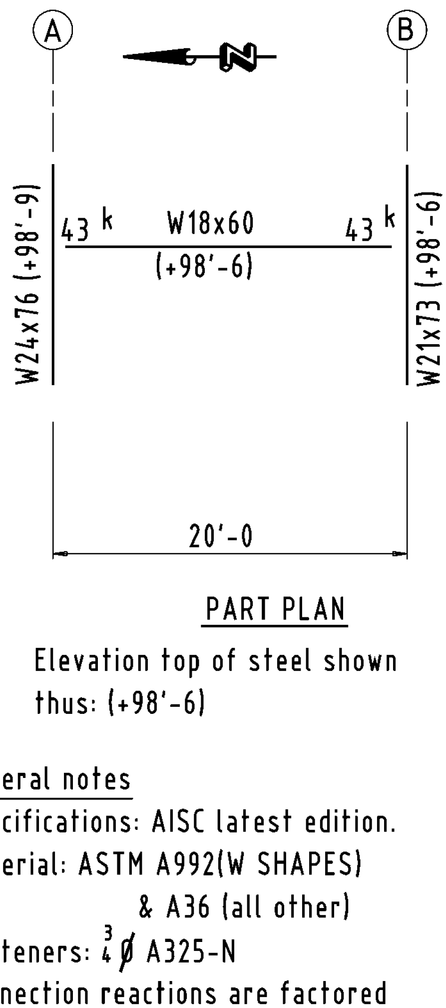


Figure 7-28. Part plan.

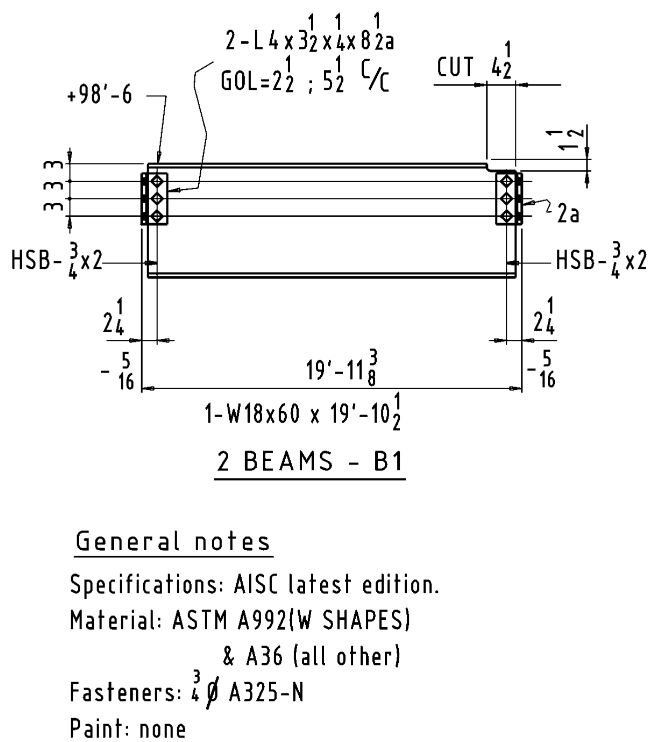


Figure 7-29. Shop detail for the W18×60 beam in the part plan shown in Figure 7-28.

possible here only because no beam frames opposite to create interference in inserting and tightening the field bolts. Alternatives that are available to create sufficient tightening clearance are:

- Stagger the shop and field bolts as shown in Figure 7-30. In this case a true stagger provides ample clearance. The *Manual* Tables 7-16 and 7-17 are convenient for use in checking both the entering and tightening clearances, as well as the stagger clearances, for impact wrench tightening.
- Increase the connection angle gages to provide adequate driving clearance, as illustrated in Figure 7-31. Again, the clearances in Tables 7-3a and 7-3b of the *Manual* are convenient to use, where dimension H2 gives the allowance to cover the washer, nut and bolt projection. Table 7-1 in the *Manual* lists dimensions of high-strength bolts, nuts and washers.
- As described in Chapter 6, to facilitate erection and for reasons of safety, the connection angles should be staggered as illustrated in Figures 7-30 and 7-31.

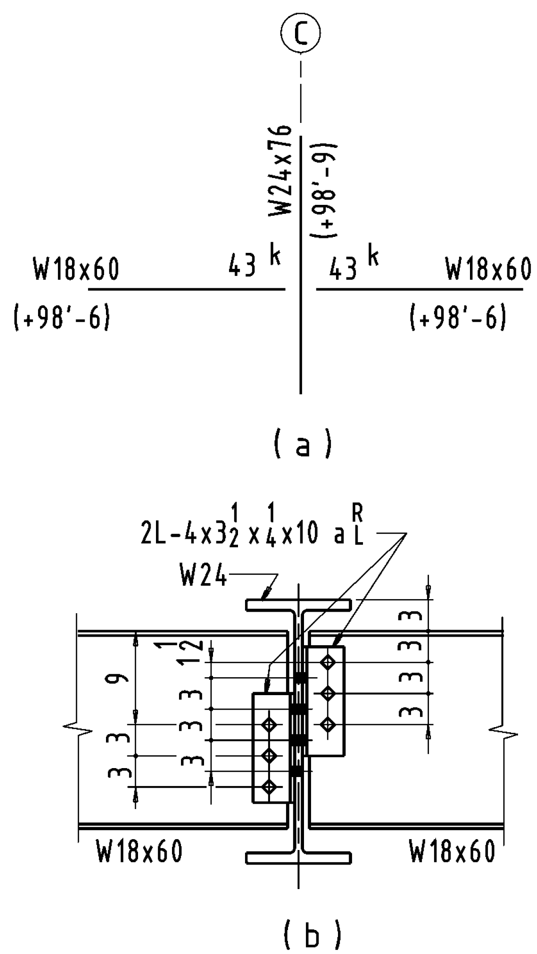


Figure 7-30. Staggered shop and field bolts.

The outstanding leg of each connection angle is 4 in. and its gage is one-half of the center-to-center of holes minus $t_w/2$ (from the *Manual*, Part 1). Thus, the gage on the outstanding leg (GOL) equals $(^1/_2 \times 5^1/_2) - {}^1/_4 = 2^1/_2$.

The edge distance, center of fastener holes to the ends of the angles, usually is $1^1/_4$ in. for connection angles (see footnote d at the bottom of Table J3.4 of the AISC *Specification*). The length of the connection angles is the sum of the fastener spaces plus the two edge distances. In the case of the angles a in Figure 7-29, the length equals $1^1/_4 + 3 + 3 + 1^1/_4 = 8^1/_2$ in. Note that the edge distance on connection angles usually is not dimensioned unless the two edge distances are not equal. Note, too, that $1^1/_4$-in. edge dimension can limit the connection design strength because of bearing.

The connection angles are given assembly marks, usually a small letter. In the notation 2a to the right of beam B1 in Figure 7-29, the number 2 indicates two angles and the letter a is the assembly mark. This system of marking is used because:

- It saves billing these angles again if they are reused on the same piece (as at the right-hand end of the beam on Figure 7-29 or on other beams on the same drawing).

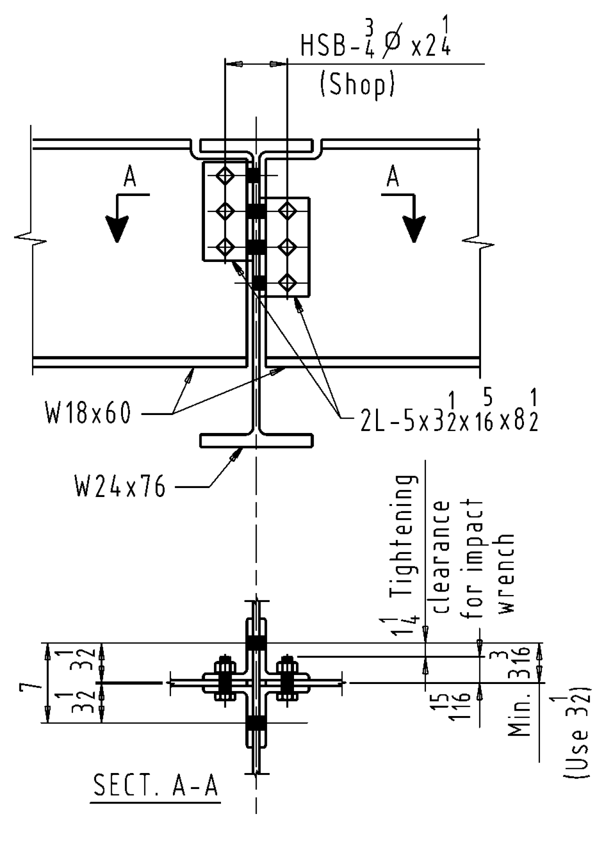


Figure 7-31. Connection angle gages to provide adequate driving clearance.

- The angles will be punched on a different machine from that used for the beam. The assembly mark ensures the correct angles will be assembled on the correct beam.
- When the gage called for on the larger of two unequal legs of a connection angle is such that it could be used only on that leg and, yet, leave the minimum allowable edge distance, giving a dimension of the leg on the detail drawing is unnecessary.
- The small amount of eccentricity (in this case $2^1/_4$ in.) from the backs of the web connection angles to the gage lines for the fasteners does not affect appreciably the design strength of the connection and is, therefore, not considered.

Throughout the discussion and illustrations of framed beam connection angles, standard holes have been shown for field connections. Where permitted by the designer, horizontal slots (usually short slots) in angles may be preferred by erectors and fabricators for field connections. Slots accommodate the minor variations often experienced in punching and drilling holes in beams and columns, and provide some latitude for adjustment in plumbing and aligning a frame during erection.

Although a pair of angles $4\times3^1/_2\times^1/_4$ connected to the beam web with two $^3/_4$ in.-diameter A325-N bolts provides an available strength using LRFD that is greater than 43 kips, as shown in Table 10-1 of the *Manual*, their use is limited to beams having a maximum nominal depth of 12 in. Thus, a three-bolt connection is required.

Beam Gages

In the interest of standardization and reduction of possible errors in matching connections, common practice is to place holes for connections on horizontal beam gage lines (rows) spaced 3 in. apart vertically, with the uppermost gage line set 3 in. below the top of the beam, when practicable, as shown in Figure 7-29.

With a 3-in. dimension from the top of the beam to the first hole, the matching connection can usually be detailed with a $1^1/_4$-in. edge distance without encroaching on the fillet k-distance of the supporting beam/girder to which it is bolted. If this is not possible the 3-in. distance to the first hole must be increased to maintain the $1^1/_4$-in. edge distance. However, minor encroachment on the fillet is permissible as shown in Figure 10-3 in the *Manual*, Part 10.

In the *Manual*, Part 1 the depth of a W18×60 is listed as $18^1/_4$ in. When this beam arrives from the mill, its actual depth at the centerline of the web may vary from $18^1/_8$ in. to $18^3/_8$ in. Hence, if the 3-in. dimension from the top of the beam to the first gage line is to be maintained, a theoretical dimension given from the bottom of the beam to the nearest horizontal gage line necessarily would not be correct. For this reason, fabricators dimension gage lines from either the top or bottom of a beam, but not from the top and bottom.

Cutting for Clearance

Figure 7-28 indicates that the W18×60 and the W21×73 are "flush top," i.e., the tops are at the same elevation (+98' -6). Therefore, the south end of the W18 must be notched at the top, as shown in Figure 7-32b, to prevent interference with the flange of the W21. Such a notch is called a cope, block or cut. Some shops dimension such cuts while others show a standard cope mark that establishes the required dimensions.

In this text these cuts are shown rectangular in shape, even though some supporting beams may have sloping inner flange faces. Dimensions are given for the depth and length of all cuts, and the intersection of the horizontal and vertical cuts (re-entrant cut) is shaped to a smooth radius to provide a fillet at this point. Although most shops routinely provide these fillets, they must be shown on shop details.

Because the two beams are "flush top," the minimum depth of cut Q_1 shown in Figure 7-32a should at least equal the k-distance for a W21×73 to clear the web fillet. The length of cut Q_2, measured from the back of the connection angles, should provide $^1/_2$-in. to $^3/_4$-in. clearance from the toe of the flange of the W21:

$$\begin{aligned} Q_2 &= \frac{b_f - t_w}{2} + \tfrac{1}{2} \\ &= 3\tfrac{7}{8} + \tfrac{1}{2} \\ &= 4\tfrac{3}{8} \text{ in.} \end{aligned}$$

Use $4^1/_2$ in.

Dimensions giving the depth and length of such cuts usually are rounded up to the next $^1/_4$ in.

Clearances between beam flanges and column flanges must be considered when beams frame to column webs. Additionally, if a bolt or detail connection projects beyond a clearance line, it must be considered in the overall erection scheme and removed or omitted if necessary.

Dimensioning

A complete shop detail and the material billing for the W18×60 of Figure 7-28 is shown in Figure 7-29. Note the following:

- The minus dimension or setbacks ($-^5/_{16}$), shown at each end of the main dimension line showing the beam length (19'-$11^3/_8$), are distances from the extreme end of the beam to a major reference line such as the centerline of the supporting beam or column. For a beam framing to other beams the setback dimension is made equal to $t_w/2 + ^1/_{16}$ in. and usually referred to as the c-dimension, t_w being the thickness of the web of the supporting beam.

- The dimension back-to-back of end connection angles is obtained by subtracting the sum of the two setback dimensions from the center-to-center dimension between supporting beams. In the case of beam B1 this dimension is equal to 20 ft-0 in. $-$ 2 $\times$ $^5/_{16}$ = 19 ft-11$^3/_8$ in.
- The length of the W18×60 is billed at 19 ft-10$^1/_2$ in., so that its ends will stop approximately $^1/_2$ in. short of the backs of the connection angles. This allows for inaccurate cutting of the beam length at the mill or in the shop and eliminates possible recutting or trimming. In Figure 7-29 the ends of beam B1 theoretically are located $^7/_{16}$ in. short of the backs of connection angles. Although not shown, the edge distance on the end of the beam is 2$^1/_4$ $-$ $^7/_{16}$ = 1$^3/_{16}$ in. This edge distance is less than the minimum requirement for "sheared" edges, 1$^1/_4$ in. for $^3/_4$-in.-diameter fasteners (see Table J3.4 of the AISC *Specification*). Although the beam would have been saw-cut or gas-cut (not sheared) to length, the connection available strength might be reduced by application of AISC *Specification* Section J3.10.

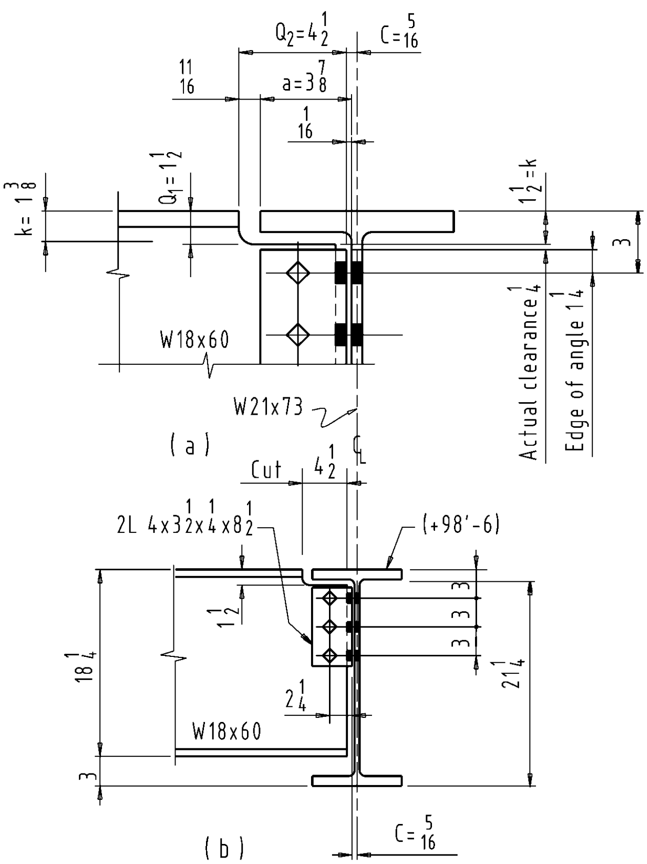


Figure 7-32. Cope, block or cut.

- Some shops require a minimum 1$^1/_2$-in. edge distance for clamping or gripping main material in a multiple-punch machine, regardless of the size of hole. Detail material, such as connection angles, need not be governed by this requirement.
- In setting edge distances to obtain the minimum required by specifications or for gripping materials, the mill tolerance for length underrun should be considered. From Table 1-22 of the *Manual*, Part 1, note that W beams 24 in. and under may underrun $^3/_8$ in. and beams over 24 in. may underrun $^1/_2$ in. If this is divided equally between both ends, the maximum underrun at each end will be $^3/_{16}$ in. or $^1/_4$ in., respectively, and edge distances should be set accordingly. The overrun tolerance is the same through a 30-ft length, with a $^1/_{16}$-in. per 5-ft additional increment for lengths greater than 30 ft.
- No top or bottom views have been shown because holes are not required in either flange. Views are not shown that do not convey positive instructions or contribute to clarification of the detail.
- If a shop coat of paint had been specified and the field bolts had been A325 or A490 in a slip-critical connection, a note would have been added to the drawing reading, "No paint on outstanding legs of connection angles." If such a note is applicable to several beams, usually it is placed in the General Notes on the drawing.

In general, details for beams are drawn to a scale of 1 in. = 1 ft-0 in. for beams 12 in. and less in depth, and $^3/_4$ in. = 1 ft-0 in. for beams over 12 in. deep. Good judgment may dictate the selection of a larger scale for small beams (such as 6 in. deep) or beams with complicated details. This scaling applies only to beam depth and to detail fittings, cuts, copes, etc. Overall lengths may be foreshortened, leaving sufficient room to show all the details, drawn approximately to scale where possible without crowding. As dimensions always control, common practice is to exaggerate very small distances to make the view clear.

Dimensions are all-important parts of shop detail drawings. They must be correct, their extent must be unmistakably clear, and all numerals and characters must be legible.

Dimensions should be arranged in a manner most convenient to all who must use the drawing. They should not be crowded and should cross the fewest possible number of other lines. Overall dimensions and long dimensions should be placed farthest away from the views to which they apply. Dimensions should be given to the centerlines of beams, to the backs of angles and, with some exceptions, to the backs of channels. Vertical dimensions should be given to the tops or bottoms of beams and channels (whichever elevation is critical), never to both top and bottom. In general, dimensions should not be given to edges of flanges of structural shapes or to the toes of angles. With few exceptions, dimensions should be referenced to some point on the steel, i.e., not beyond the extent of the piece.

Shipping Marks, Billing and Notes

As shown under the detail in Figure 7-29, the complete beam is given a shipping mark B1 to identify it for the office, shop and field. A discussion of various systems of shipping marks is given in Chapter 4. On any one detail sketch, only that material required for one complete shipping piece is identified. Thus, in Figure 7-29 two connection angles a are indicated at each end of the beam. The shop bill would show that two beams B1 are required and would list the total number of W18×60 shapes and angle fittings required for two assemblies.

Instead of noting bolt size, open hole size and painting requirements on the details for each individual beam, the usual practice is to specify such requirements only once by a General Note placed on each shop drawing near the title block. Such a note covers all holes and bolts on the sheet, unless exceptions are noted on the individual details.

Typical Framed Beam Details

Figure 7-33 is a part erection drawing of a design plan. The W36×160 girders (A2 and B2) have been detailed and the holes required for the connections of beams framing to them are shown in Sections X-X and Y-Y. The steel detailer determined the number of bolts required by using *Manual* Table 10-1 to find connections having design strengths at least equal to the given factored reactions of the beams. Although fewer bolts would have sufficed for the W24×76 beams, the number of rows of bolts selected represents the minimum required (see "Recommended Angle Length and Thickness" in the *Manual* Part 10). The following discussion will cover the details of beams A3 through G3. Beam G4 is actually opposite-hand of E3, and M4 is opposite-hand of F3 with holes to accommodate G3. To avoid potential fabrication errors, beam G4 was detailed completely instead of noting the piece to be a "Left" or "Opposite-hand" of member E3. A simple CAD command applied to the detail of E3 produced the detail of G4. Beams A2 and B2 are indicated to be detailed on drawing Sheet 2, with G4 and M4 detailed on Sheet 4. The details of the unmarked W24×94 and W24×76 beams will resemble beams B3 and A3, respectively, the differences being in locations of web holes.

Note that the shipping (erection) marks of the various members are shown in Figure 7-33. While a separate erection drawing may be made, in this case the steel detailer ascertained that the design drawing could serve as the erection drawing. The system of shipping marks in this case consists of a capital letter followed by the number of the shop drawing. The shop drawing number is Sheet 3 (see Figure A7-34). Generally, the web view of a beam is drawn as it would be seen if the steel detailer were looking toward the top or toward the left side of the design plan. Therefore, in this case one should face the north in detailing the east-west beams and face the east in detailing the north-south beams, except for the C10×15.3 (E3), which is detailed by viewing from the back.

In detailing this framing it was required that, once gage lines are established on any beam, all bolts and holes in that beam must be on those gage lines. Furthermore, all breaks in gages, as between framing beams, must be made in the connection angles. This common practice saves money in the shop and tends to reduce drafting errors. Beam connections that are adequate to resist the given reactions are selected from the *Manual*, Part 10 and will be used throughout for beams and channels. These connections will be modified as required because of breaks in the gage lines.

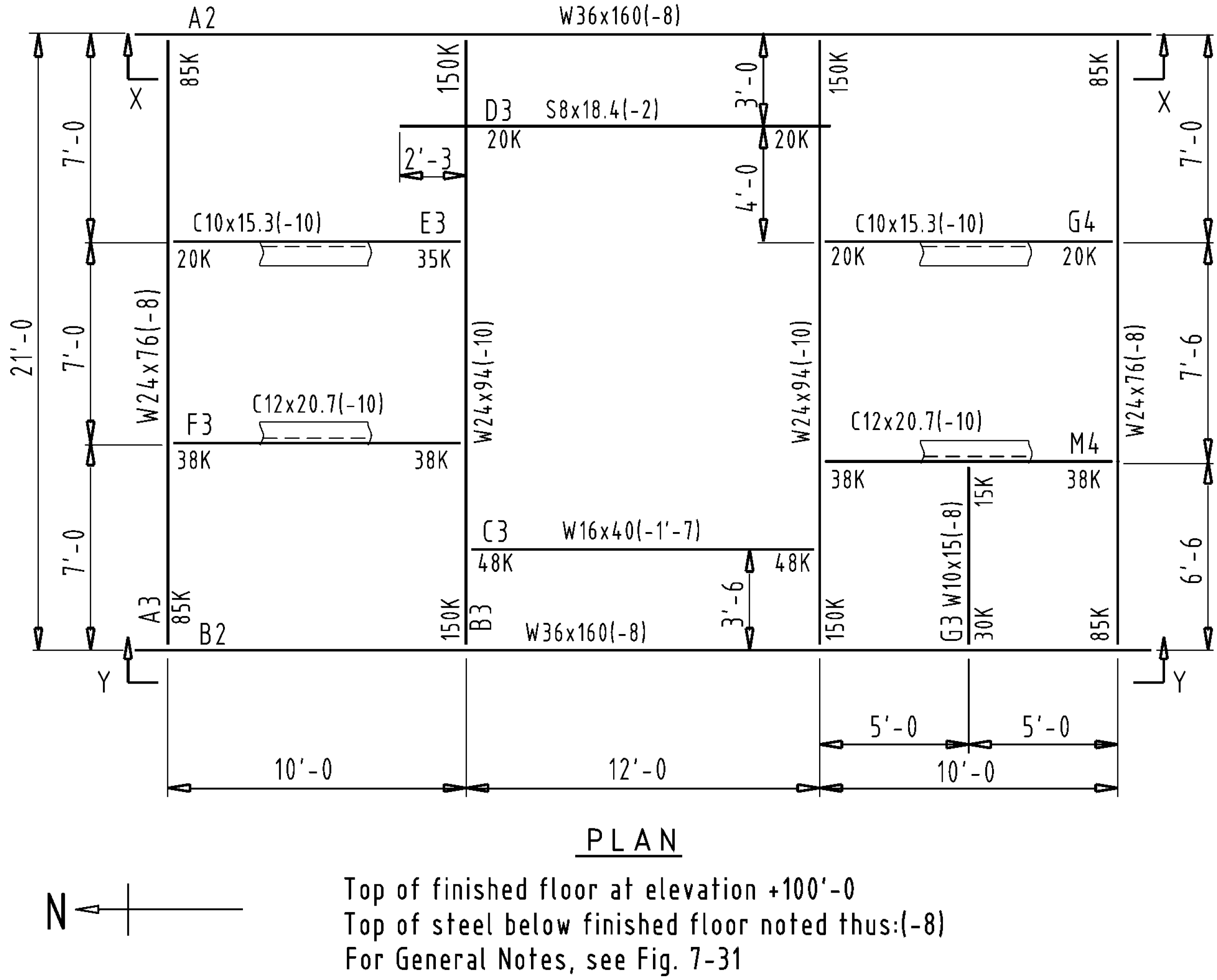
In Figure A7-34 the shop drawing of beams A3 and B3 shows holes that have been provided for connections of beams that will frame into them.

In setting gage lines along the web of a beam the lines should be located to accommodate all of the pieces that are to frame into the beam web and, at the same time, accommodate the end connections of the beam. Breaks to adjust the gage lines selected are provided in the connection angles at both ends of beam B3, at the north ends of beams E3 and F3 and at the east end of beam G3.

On the detail of beam B3, the holes in the top flange must match holes in the bottom flange of the S8×18.4 beam D3. (Note from Figure 7-33 that the top of beam B3 is 8 in. below the top of beam D3.) These top flange holes are noted on the detail of D3 to be on a 2¼-in. gage. The transverse distance between gage lines on the top of beam B3 is given by the note "GA = 5½." The holes are understood to be placed symmetrically about the centerline of the beam web. If only two holes were required (probably located on both gage lines diagonally opposite each other), a top view would be advisable.

The top of beam B3 is cut away at both ends to accommodate the end connection angles that otherwise would encroach on the fillet of the top flange. The left end of channel F3 and the right end of channel E3 are cut at either the top or bottom flange for the same reason. As no question of clearance for the flange of an adjoining beam is involved in these cases, the Q_1 dimension of these cuts (see Figure 7-32a) is made sufficient only to remove the flange (Q_1 in these cases is equal to the *k*-distance of the member being detailed, rounded-off to the next larger ¼ in.). The Q_2 dimension is made sufficient to provide about ½-in. to ¾-in. clearance from the toe of the framing angle.

Beam C3 requires a deeper cut at the bottom of the beam at each end in order to provide clearance for the bottom flange of both W24×94 beams. Note: All flange cuts are dimensioned to the ends of the beam.



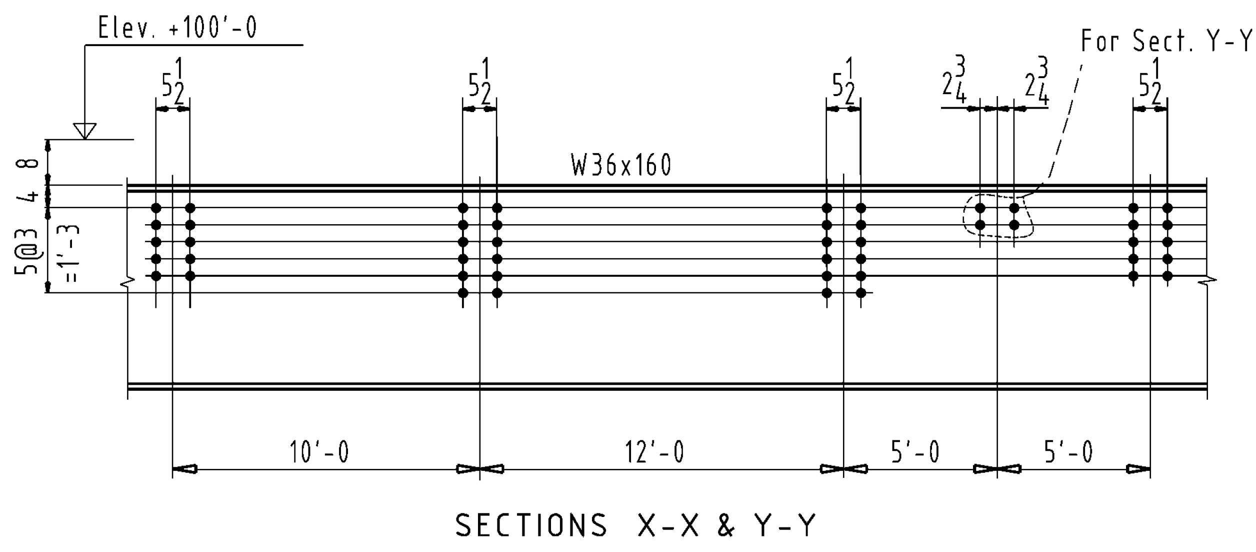


Figure 7-33. Part erection drawing of a design plan.

The notation ± 0 at the left end of beam D3 indicates that no overrun or underrun is permitted at this end. The steel detailer must assume that the 2 ft-3 in. overhang must be maintained as closely as possible. To give the shop some leeway an allowable variation of $+\frac{1}{2}$ in., $-\frac{1}{4}$ in. in length is shown at the right end, as nothing on the design drawing is given to limit the exact location of this end. This will allow the beam to overrun or underrun the full mill tolerance, so that cutting the beam in the shop should not be necessary. Refer to Table 1-23 for "Permissible Variations in Length" for S-shapes in the *Manual* Part 1. A bottom flange view is not needed to locate the holes because the notation "GA $= 2\frac{1}{4}$" conveys to the shop this necessary information.

Note that the main material (the beams and channels) is identified by a shipping mark in the details shown on Sheet 3 (Figure A7-34). Some fabricators assign assembly marks to the main material if it consists of only a beam or channel. The exception to this latter practice is beam D3. As this beam is plain (it has no fittings), it can be identified by its shipping mark.

Channels E3 and F3 are drawn looking at the backside of the web, which is the usual detailing practice for these shapes. The direction in which the channel flanges are to face has been indicated on the erection drawing by showing a small section of the flange (see Figure 7-33). When they are installed, the flanges of these channels must point in the direction indicated.

The ditto mark (") is never used on structural steel shop drawings. In the shop bill always use the word ONE, not the number 1, in designating one shipping piece.

Neatness and legibility of shop drawings are obtained by lining up notes and dimensions that have the same purpose. Thus, the 4-in. cut instructions at both ends of beam B3 are placed the same distance above the sketch. Attention to these features results in an orderly, systematic presentation of the information and helps prevent shop errors.

The flange cut at the left ends of beams G3 and E3 has reduced the beam design strength significantly. Beam E3 was reinforced by using a longer leg on angle a3d. Beam G3 was reinforced by the addition of the $\frac{1}{4} \times 6$ in. plate p3a. The steel detailer is referred to the *Manual*, Part 9 for discussion and examples, which are useful in analyzing coped beams.

The shop drawings presented thus far are, of necessity, very simple. They have been explained in considerable detail because they illustrate many of the fundamentals that are common to the details of all ordinary framing. These fundamentals must be understood clearly.

Dimensioning to Channel Webs

On the design drawing in Figure 7-33 dimensions are given to the backs of the 10-in. and 12-in. channels, which is the usual dimensioning practice for these shapes.

Most fabricators follow the generally accepted practice of referring dimensions on shop drawings to the backs of channels. Beams A3 and B3 are detailed in this manner. (The practice of dimensioning to the centerline of channel webs is preferred by some fabricators and their shop workers have been trained accordingly.)

Note: Longitudinal dimensions along the beams shown in Figure A7-34 are given to the centerlines of the groups of open holes required for the field connections, rather than to each transverse gage line as in the case of the groups of shop bolts. This practice simplifies the dimensioning work for the steel detailer and, later, for the checker as the dimensions given on the design drawing and erection drawing are to the centerlines of the beams (an exception in the case of channels already has been noted).

Use of Extension Dimensions

Instead of locating groups of holes by means of a series of dimensions giving the distance between the backs of adjacent channels, as on beams A3 and B3 in Figure A7-34, many fabricators prefer the use of extension dimensions. Extension dimensions are distances from a definite point at the left end of the beam to any line or group of holes or to a connection. To avoid any misunderstanding the dimension line for the extension figure, locating the first connection from the left end of the beam should run unbroken to the point of origin from which the extension is given. In Figure A7-34 the point of origin for each beam is labeled RD, which stands for "Running Dimension." It is the left end of the beam. The backs of the connection angles are set back $\frac{1}{2}$ in. from the RD.

In Figure A7-34 the extension dimensions are taken to each line of holes in each group, moving from left to right along the beam.

Extension dimensions taken to each line of open holes are shown for beams A3 and B3 in Figure A7-34. For beams with square connection angles or with connection angles with small bevels (as defined in Figure 7-35) on at least one end, that end should be made the left end and extension dimensions given from the back of the left end connections.

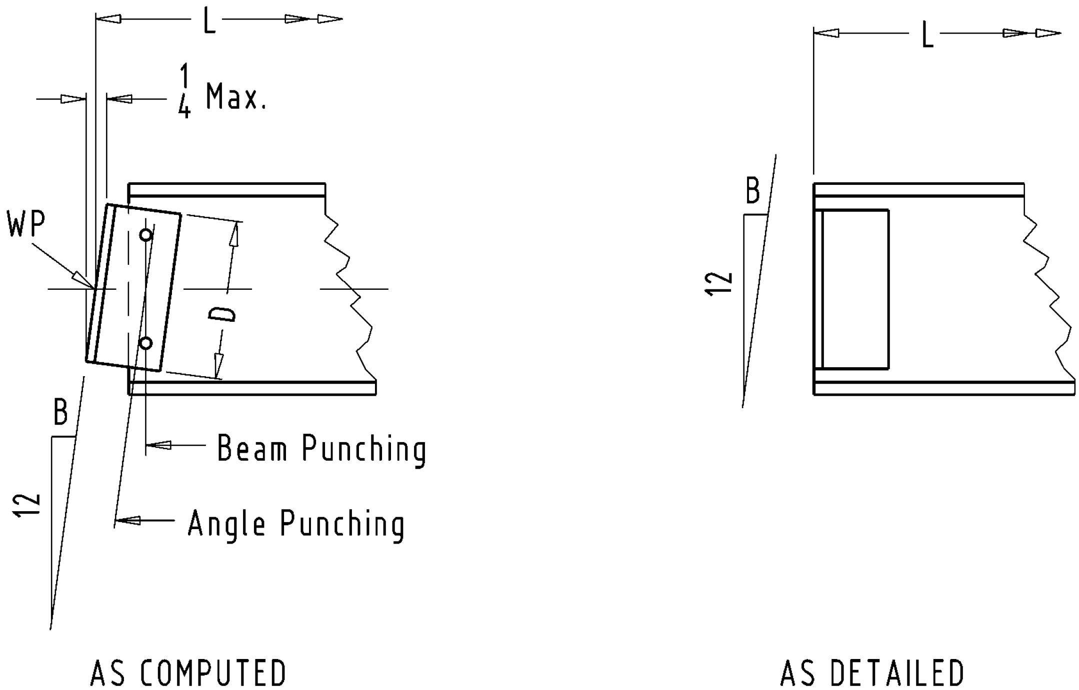
For beams with at least one square end and no connection angles attached to either end, the square end should be made the left end and the extension dimensions given from it.

When a beam has one end milled (± 0 tolerance), that end must be shown at the left end of the beam sketch and all extension dimensions measured from it.

When beams are fitted with beveled connection angles (except for small bevels as noted above), extension figures should be given as shown in Figure 7-36. Work points should be noted WP.

When beams are cut on a vertical bevel and the left end is plain or has holes, extension figures should be given from

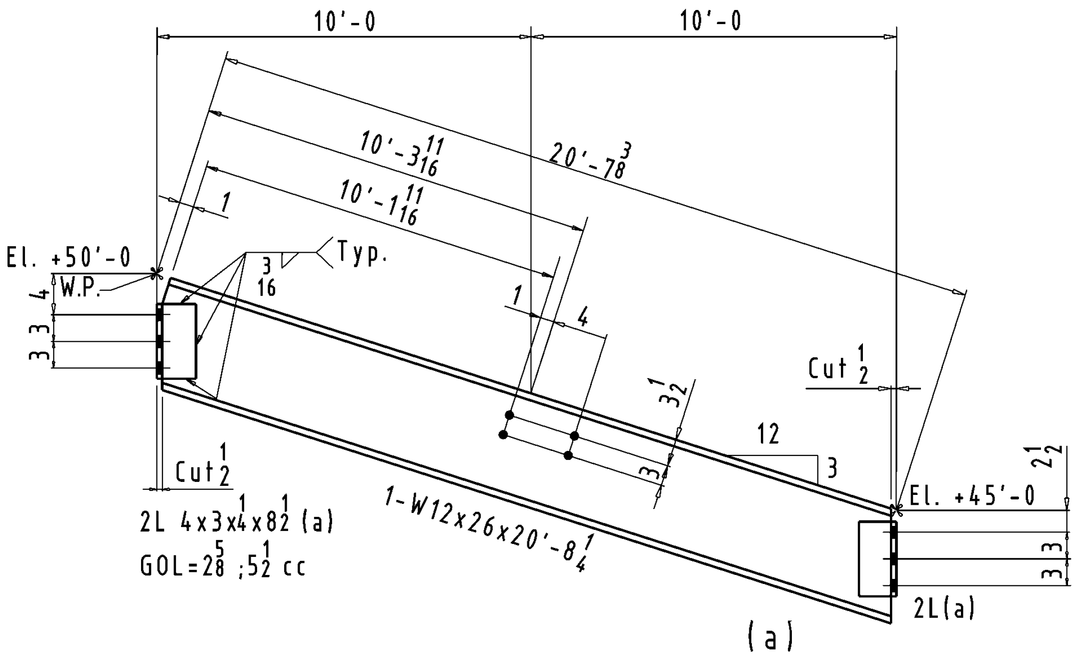
SMALL BEVELS
BEAMS CUT SQUARE; ANGLES SET TO BEVEL BY SHOP



MAXIMUM D FOR 1/4" SKEW							
B	1/8	3/16	1/4	5/16	3/8	1/2	1
D	24	16	12	10	8	6	3

When skew in length of angle does not exceed 1/4", both beam and angles may be punched square and fitted by shop to required bevel. Detail length "L" is determined at mid-length of angle.

Figure 7-35. Connection angles with small bevels.



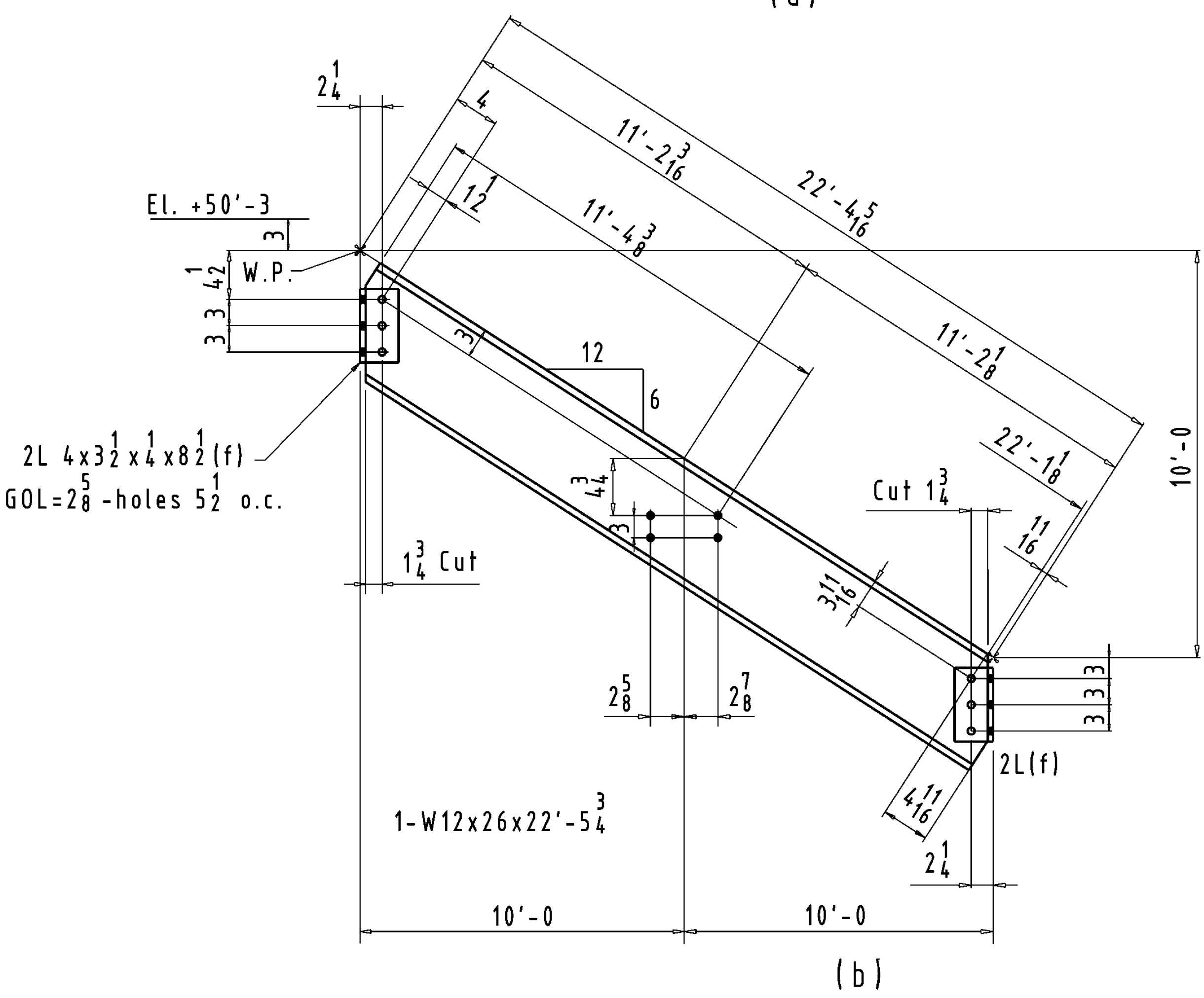


Figure 7-36. Beams cut on a vertical bevel.

the left end of the beam similar to Figures 7-36a and b, respectively.

Groups of holes or connections closer than 1'-0 to either end connection of a beam, or to an intermediate connection, should be located by dimensions from such adjacent connections and not located by extension figures only.

When holes are formed in a beam with a multiple punch or drill, it is not necessary to lay out and center punch the location of each hole. The beam is clamped to a movable carriage that can be stopped when any given point along the beam is in position for punching or drilling, as explained in Chapter 1.

Framed Connections to Columns—Bolted

Framed end connections also are used to connect beams to column flanges. In the case illustrated in Figure 7-37, a pair of connection angles (or framing angles) is shop bolted to the column flange. Note the stagger between the shop bolts and the holes for the field bolts. This is necessitated by the narrow (4-in.) gage on the column flange in order to provide clearance for inserting and tightening the field bolts. As this is a "knifed" connection, the bottom of the beam is coped to permit it to slide down between the angles during erection. The $^1/_2$-in. gap between the angles furnishes the acceptable $^1/_{16}$-in. clearance for erecting the beam.

The beam connection shown is to a W8×24 column (with a $6^1/_2$-in. flange width), necessitating a maximum column gage of 4 in. to provide sufficient edge distance for the holes in the column flange. The gage in the angle legs is $1^3/_4$ in. The holes in the column flange are staggered with those in the beam web; otherwise clearance for the impact wrench in tightening the field bolts would be insufficient. Horizontal short slots in the outstanding legs of the angles allow for slight variations in the column gage.

The connection shown in Figure 7-37a permits detailing beam B as a plain punched beam. Such a beam carries no fittings (see Figure 7-37b) and can bypass the assembly areas in the shop. Most shops prefer plain punched beams and detail the beam connections for shop assembly on the columns where possible. Placing the connection angles on the column flange affords more flexibility in plumbing up the steelwork in the field before tightening the field bolts. Many fabricators and erectors prefer this procedure.

A detail such as the one in Figure 7-37 is not generally used to frame a beam to a column web because of interference of the column flanges in tightening the field bolts with impact wrenches and requirements for erection seats. The connection angles could be shop fastened to the beam. However, this may present clearance problems in erecting the beam between the column flanges.

Design procedures for framed connections to columns are the same as for beam-to-beam connections, discussed in Chapter 3.

Seat Details—Bolted

Figure 7-23a is an example of a part plan taken from a design plan. The columns have been detailed and Type C seated connections have been provided at each column for the W18×71 beam B.

The steel detailer is advised to study gages, erection clearances and flange cuts required for connecting a beam to two different sizes of columns, as shown in Figure 7-23a, and to prepare a sketch such as Figure 7-24. The sketch is discarded later after it has served its purpose. (The sketch shown in Figure 7-24 may show more information than is necessary and is for illustrative purposes only. With experience the steel detailer will be able to make these study sketches freehand.)

Figure 7-38 shows a completely developed shop detail of beam B with top and bottom views. These latter views

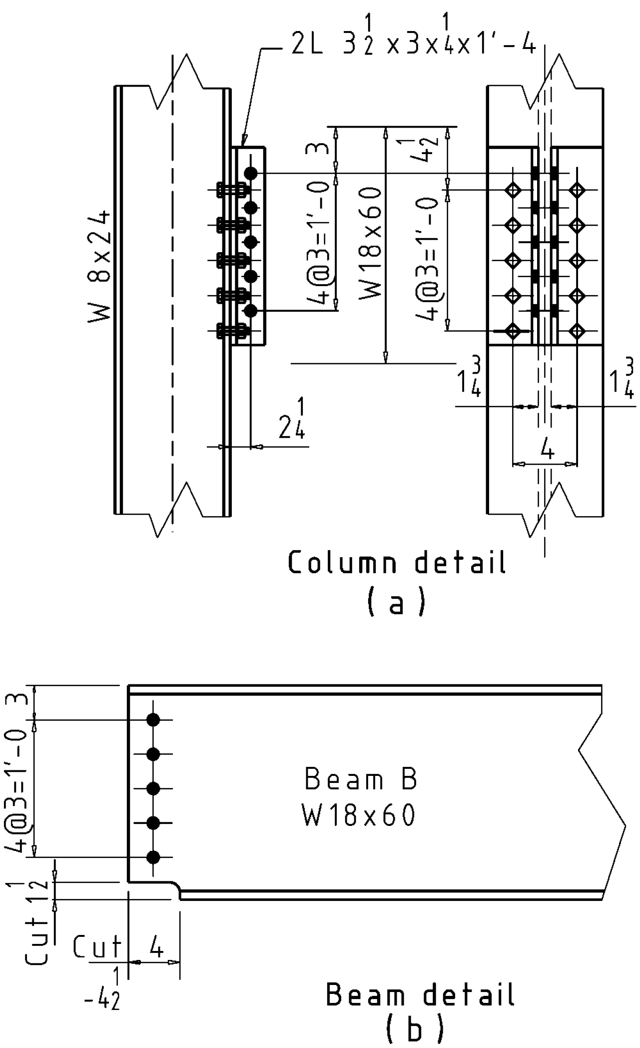


Figure 7-37. A pair of connection angles (or framing angles) shop bolted to the column flange.

customarily are not shown on shop drawings. Figure 7-39 shows the preferable complete “short cut” shop detail of the same beam. Both figures contain exactly the same information. Note the following points:

The minus dimensions ($-3/4$), shown outside and opposite the dimension line are the setback dimensions, the distances from the centerline of the supporting columns to the end of the beam.

The ordered overall length of the beam is used to determine the setbacks by subtracting its length from the distance center-to-center of supporting columns. In this problem each setback equals:

The 18 ft-$4\frac{1}{2}$ in. length of the W18 is the ordered length, probably determined long before the detail is made. Had it been ordered slightly longer or shorter, it still would be used as ordered, provided that erection clearances are adequate, end or edge distances meet minimum AISC *Specification* requirements, and the design bearing strength is adequate.

The plus-and-minus figures ($\pm 1/4$) shown at the ends of the web view tell the shop that the beam length can vary up to total of $1/2$ in. shorter or longer than detailed. Therefore, the beam would be usable without trimming, provided its length is between 18 ft-4 in. minimum and 18 ft-5 in. maximum. The shop would adjust the edge distance, shown as $2\frac{1}{4}$ in., and the length of clearance cuts, shown as $4\frac{1}{4}$ in., to suit. The longitudinal distance between open holes would be held to 18 ft-6 in. − 3 in. − 3 in. = 18 ft-0 in. to match the column details (see Figure 7-24). This distance cannot be varied. The steel detailer should endeavor to allow the shop as much variation in length as possible, up to the full mill tolerance. If full mill tolerance or something approaching it can be allowed at each end, the shop does not have to divide the mill overrun or underrun equally at each end. For many shops, the ($\pm 1/4$) tolerance is a standard amount and the steel detailer does not have to show it on the shop drawings. However, tolerances other than standard must always be shown. The detail should meet the standards of the employing fabricator.

If detailing requires both a top view and a view of the bottom flange, the practice of most fabricators, when showing a bottom flange, is to view it looking down as illustrated in Figure 7-38.

Detailing as in Figure 7-39 eliminates the flange views. Instructions (including necessary dimensions) for cutting the flanges at the right end are covered by a note on the web view. The radius of each corner fillet at the flange cuts is understood universally and need not be noted on Figure 7-39. The transverse distance between gage lines on the flanges is covered by the note “Ga = $3\frac{1}{2}$.” In such cases symmetry about the centerline of the beam web is understood. The notes must be explicit in showing what work, if any, is required on each flange.

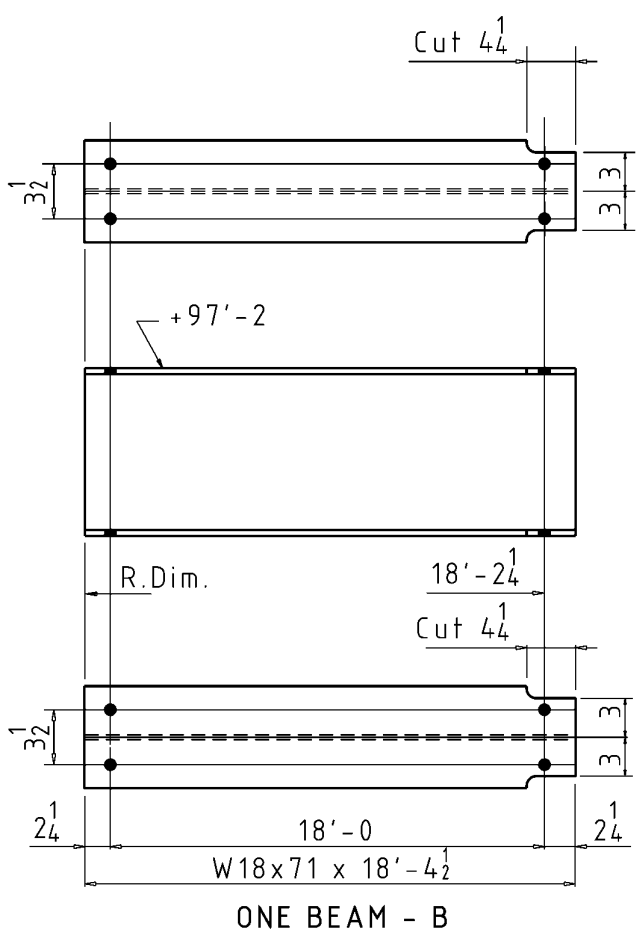


Figure 7-38. Completely developed shop detail of beam B with top and bottom views.

Typical Framed Beam Connections—Welded

Three similar beams are detailed in Figure A7-40 to illustrate the use of different types of welded framed connections. Beam B10 has Case I connections, shop welded and field bolted; beam B11 has Case II connections, shop bolted and field

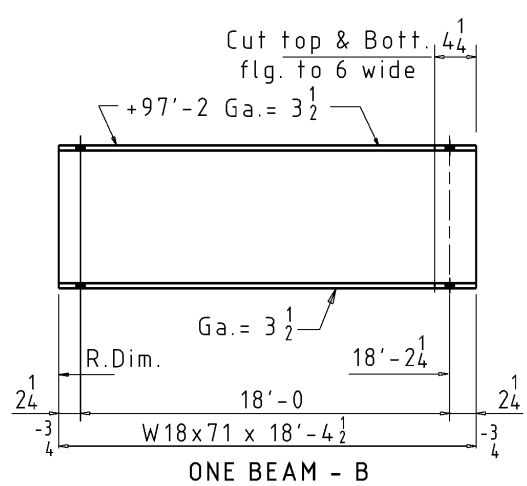


Figure 7-39. “Short cut” shop detail.

welded; beam B12 has Case III connections, shop and field welded. The details follow the principles discussed earlier for bolted connections. The use of welding symbols follows the rules discussed in Chapter 4.

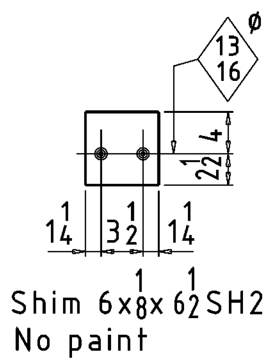
Where open holes for bolts are used in combination with welds, the connection angles are located on the beam by the gage lines established for the beam or the supporting steel. Where shop and field welding of end connection angles is used, a dimension is shown to the open holes provided in the outstanding legs for erection bolts. Note that welding and bolting are never used in the same connection plane except under rare and special conditions.

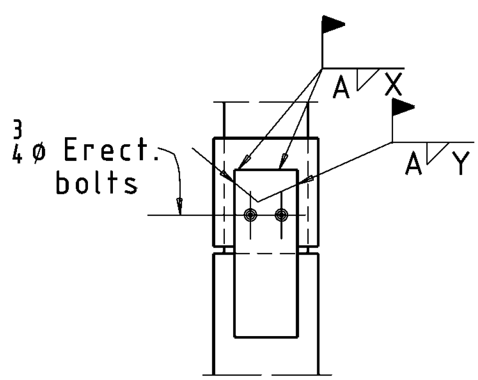
Four holes for erection bolts have been provided in the connection angles for beams B11 and B12. Two bolts would have been sufficient to support the dead weight of the beam. The additional bolts serve two purposes: they provide for the contingency of large temporary loads during erection and they assist in pulling the connection angles up tightly against the web of the supporting beam. Generally, these bolts are not required to be removed subsequent to final welding.

OSHA safety regulations require that two bolts be installed for erection. As a general rule at least two erection bolts are used for framing angles or similar connecting elements up to 12 in. long, four bolts for connecting elements up to 18 in. long, and six bolts for longer connecting elements. Some engineers prefer to locate these bolts below the mid-height of the angle. Theoretically, this provides the greatest possible flexibility near the top of the connection, where the angles are expected to flex away from the supporting member. However, this practice does not ensure a close fit-up of the angle before welding. Other engineers prefer the more general practice of spacing the bolts equally along the length of the angles. In this latter case, the bolts are placed as close to the toes of the outstanding leg as practical to provide greater flexibility.

The steel detailer should study carefully the details for beams B10, B11 and B12 in Figure A7-40 and note the following:

The setback for beams B11 and B12 is made 5/16 in. at the right end to provide a closer fit of the field-welded connection





Col. Mark	Floor	Size A	Length X	Length Y
D4	2	$\frac{1}{4}$	2	2
	4	$\frac{5}{16}$	5	5

Figure 7-41. Typical splice weld.

angles against the supporting members after the beam is erected. However, some fabricators would use $^3/_8$ in. at each end. The field weld size must be increased by the size of any gap existing at field fit-up in accordance with AWS Code D1.1 requirements.

Although the welding symbol on beam B10 indicates shop welding of the near-side connection angle only, by convention the far-side angle is to be welded the same as the near-side as long as it carries the same assembly mark. See note at the bottom of Figure 4-20.

The welding required for each detail piece identified by an assembly mark may be shown only once with the note "Typ" in the tail of the welding symbol. The welding is presumed to be duplicated in all other uses of that assembly mark on that particular shipping piece, provided no other welding symbol is shown (Figure 4-27). This rule applies for a single shipping piece. If the same detail piece assembly mark appears on another shipping piece on the shop drawing, the welding symbol must be repeated on that piece.

The drawing carries the note, "All welds to be made with E70XX electrodes." Noting the type of electrode to be used is advisable even if E70XX electrodes are used commonly unless otherwise noted. The E70 designation indicates the design strength level of SMAW welding. Other welding processes also are acceptable, unless prohibited by contract, provided the same design strength is furnished.

As the edge distance at the end of the connection angles for beam B12 is not the usual $1^1/_4$ in., the dimension of 6 in. is shown on the detail of the connection.

Seat Details—Welded

Figure 7-42 is a part plan of an all-welded project. In detailing beam B13 and its connections (Figure 7-42b), important considerations are:

- As the design dimensions are shown from center-to-center of columns (Figure 7-42a), the minus dimension ($-7^3/_4$ and $-7^1/_2$ in.) is given from the center of the column.
- The overall length of the beam is 20 ft.$-$ ($7^3/_4$ in. $+$ $7^1/_2$ in.) $=$ 18 ft-$8^3/_4$ in.
- Holes have been provided in the bottom flange of the beam for two erection bolts. The beam will be field welded to both the seat angle and the top angle. Shop painting of the steel is not required according to the "Notes" in Figures 7-42b and 7-42c.
- The detail of top angle TA1 is made once as a job standard to suit as many beam flanges and column sizes as practicable. Although the top angle could be shop welded to the column flange, loose fills would have to be provided for beam overrun or underrun. To permit fit-up without fills, the angles will be shipped loose for field welding to the beam and columns.

To furnish the erection contractor instructions for making the field welds for seated beam connections, a drawing must be prepared. By omitting all details not essential to the field welding, a single sketch can be made to serve many sizes of beams, as shown in Figure 7-43. Some fabricators prefer to show the shipping mark for the top angle on the erection drawing at each end of the beam. Others show the top angle shipping mark on the field-weld sketch and relate it to the point numbering.

OTHER TYPES OF CONNECTIONS

The discussion on beam and girder details has dealt with bolted and welded framing angles and seat connections. Although the earlier guidelines for detailing beams and girders apply to the types of connections listed herein, each type of connection has certain features that the steel detailer must recognize.

Shear End-Plate

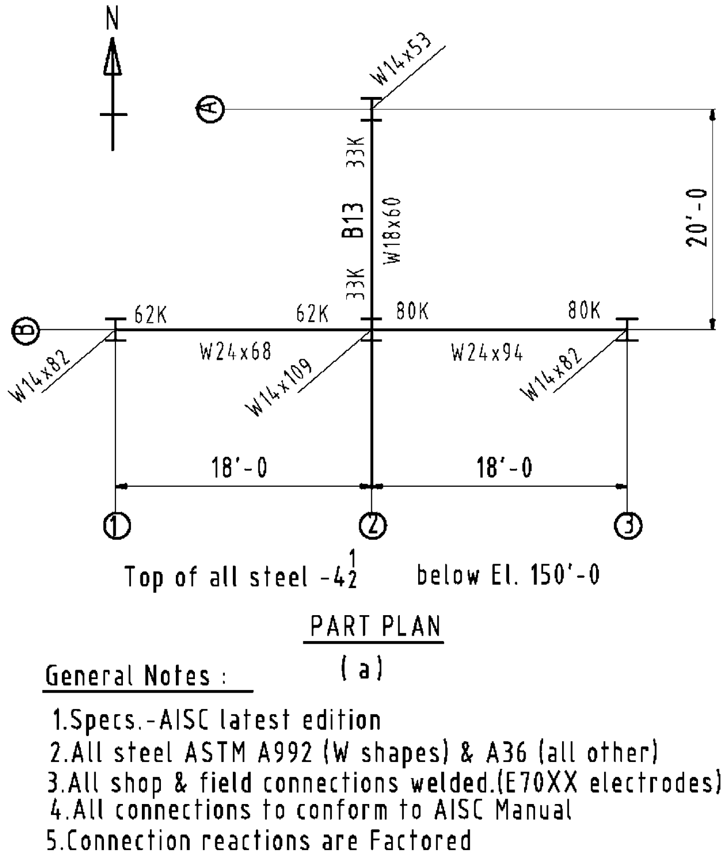
When detailing a shear end-plate connection (Figure 3-8), the steel detailer must give special attention to the locations of holes in the plate. Maintain sufficient installation and tightening clearances for field bolts nearest the insides of the top and bottom flanges. Also, be certain that the line of bolts on each side of the beam web clears the toe of the fillet weld. Generally, rows of bolts are spaced at 3 in. Beams detailed for shear end-plate connections are dimensioned from the outside of the end-plate, similar to beams having double-angle framing connections. Shear end-plate connections can be selected from Table 10-4 in the *Manual*, Part 10.

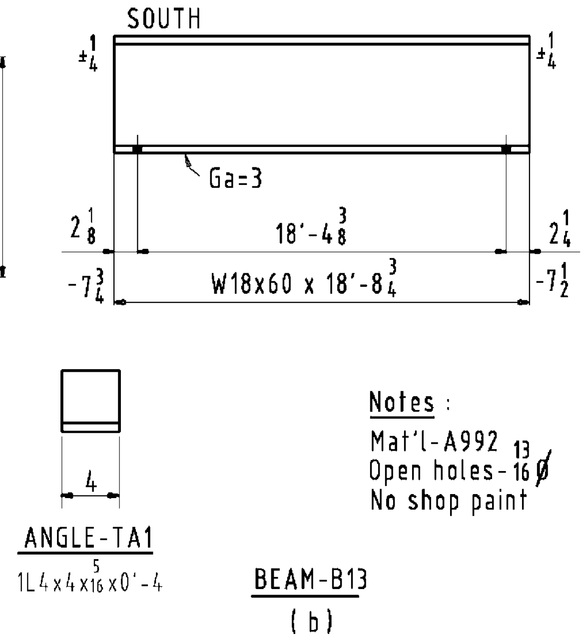
Single Plate

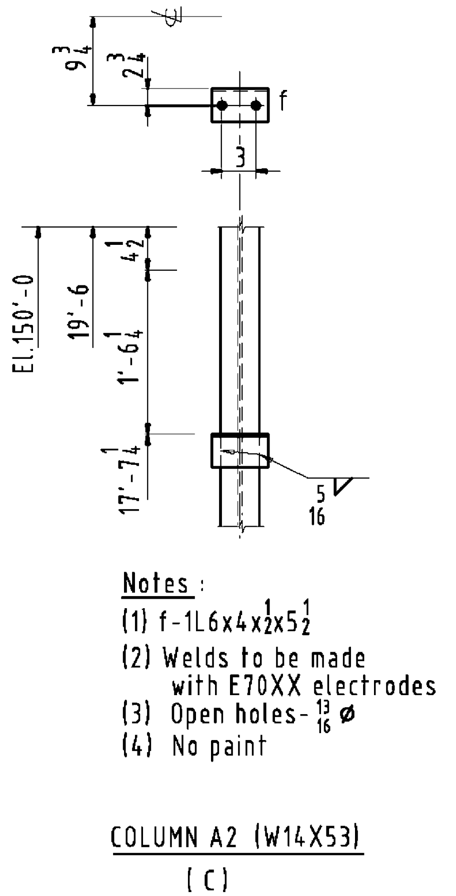
Spacing of rows of holes in single-plates (Figures 3-12 and 3-13) is 3 in., with the top hole in the beam web located usually 3 in. from the top of the beam. If the beam fillet is too large to accommodate the 3-in. location, the top hole in the web can be lowered. The steel detailer must be certain that the end of the beam will clear the toe of the fillet weld connecting the single plate to the supporting member. The steel detailer is referred to Table 10-9 and its introductory discussion in the *Manual*, Part 10 for information on detailing single plates. Beams and girders using these connections are detailed similarly to beams having seat connections. Extension dimensions are measured from the end of the beam.

Single Angle

Beams to be supported by single-angle connections (Figure 3-14) attached to the supporting member are detailed similarly to beams having seat connections. The extension dimensions are given from the end of the web. Where the supported member is to be field bolted to a single-angle connection, horizontal short slots in the outstanding leg of the angle will







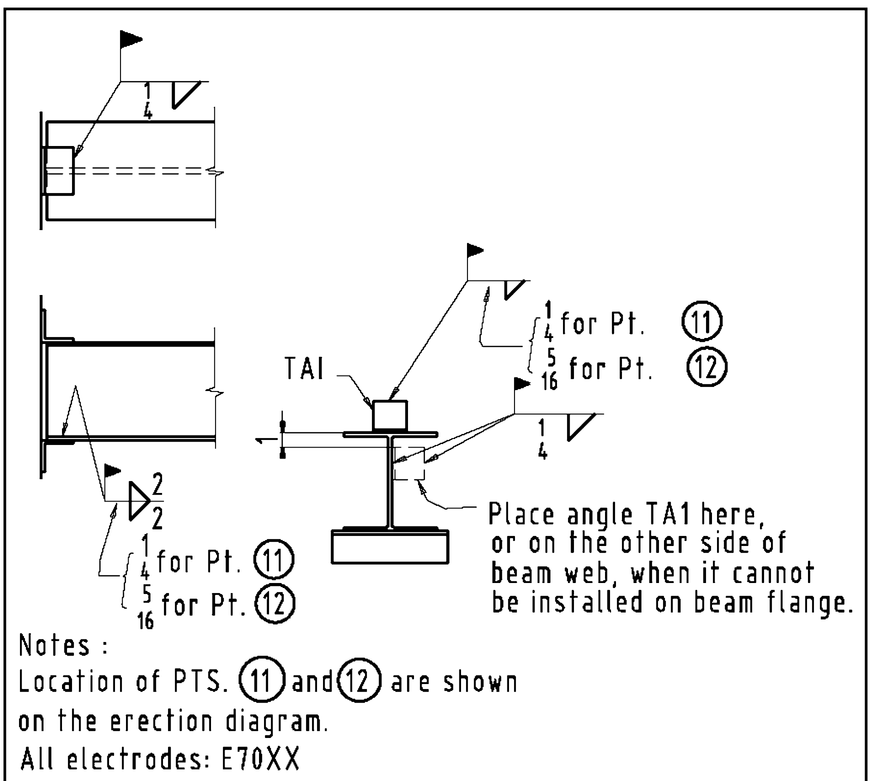


Figure 7-43. Seated connection.

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

facilitate erection. Tables 10-10 and 10-11 in the *Manual*, Part 10 furnish information for selecting all-bolted and bolted-welded single-angle connections.

Tee

When determining the location of holes in the stem (Figure 3-15), the steel detailer must be sure that the end of the supported member clears the fillet of the tee. Also, installation and tightening clearances for field bolts must be observed. Beams detailed for tee connections (the tee being on the supporting member) are dimensioned similar to beams detailed for seat connections.

Camber

Historically, camber has referred to the amount that a beam, girder or truss is built or set above its geometric profile when it is under no stress and lying on its side. Measurement of camber with the member in this position is standard fabricating procedure. Camber is added to beams, girders, trusses and certain other horizontal load-carrying members to compensate, to varying degrees, for deflection of the members when loaded and to improve their appearance by lessening the possibility of sag. Camber that results in an upward rounded arch is referred to as positive camber and is the most common. However, rare cases occur, such as in cantilevered beams, where negative camber is required. Truss camber is discussed later in this chapter.

The camber of a beam or girder will be a comparatively flat, approximately smooth circular curve, continuous from end to end. The amount of camber usually is specified on the design by a notation near the member size such as "C = $1^1/_2$" to indicate a camber of $1^1/_2$ in. On shop drawings the amount of camber is specified as an ordinate from chord to curve at the mid-length of the beam or girder. Below the sketch of the member to be cambered will be a note "Camber = $1^1/_2$" or a similar notation according to the fabricator's preference. The curve in the beam is not shown; the shop will understand this camber to be convex upward unless otherwise noted. As small camber ordinates are impractical, camber less than $^3/_4$ in. should not be specified. Also, the minimum recommended length of beam to camber is 25 ft because of limitations imposed by the fabrication machines generally used to camber beams.

When beams and girders are to be cambered in the shop, the normal procedure is to punch or drill flanges and webs prior to bending. This results in holes that are normal to the flanges and, therefore, on approximately radial lines after bending. If job specifications or other considerations require that holes be punched or drilled normal to the camber, shop drawings must be noted "Holes to be normal to chord." Normally, in building work connections are fabricated square with the cambered beam end. Generally, the rotations and movements are very small and pose no problem during erection. Beam ends, which initially are radial, tend to rotate to a vertical position as loads are applied.

Wall-Bearing Beams

Wall-bearing beams often fall into the category of those steel members that attach to other building components, but may not be shown on the structural steel design plans.

These beams require a bearing or setting plate to distribute the concentrated end reaction to the supporting material. The length of required bearing depends upon the design strength of the supporting material and the magnitude of the loading. The plates are set to the specified elevation and grouted into place before the beam is mounted on them. An alternate detail of bearing on top of a wall is shown in Figure 7-9c. The beam is field welded to the plate unless the beam is required to slide as at an expansion joint. To permit the beam to slide while restricting any other type of movement, 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.

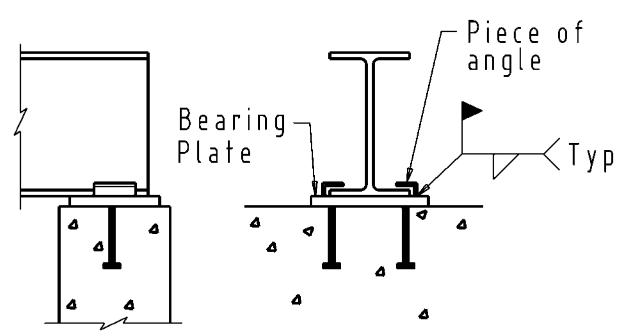


Figure 7-44. Detail to permit the beam to slide while restricting any other type of movement.

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 appreciated after a careful study of the table "Available Strength in Axial Compression" on Double Angles in the *Manual*, Part 4. The total available concentric compression strength of a pair of angles having their long legs back-to-back may be compared directly with the available strength allowed when the same angles, having the same unbraced lengths, are assembled with their short legs together. The orientation of unequal-leg angles should be as specified in the contact documents, unless a change is approved by the structural engineer of record.

As can be noted in the sketch at the top of the tables, the Y-Y axis of the double-angle member will lie in the plane of the stems of the tee sections or the gusset plates when they are placed between the angles. In the plane of the tee section stems or gusset plates, the web members of a truss provide transverse support to the compression top chord at each panel point. Therefore, the unsupported length, as far as the X-X axis of the top chord is concerned, is the distance between panel points.

To prevent buckling from occurring about the Y-Y axis of the top chord when the truss is fully loaded, horizontal support

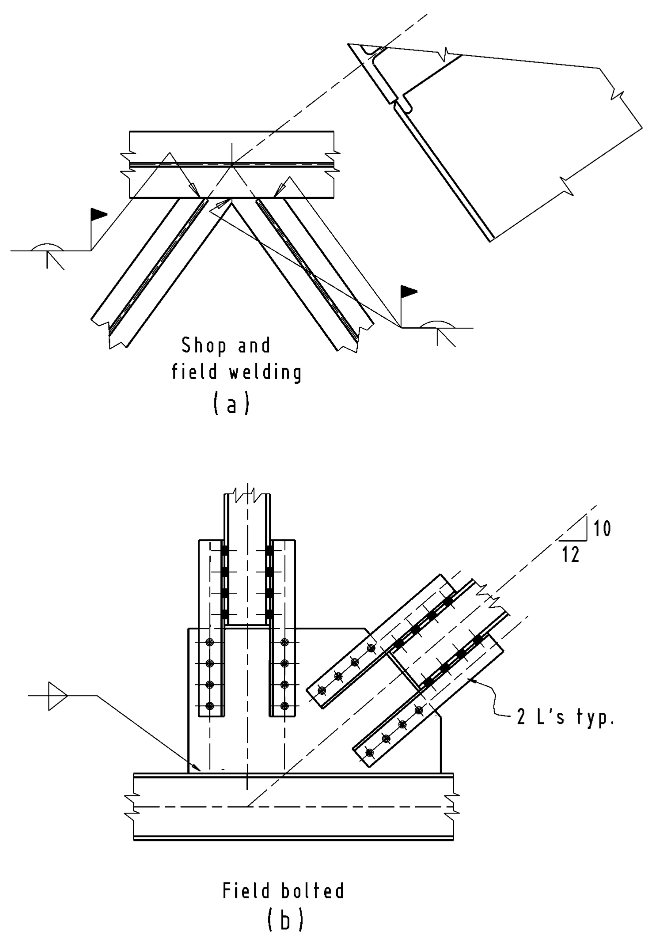
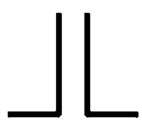


Figure 7-47. Construction of heavily loaded trusses consisting of W-shapes for both chord and web members.

must be provided directly by the floor, the roof construction or a system of bracing. Thus, the unsupported length of the top chord, as far as Y-Y axis loads are concerned, may be zero in the case of a directly connected floor or roof deck. However, if the loading is provided through beams or purlins, the unsupported length may be the distance between these members and may extend over more than one panel length.

When the angles are to be fabricated with their long legs outstanding, this is indicated on the shop drawing by the symbol:

This practice is illustrated in Figures A7-66 and A7-52. When the long legs are to be back-to-back, the symbol



is used. Design drawings must show the relative position of the long and short legs. On detail drawings these symbols are replaced by the angle symbol:

In bolted construction the placement of the long and short legs is frequently obvious from the gages. Otherwise, the size of one of the legs must be dimensioned. In welded construction the size of one of the legs of unequal leg angles always must be shown on the detail drawing.

As an alternate to using symbols to indicate the orientation of back-to-back angles, abbreviations may be used, such as: LLBB (long legs back-to-back) or SLBB (short legs back-to-back).

TYPICAL DETAILING PRACTICE

General Arrangement of Details

Figures 7-48a and 7-48b are erection drawings for the roof framing of an industrial building. The trusses are detailed in Figure A7-49. Although these trusses might not be too large to be shipped as single pieces, this might not be true of similar larger trusses. Therefore, the details have been prepared to illustrate the usual separation (into half trusses, bottom chords, hangers and knee braces), which normally would be used in the fabrication and shipment of larger trusses.

Note in Figure 7-48a that instead of showing shipping marks for the component shipping pieces comprising each roof truss, each fully assembled truss is given an erection mark—T1, T2 or T3—and these erection marks are shown on both the top chord and bottom chord plans. The trusses will be assembled on the ground, using the information provided on the assembly diagram, and their field splices will be bolted completely before they are lifted into position.

Even when trusses are to be shipped to the site in more than one piece, the several component shipping pieces usually are detailed in their assembled position.

Trusses of the type shown in Figure A7-49, after having been assembled on the ground, are vulnerable to "jack-knifing" when being lifted (i.e., buckling out of the plain of the truss and bending sideways). One location where this may occur is at the splice of the bottom chord members. Another location on the truss that is vulnerable to jack-knifing during erection is at the peak. The bent plate ac, with a four-bolt connection to each half truss, would be considered incapable of resisting lateral bending. Stiffening angles can be provided as shown in Figure 7-50, to resist lateral bending. The extent to which stiffening of the truss at these locations is required would be determined by the erector. An option to stiffening the truss might be for the erector to arrange the pick points to reduce the possibility of a lateral buckling failure.

Decisions as to which connections are to be field connections and which are to be shop fastened generally would have been made at the Pre-construction Conference discussed in Chapter 5 of this text. Sometimes, however, the decisions are made after the conference during discussions between the fabricator/steel detailer and the erector. Individual shipping pieces must be:

- Stiff enough to be handled without damage
- Small enough to fall within shipping clearance limitations
- Light enough to be lifted by available equipment
- Capable of being erected without interference from other framing

Within these limitations, shipping pieces are usually arranged so that only a minimum amount of fastening will be necessary in the field.

Layout and Scales

The first step in preparing shop details of trusses and bracing is to establish work points. The distance between work points is computed, as well as the bevels which sloping working lines make with the horizontal or vertical. After working lines and points are established and distances calculated, a scaled layout is usually made of each joint. These may be placed on work sheets (not issued to the shop) to be later placed on the shop detail drawing to a smaller scale. Generally, they are made in advance at the time material is being ordered. The layouts of the joints sometimes are made directly on the shop drawing, instead of separate layout sheets, and they, then, become a part of the finished detail.

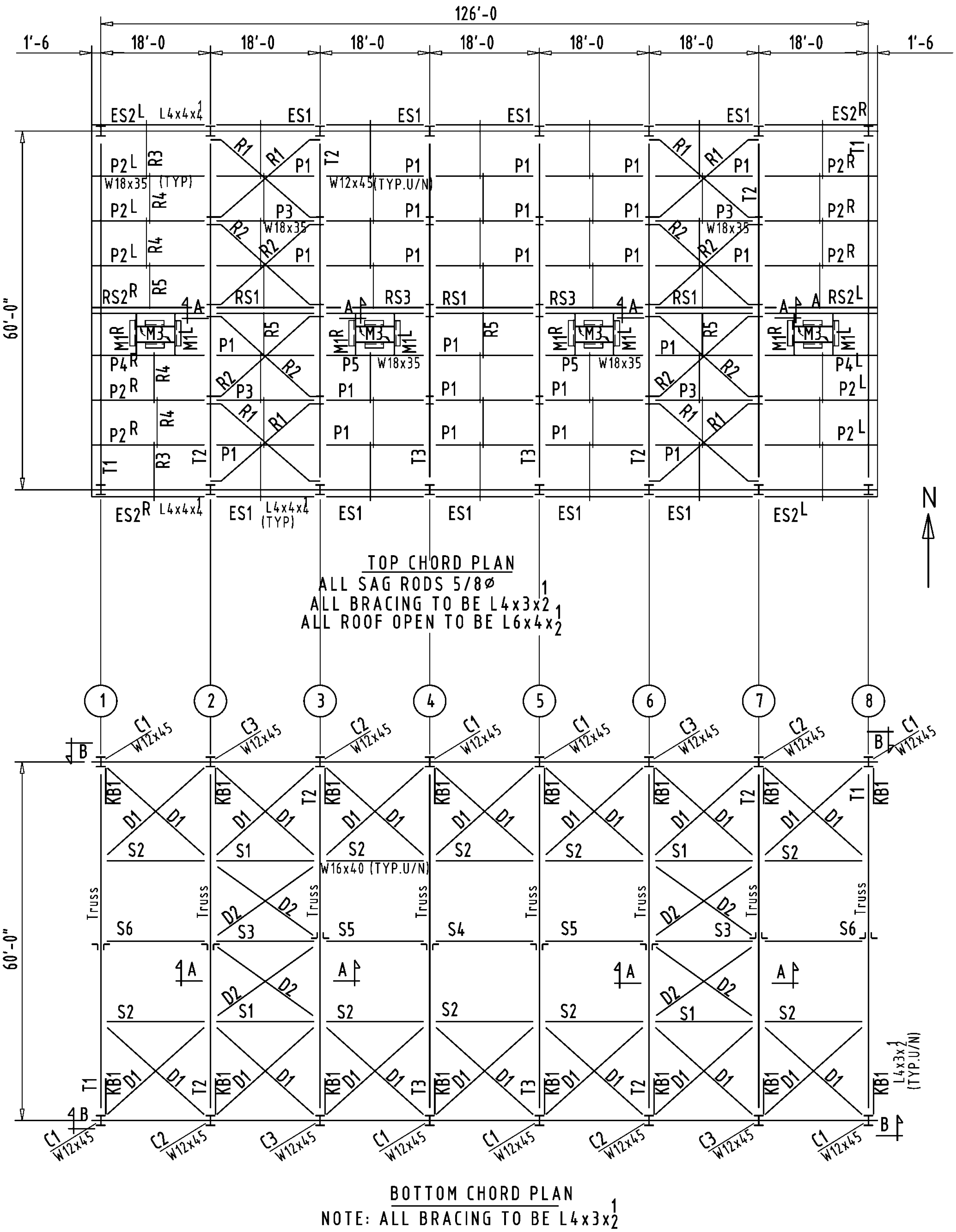
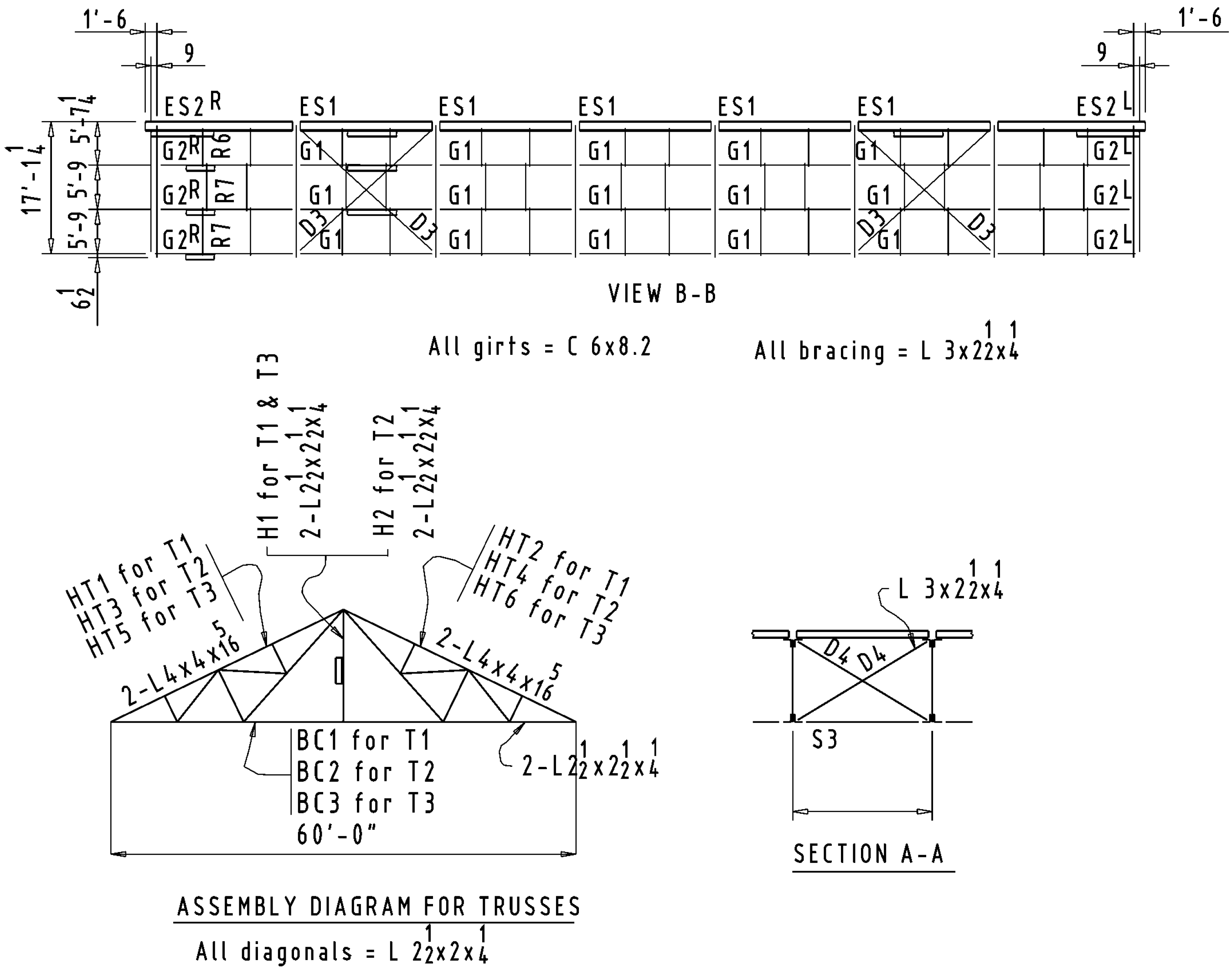


Figure 7-48a. Erection drawings for the roof framing of an industrial building.



Note to Erector :

Shipping marks are painted on the left end of the member as detailed. Erect member so mark is in position shown on plan.

Field connections: 3/4 Ø A325-N

Figure 7-48b. Bottom chord plan.

The length of main material is established by deducting the scaled distance from working points to each end of the member from the calculated distance between working points.

Layouts, whether a part of a detail drawing or on a separate layout sheet, must be drawn accurately and to a large enough scale so that the setback of the main material from the working point and the size of the gusset plates may be scaled from them. When the layout is on a separate drawing, a good scale is 1 1/2 in. = 1'-0. However, when the layout is part of the shop detail drawing, a scale larger than 1 in. = 1'-0 is usually impractical to use. Scales smaller than this should not be used if size of material is to be determined by scaling.

The scale used in laying out the working lines need not be the same as that selected for the details along these working lines. A scale of 1 in. = 1'-0 was used to fill in the details in Figures A7-49 and A7-51. The layout scales were 1 in. = 1'-0 and 1/2 in. = 1'-0, respectively.[3] The more complicated the layout, the larger the layout scale must be to provide sufficient room for all of the required dimensions, notes, marks and descriptions of assembly pieces.

Symmetry and Rotation

Fewer shop drawings will be required for detailing trusses and bracing when use is made of any symmetry in the framing. Because of symmetry, a shipping piece on one side of the building centerline may be exactly the same as another piece on the opposite side of this centerline rotated 180° in plan. In other cases such pieces may be opposite hand to one another. Refer to Chapter 4 for discussion of left/right and opposite-hand details.

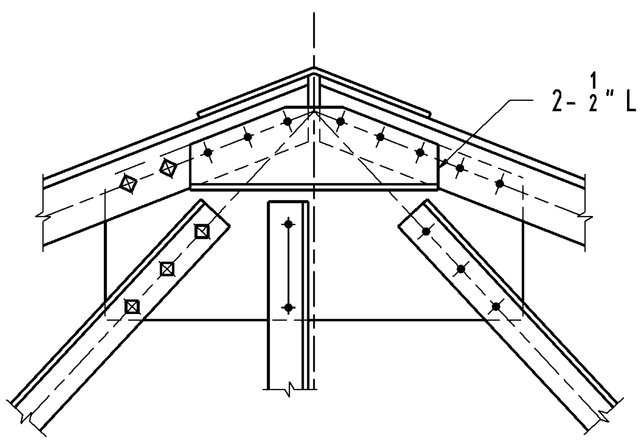


Figure 7-50. Stiffening angles at truss peak.

[3] All of the details shown in this textbook have been photographically reduced in scale from the original drawings. For this reason, the scales mentioned cannot be used to scale dimensions on the figures in this text.

The only differences between the left and right halves of truss 105T6, shown in Figure A7-45, occur at the centerline of the truss, which is also the building centerline. They are minor in nature and affect only the center vertical member $wk^{R/L}$ and purlin clip ha. When these differences are detailed completely or noted in the view of the left-hand half of the truss, a view of the right-hand half is not necessary. Except for small differences at the centerline, the details of the left-hand portion can serve as the details of the right-hand portion, thus cutting the required amount of drafting and shop layout work in almost half. When this method of detailing is used, the left half of the "as-shown" shipping piece is shown on the shop drawing.

A similar situation exists in the case of half-truss HT1 in Figure A7-49. Except for the differences noted on the shop drawing, the details for this truss could be rotated 180° about the truss centerline to be the supplementing half-truss HT2.

Because of the holes for connecting the bottom chord bracing near the column end of the inner bottom chord angle b^R, and the omission of these holes in angle c^R, the details for half-truss HT1 cannot be rotated 180° to serve as the details for half-truss HT2 (even if the differences between these half-trusses at the peak are neglected). In effect HT2 will be detailed opposite-hand to HT1 in order to keep these holes only on the inner angle at both ends of the truss.

Truss T1 must be rotated 180° during erection at the east end of the building to locate the holes for the bottom chord bracing toward the inside of the building. This is made clear on the erection drawing by locating the assembled truss mark on the opposite sides of the building centerline at opposite ends of the building (see Bottom Chord Plan, Figure 7-48b).

As drawn, the outstanding leg of the near angle *k* of hanger H2 is to be punched to provide connections for the sway frame members D4 and S3. Angle k will be in the correct position in the structure when hanger H2 is placed in the truss as shown on the Assembly Diagram for Trusses in Figure 7-48b.

When differences are so slight that a single detail can be made to serve for two or more shipping pieces, notes pointing out the differences should be written in a positive manner. For example, the note on the far side bottom chord angles (Figure A7-49) reads:

"Holes in HT2"

Do not use a negative statement such as:

"Omit holes here for HT1."

A note usually is written to refer to the affected piece itself, not to the affected assembly piece in the shipping piece. For example, a note should read:

"Cut on this line for BC1 and BC3"

Do not use a note reading:

"Cut on this line for bc and bf."

Dimensioning

Working dimensions (i.e., those dimensions appearing on the erection drawings, such as the distance center-to-center of columns) generally are repeated on the shop details. These working dimensions are placed conspicuously outside of all other lines of dimensions for ready reference in checking the details and for any subsequent study of the matching of adjoining shipping pieces.

Lines of dimensions locating intermediate panel points and other important reference points at or near the intersection of working lines are placed next to the working dimensions. When working points lie outside of the shipping piece, dimensions giving the distance along the working lines from the working points to reference points on the shipping piece are located prominently. Thus, for diagonal D4 in Figure A7-51 a second line of dimensions, located immediately inside the 18'-0 and 10'-0 working dimensions, locates reference points on the fittings. All of the detail dimensions can be laid out from these reference points.

Camber in Trusses

Camber is added in a truss to compensate for deflection when it is loaded by construction loads. Before calculating the distances between working points and bevels, the steel detailer must know how much camber, if any, is to be provided to compensate for the anticipated deflection of the truss under loading. This information must be given on the construction documents. Generally, industry practice recommends that trusses less than 80 ft in length are not cambered. In Figure A7-52, however, the designer has indicated a $1^1/_2$-in. camber, meaning that the mid-span elevation of the truss, before loading, is to be made $1^1/_2$ in. higher than corresponding points at the end of the truss. This rise from the ends to mid-span takes the form of a smooth, flat, approximately parabolic curve. Camber is assumed to be positive (arch upward) unless specifically noted otherwise.

For reasons of convenience and simplicity, usual practice is to detail trusses in the normal or flat (uncambered) position. However, the lengths of diagonal members and the bevels are computed and dimensioned for the cambered position.

In Figure 7-53a the working lines have been drawn for a portion of the truss shown in Figure A7-45, using a greatly exaggerated vertical scale for camber offsets. For the end panel, the adjusted length of end diagonal af and the bevel it makes with the horizontal may be calculated using the vertical distance between points f and a ($6'\text{-}1^1/_{16} - {}^{21}/_{32} = 6'\text{-}0^{13}/_{32}$) and the horizontal distance ef' ($7'\text{-}10^5/_8$). Converting to decimal equivalents, these dimensions are 6.034 ft and 7.885 ft. The square root of the sum of the squares of each of these numbers yields 9.929 ft or $9'\text{-}11^1/_8$.

The original length af′, when computed, was found to be $^7/_{16}$-in. longer than the adjusted length af. Note, however, that no significant difference remains in the bevel before and after the camber adjustment.

Had the end diagonal been reversed, as in Figure 7-53b, making it a compression member instead of a tension member, the vertical distance bf′ ($6'\text{-}5^1/_{16}$ plus the offset) would be used in calculating the adjusted length. In this case the adjusted length is longer than the original length.

To obtain the adjusted length of tension diagonal bg (Figure 7-53a), a vertical distance of

$$6'\text{-}5^1/_{16} - (1^1/_8 - {}^{21}/_{32}) = 6'\text{-}4^{19}/_{32}$$

would be used in the calculations.

On the other hand had the truss been made with compression diagonals as illustrated in Figure 7-53b, the vertical distance for compression diagonal fc would be

$$6'\text{-}9^1/_{16} + (1^1/_8 - {}^{21}/_{32}) = 6'\text{-}9^{19}/_{32}$$

as the adjusted lengths are shown on the shop details. A note to that effect should appear on the shop drawing. Such a note might read:

"Camber has been figured in truss"

Some fabricators have the shop adjust the lengths to obtain the required camber. In this event a note should be placed on the shop detail drawing under the camber diagram reading:

"Lengths of members to be adjusted by shop"

Building members (girts, purlins, etc.) attaching to cambered trusses should be detailed assuming the trusses are in their normal (uncambered) position. At the ends of buildings vertical members comprising the framing (i.e., wind columns) should have connections to trusses made with vertically slotted holes whether or not the trusses are cambered to account for deflection of the truss when loaded. The slotted holes must be in the connection material.

Where permitted by design, Fink or steep-pitch trusses may be cambered simply by raising the bottom chord in the center-section of the truss the required amount and permitting the chord to slope to zero at the ends.

The camber diagram should always be included with the truss shop details so that the fitter and inspector can check the camber as the truss is assembled.

Bottom Chord Connection to Column

When the truss is erected, the tension chord lengthens under load and tends to encroach on its end connections. How to deal with this situation and to calculate the change in length is addressed in the *Manual*, Part 13 under the section titled "Shop and Field Practices." For the truss shown in Figure A7-45 the increase in length of the bottom (tension) chord was found to be $^{3}/_{16}$ in. on either side of the centerline. To compensate for this, provide holes in the bottom chord and slots in the column details matching the holes in the chord flange. The proper time during erection of the structure to tighten the bolts should be determined by the owner's designated representative for design.

Stitch Fasteners and Welded Fills

Sections D4 and E6 of the AISC *Specification* cover the stitching of double-angle members to ensure that they have the stiffness of a combined section. Note that a minimum of two stitch-fillers is required. When not specified, fasteners may be either bolts (A307 or A325) or welded fills, depending on fabricator preference. Stitch fasteners and welded fills should be shown in the contract documents.

If the pair of angles must be separated to straddle a gusset plate, a round or rectangular washer is furnished with each stitch bolt in bolted construction. These maintain the desired spacing between the angles and provide a solid abutment against which to tighten the bolt. In welded construction a fill is used, consisting of a small rectangular piece attached by welds. A fill with protruding ends, as shown in Figure 7-54a, is preferred because it is easy to fit and weld. The short fill shown in Figure 7-54b is used if a smooth appearance is desired.

If the pair of angles is to be shipped as a unit (e.g., as in hanger H1 in Figure A7-49) and is held together only by stitch fasteners or welded fills, a minimum of two fasteners or welded fills should be used. If the member has holes for field connections at each end, a stitch fastener should be placed not more than 2 ft from each end in order to hold the component parts in proper alignment to make the field connections, as shown in Figure A7-49 for hanger H1 and bottom chord BC1.

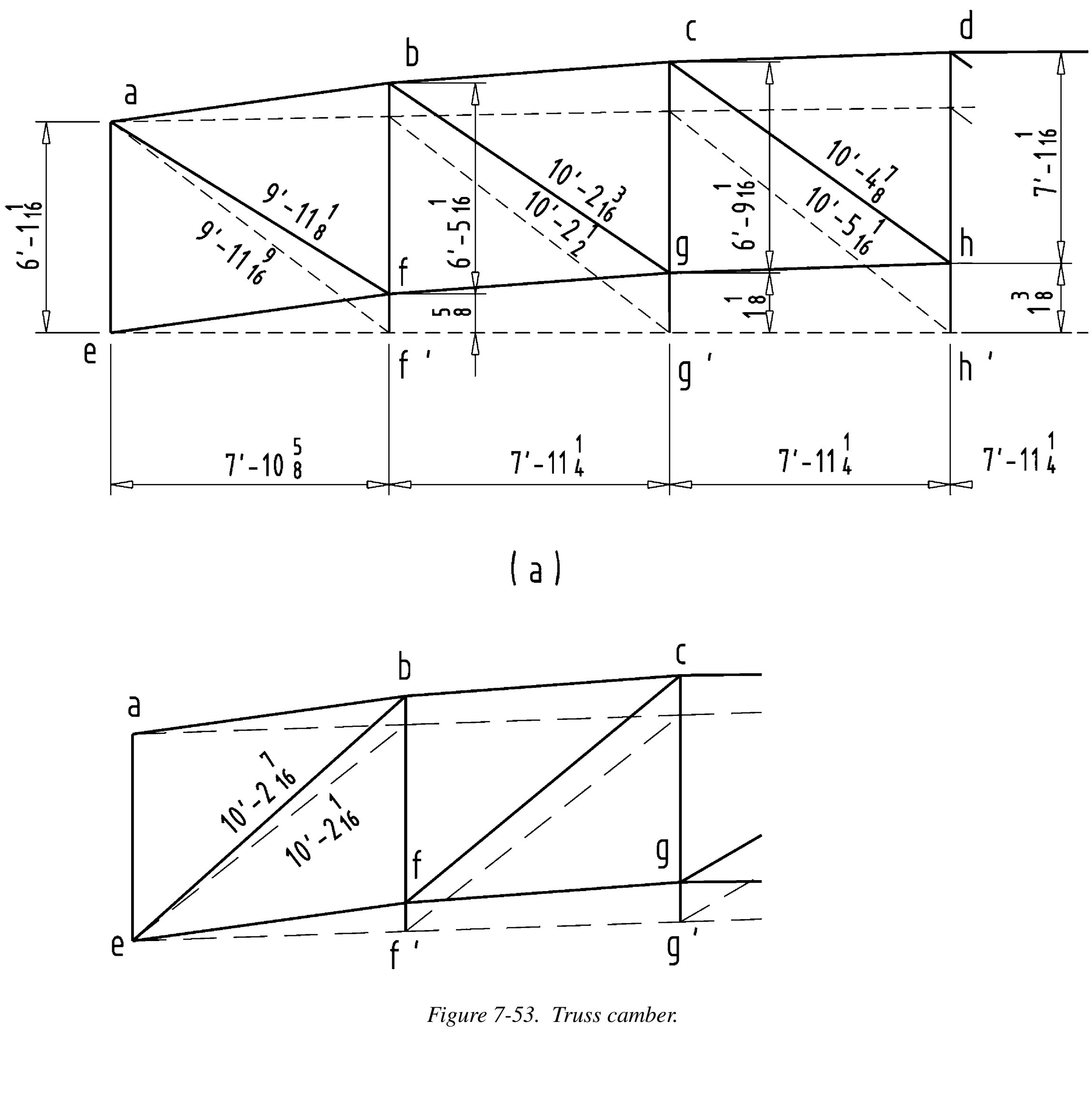


Figure 7-53. Truss camber.

As welded fills do not require holes in the angles, dimensioning the spacing of such fills on detail drawings is usually unnecessary. The shop understands that the fills indicated must be spaced equally between end connections when no dimension is given.

Continuous fillers often are used in a corrosive environment. The bolts or welds are spaced to seal the edges.

BRACING SYSTEMS

Shop-Welded – Field-Bolted Construction

Refer to Figure A7-52 and note that this structure is provided with two principal bracing systems. The main truss bracing system consists of bottom lateral bracing, shown in the bottom chord plan, and three lines of sway frames—one at each end of the trusses (between the columns) and one at the center of the building between the trusses. The arrangement and material of these sway frames are shown in Figure A7-52 as part of the sidewall elevation. The second bracing system, also shown in the sidewall elevation, consists of vertical bracing and a horizontal strut between the lower column shafts. This bracing extends from the bottom of the crane runway girders to the floor. Both systems stiffen the building against lateral and horizontal crane forces and provide the path to carry both crane and wind loads to the ground.

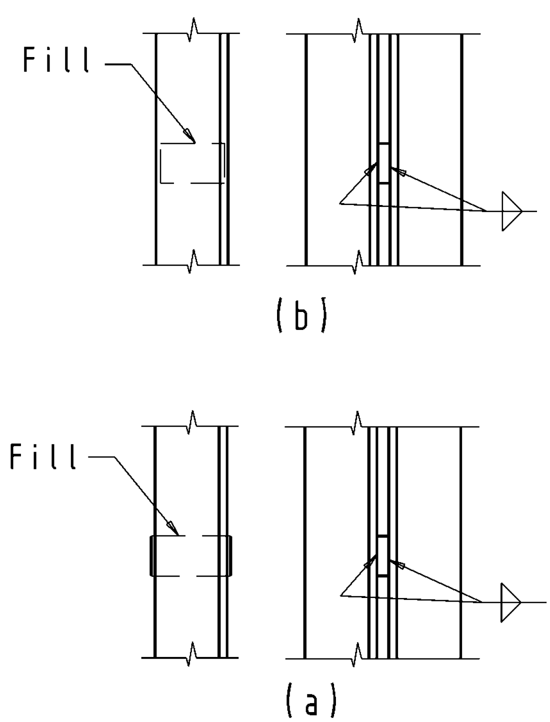


Figure 7-54. Fills.

Truss Bracing

The layout and detailing of lateral bracing and sway frames is interdependent, but the connections of all the members entering a joint will need to be developed at more or less the same time. A convenient starting point is the connection of the bottom lateral bracing and the end sway frame to the column shaft. The plan view in Figure 7-55 shows a scaled layout of the bottom lateral gusset plate connecting the diagonal brace to the bottom chord of the sway frame at the column line. One component of the force in the diagonal brace passes through the lateral plate to the sway frame. As this lateral plate does not lend itself to shop attachment to any of the members entering the joint, it will be detailed and marked for separate or "loose" shipment.

The work line of this diagonal brace is laid out in this plan view from the centerline of the column, using the bevel it makes with the truss centerline. Calculations for this bevel and for the length of the diagonal are as follows (see Figure 7-56):

Out-to-out of columns (Figure A7-52)	= 64′-5
Depth of two half-columns (W12×50)	= 1′-0$^1/_4$
Subtracting	63′-4$^3/_4$
Half-distance (c. to c. upper shafts)	= (63′-4$^3/_4$)/2
	= 31′-8$^3/_8$
Length of bay	= 26′-0
By the square root of the sum of the squares, the diagonal is calculated as	= 40′-11$^{15}/_{16}$

Field fasteners for the bracing members are proportioned according to the general notes on Figure A7-52, which call for three bolts in single angles and two bolts in each double-angle member.

The bottom chord sway frame connection to the column web is shown in Figure 7-55 to be a tee cut from a W16×57. However, the detail cannot be completed until the bevel of the sway frame diagonal has been calculated. The location of work points and establishment of distances necessary to calculate this bevel are given in Figures 7-57 and 7-58. Information that will be used to prepare shop drawings for the trusses and columns is also developed in Figure 7-57.

Each center intersection of the bottom lateral system is supported by a W6×16 beam which spans from truss to truss (see Figure A7-52). Because the vertical leg of all the bracing angles must point upward, so as not to encroach on the crane clearance, each diagonal must be interrupted and attached to a gusset where it crosses the W6×16 (see Figure 7-59).

The center sway frame, shown detailed in Figure 7-60, is field bolted to the truss at the top and bottom chords and to

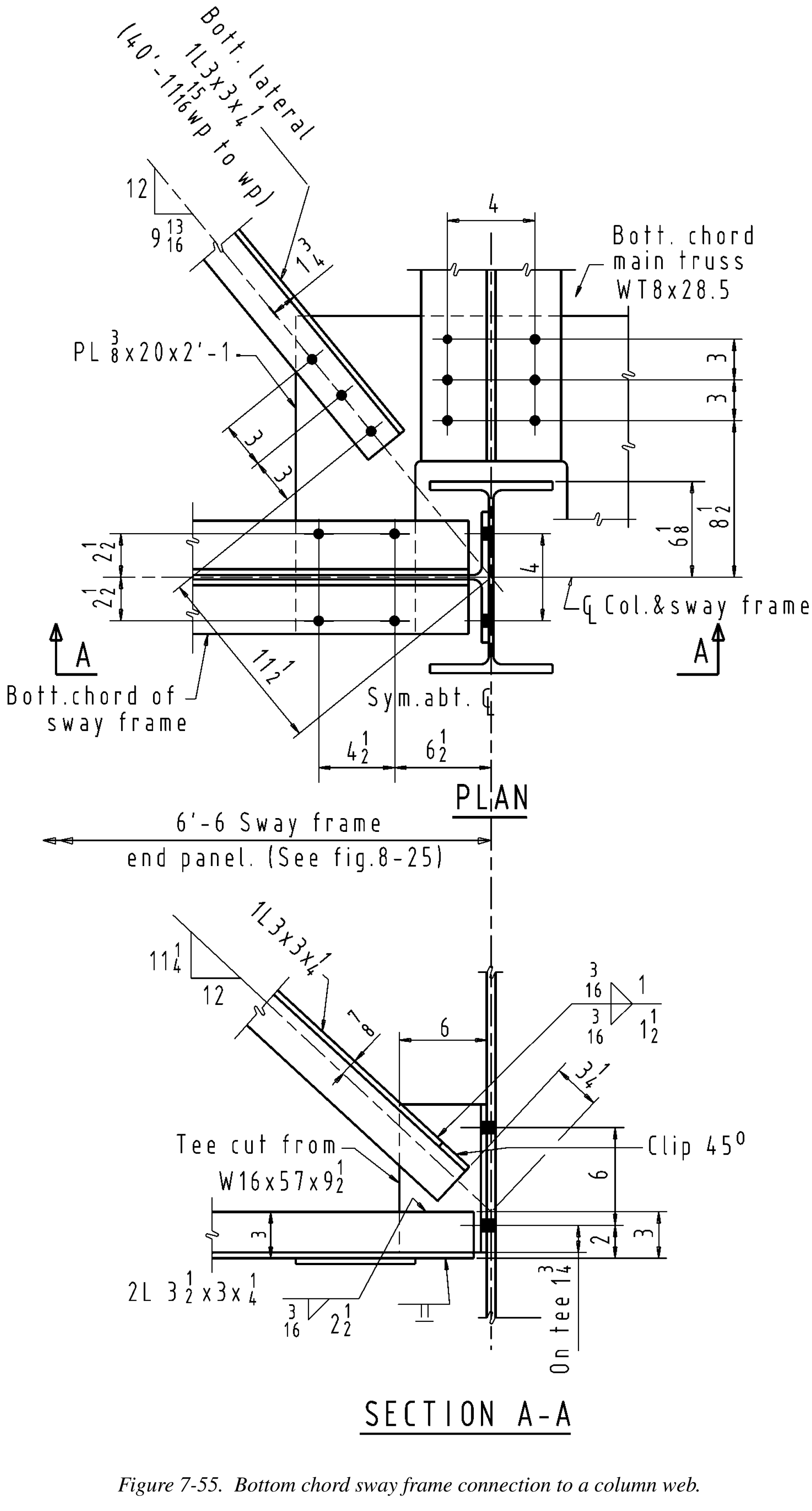


Figure 7-55. Bottom chord sway frame connection to a column web.

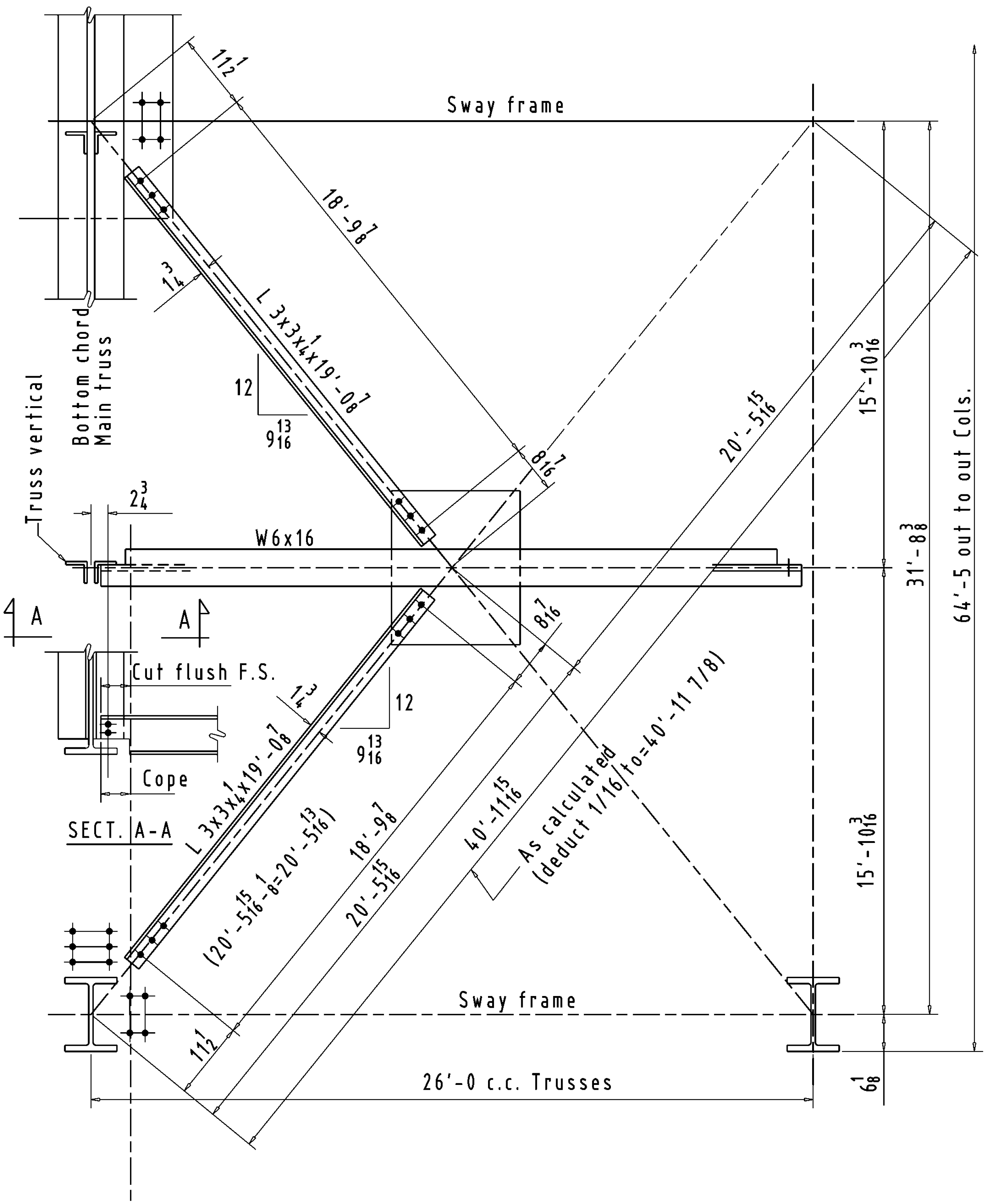


Figure 7-56. Diagram showing the relationship of all parts of the bottom lateral bracing system.

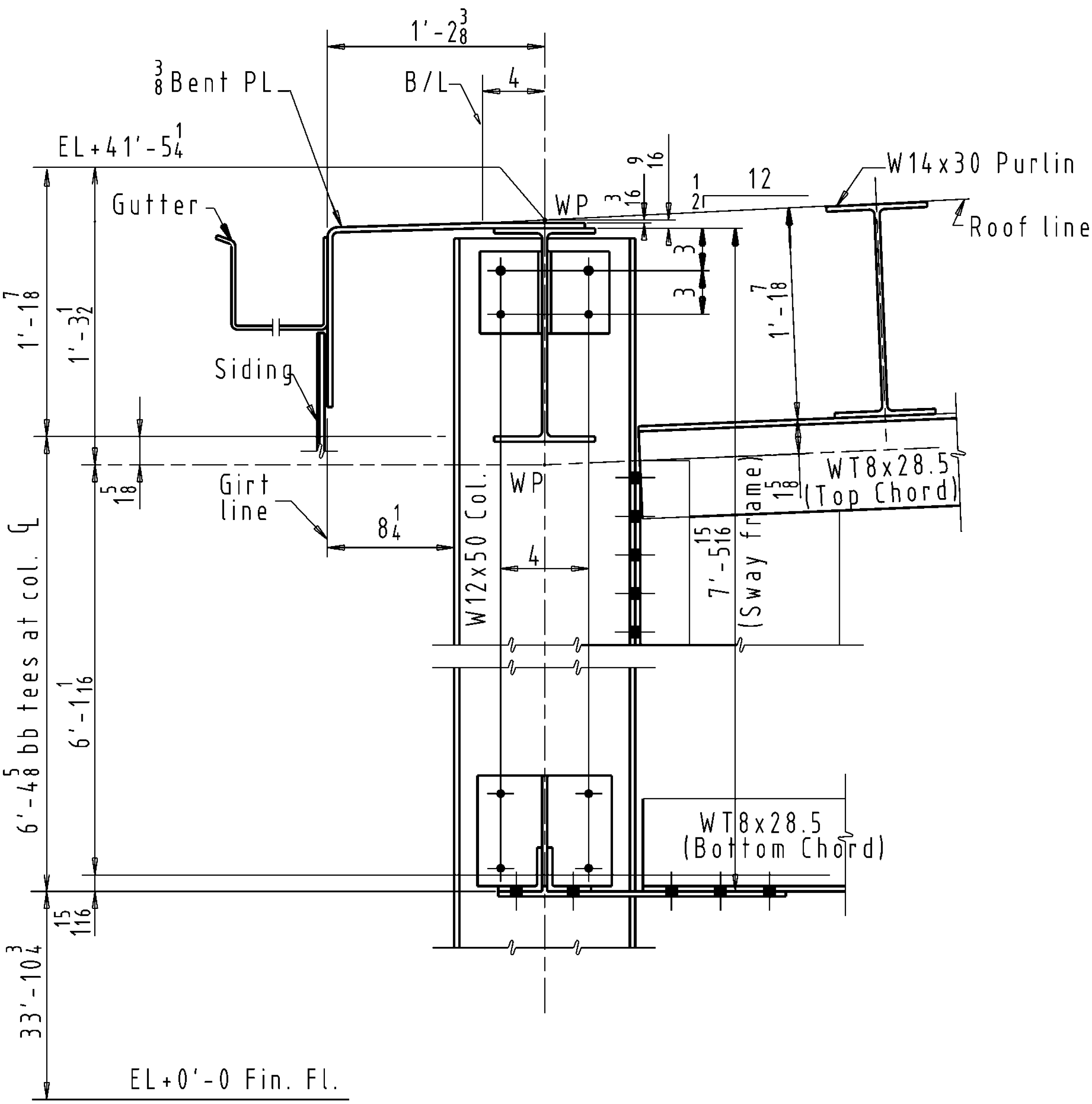


Figure 7-57. Information that will be used to prepare shop drawings for the trusses and columns.

the bottom lateral gusset on the truss (see Figure A7-45). The bottom lateral diagonal is attached to the gusset also.

A diagram showing the relationship of all parts of the bottom lateral bracing system is shown in Figure 7-56.

Pretension (Draw) in Tension Bracing

Long flexible tension bracing, if furnished in exact theoretical length, may sag because of fabrication and erection tolerances or temperature differential. When this is the case, considerable and objectionable movement of the frame may take place before the bracing becomes effective. To avoid this, such bracing generally is fabricated to a slightly shorter length. The calculated or "theoretical" distance center-to-center of end working points is reduced for fabricating purposes according to the following schedule:

- For lengths up to 10 ft, no deduction
- For lengths over 10 ft to 20 ft, deduct 1/16 in.
- For lengths over 20 ft to 35 ft, deduct 1/8 in.
- For lengths over 35 ft, deduct 3/16 in.

To locate a hole or work point properly at the intersection of diagonal cross-bracing without the use of dimensions in thirty-seconds of an inch, an additional 1/16 in. may be de-

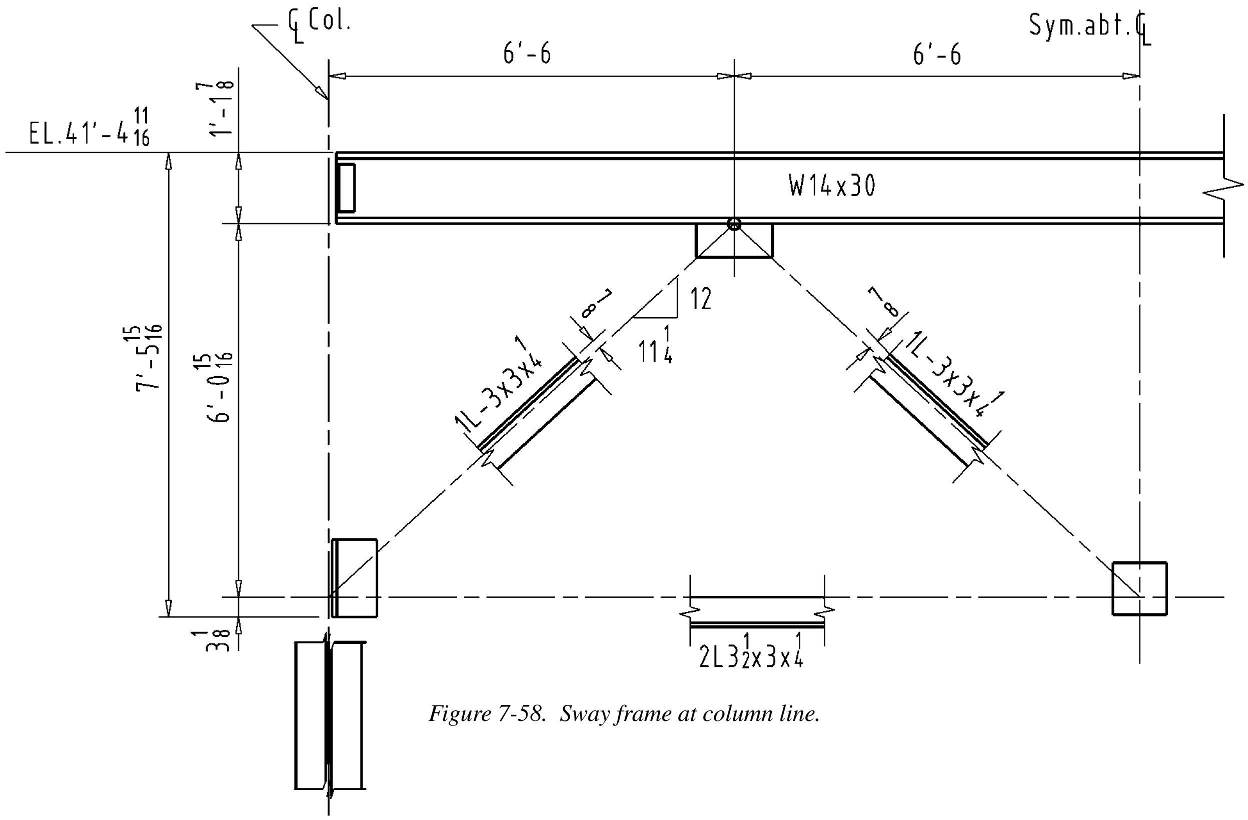


Figure 7-58. Sway frame at column line.

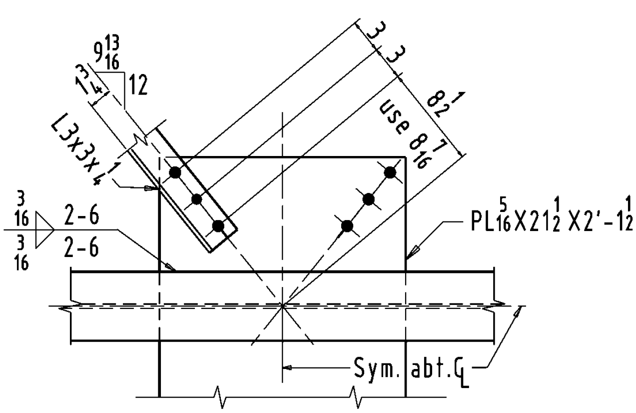


Figure 7-59. Diagonal attached to a gusset where it crosses the W-shape.

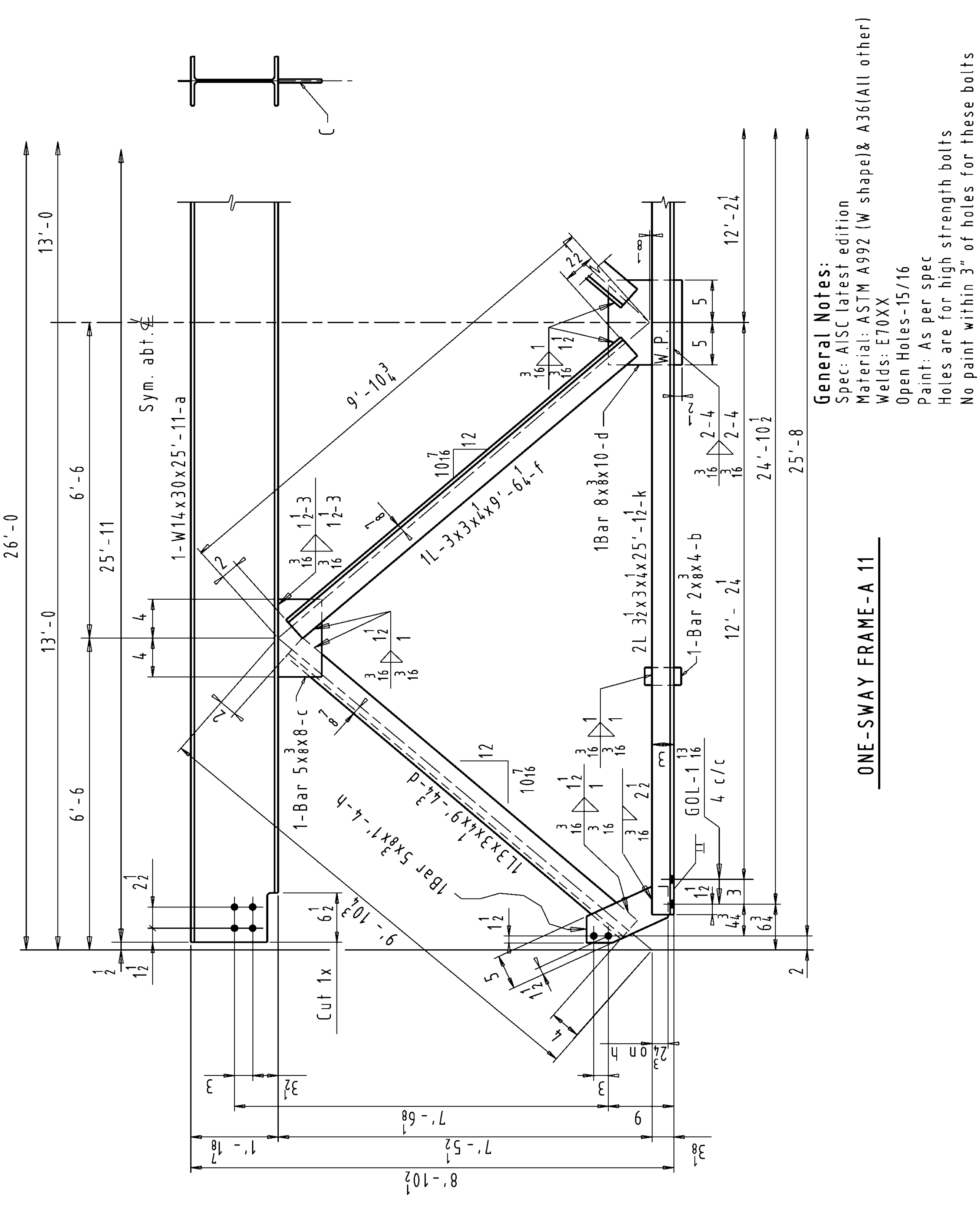


Figure 7-60. Detailed center sway frame.

ducted from the overall dimension where necessary. For an example of the application of draw in diagonals, refer to Figure 7-56. The overall diagonal length, computed previously, is 40'-11$^{15}/_{16}$. In order to eliminate thirty-seconds of an inch from the half-length, deduct $^{1}/_{16}$ in. from this dimension, which becomes 40'-11$^{7}/_{8}$. Then, the dimension, work point to work point for each angle, will be 20'-5$^{15}/_{16}$, a length that calls for a reduction of $^{1}/_{8}$ in. to become the "as-fabricated" dimension, 20'-5$^{13}/_{16}$. Deducting the two gusset plate set-back dimensions, 11$^{1}/_{2}$ in. and 8$^{7}/_{16}$ in., from 20'-5$^{13}/_{16}$ results in a center-to-center of end holes of 18'-9$^{7}/_{8}$. Note that the 8$^{7}/_{16}$ in. dimension, originally scaled at 8$^{1}/_{2}$ in. (see Figure 7-59), was changed to 8$^{7}/_{16}$ to eliminate sixteenths of an inch from the angle punching.

When the piece is erected, the connection at one end is fully bolted to prevent slippage. Tapered drift pins are then used to line up the connection holes at the other end, after which bolting is completed. This tends to stretch or draw the angle so that a slight pretension is present. Although the deductions for draw given in the schedule have proved satisfactory for general work, the steel detailer should be guided by the design drawings or by the fabricator or erector's standard requirements.

The preceding draw allowances are not applicable to tension diagonals fabricated from shapes other than light angle shapes. The force required to draw heavier members will cause distortion of the holes, usually in the gusset plate, before the member will stretch the indicated amount.

Vertical Bracing

The vertical bracing panel shown in the wall elevation of Figure A7-52 (with an indication of the desired connections in the Typical Column Detail) is developed further in the dimensioned layout of Figure 7-61. Calculations for bevels, diagonal lengths and pretension follow the same steps taken for the horizontal bracing previously discussed. In this and adjacent panels the W18×35 horizontal strut serves to brace the columns against buckling about their Y-Y axis. Figure 7-62b shows a partial detail of this strut with the bracing gusset attached.

A suitable connection of the strut to this column is shown in Figure 7-62a. Note that a $^{1}/_{4}$-in. erection clearance has been provided to allow for variation in the 17$^{3}/_{4}$-in. depth of the W18 beam (see "Standard Mill Practice" in the *Manual*, Part 1). Details of the strut and bracing connections to the column shaft are shown in Figures 7-62 and A7-10.

Double-Angle Bracing

A simple case of double-angle cross-bracing is shown in Figure 7-63. Calculations required for detailing this bracing are the same as for the single-angle bracing previously discussed, except that the inherent stability of the paired angles renders deductions for draw unnecessary. Note that no bevels are shown on this detail. Although bevels must be computed to develop the end gussets (not shown), locating the punching in the center gusset by only using dimensions is possible.

Double angles separated by gussets must be connected by stitch fasteners if their unsupported lengths exceed the maximums permitted by AISC *Specification* Sections D4 and E6, respectively, for tension and compression members.

Two types of intermittent fills for use in welded work are illustrated in Figure 7-54. The double-angle (tension) bracing panel shown in Figure 7-63 illustrates a bolted type that generally is preferred when the angles must be punched and that is spaced to meet the requirements of tension members. Note that the spacing of all holes is given on the shop drawings and that a minimum of two stitch-fillers are required.

Actually, the case illustrated in Figure 7-63 does not require intermittent fills for the purpose of buckling resistance. Fills are needed to provide the separation required between angles for erection. Shipping pieces B1 and B3 have two fills to prevent "windmilling" of the individual angles during handling and shipping. In shipping piece B2, only one fill is required as gusset plate pa is permanently bolted to the shipping piece and, thus, can be considered as a fill.

Because welded fills do not require holes in the angles, dimensioning their spacing on shop detail drawings frequently is unnecessary. The shop understands that the fills must be spaced equally between the end connections when no dimensions are given on the shop drawings.

Knee Brace Connections

Knee braces generally appear in conjunction with peaked roof trusses that have very little depth at their ends and, therefore, can furnish no significant lateral stiffness to the bent. A bent is a vertical frame used to support other members. The bent shown in the Cross Section of Figure A7-66 is a braced bent. Nonbraced bents consist of columns connected to beams or girders using fully restrained (type FR) moment connections.

The loading on a bent is from lateral forces, which cause reversible forces on the connections depending on the direction of load application. The knee brace detail is shown in Figure 7-64. A similar plate connection is used to connect the knee brace to the bottom chord of the truss (Figure A7-49). Erectors prefer that the knee brace connection material be shop attached to the column and truss and that the brace angles be field bolted to the gusset plates.

Shop-Welded – Field-Welded Construction

Although this construction is not used widely in industrial work, furnishing bracing with connections prepared for field welding may be shown on the construction documents. As bracing is used frequently to square and align the structure

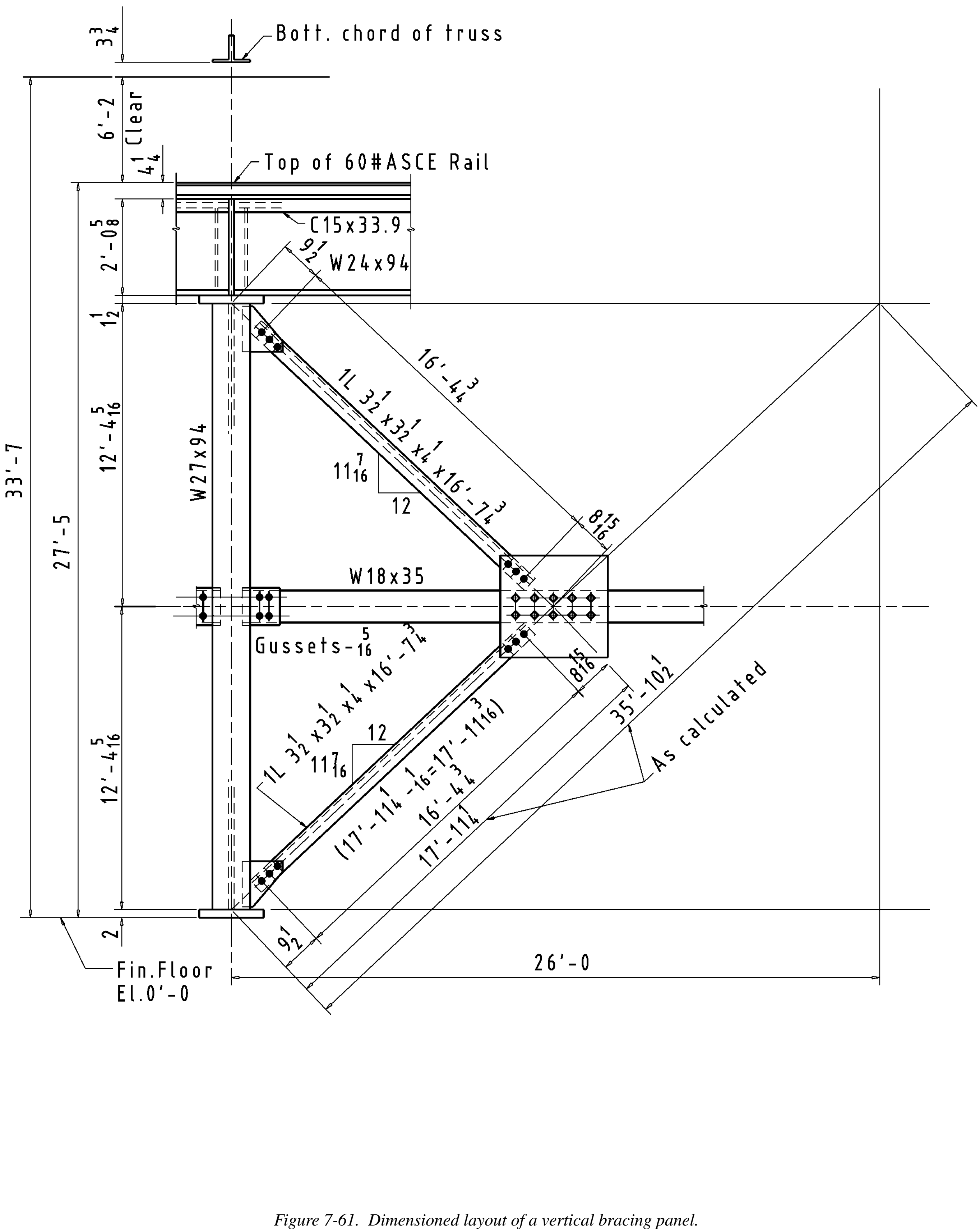


Figure 7-61. Dimensioned layout of a vertical bracing panel.

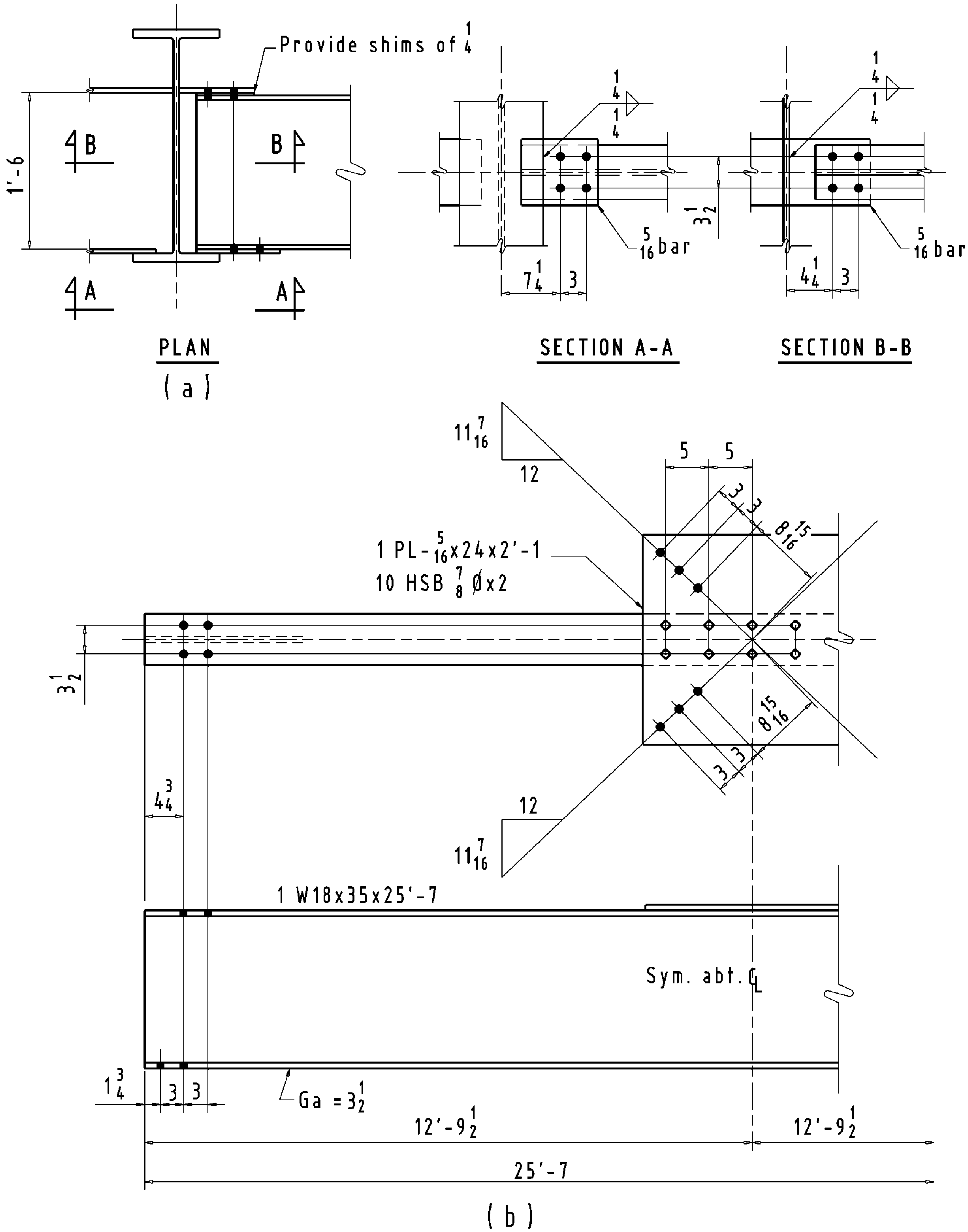


Figure 7-62. Details of the strut and bracing connections to the column shaft.

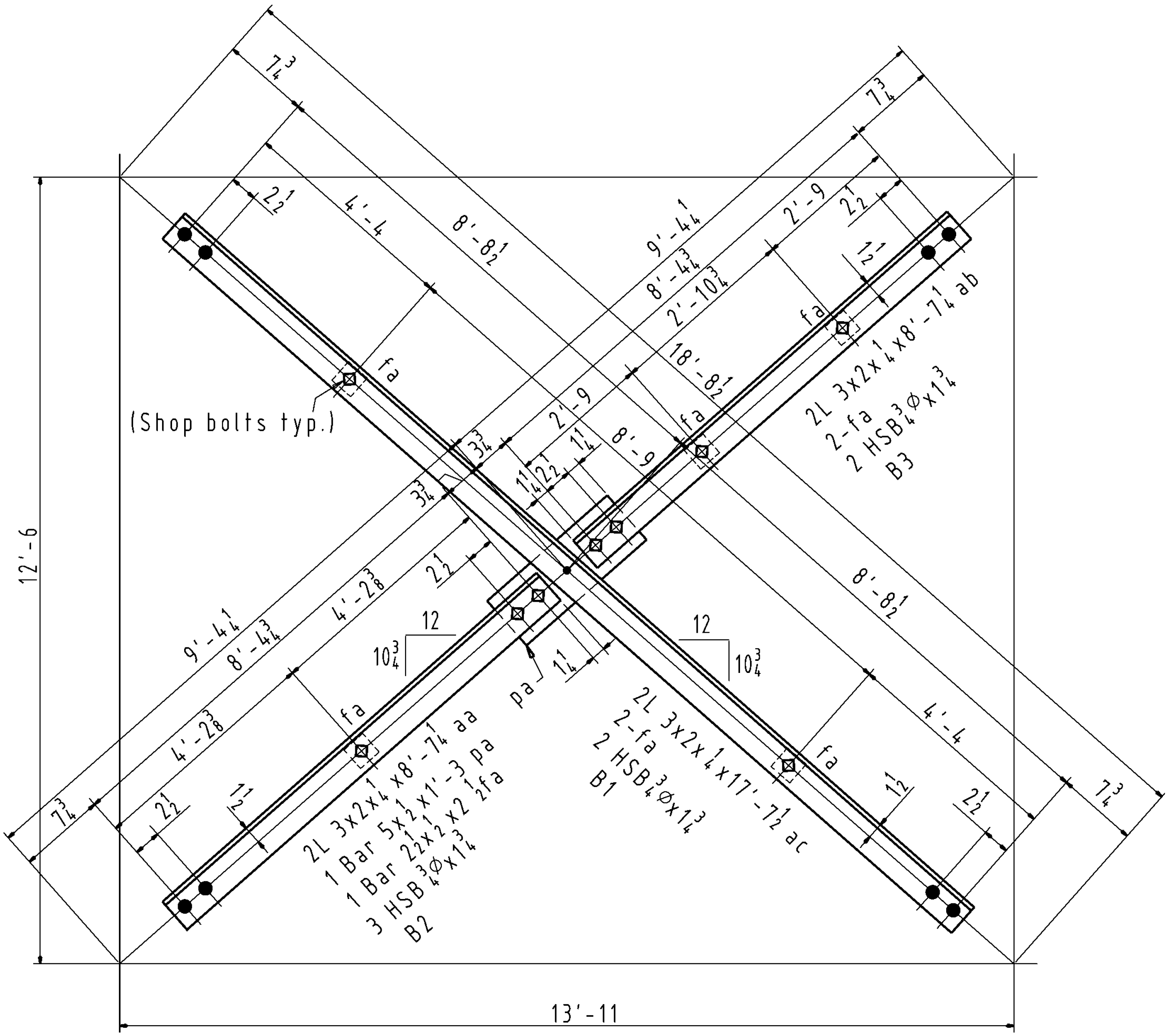


Figure 7-63. Double-angle cross-bracing.

during erection, provision should be made for at least one fitting bolt at each connection. Calculations and details for this kind of bracing will differ from the field bolted type only in the reduced number of holes required. If only the hole at the end of each angle of the bracing panel shown in Figure 7-63 were punched/drilled, the general appearance would be that of field-welded construction. In addition to providing adequate space for weld lengths required, the steel detailer must make sure that the weld areas are accessible to the welder. Although some field welding is done on the ground, as in the preassembly of truss sections, most of it must be accomplished from staging under adverse and frequently hazardous conditions. Visualizing the joint with all members in place is often helpful in avoiding impossible welding situations.

In the event that furnishing field-welded bracing with no holes for erection bolts is necessary, shop details are simplified, but diagrams on erection drawings will be required. Figure 7-65 shows the field assembly diagram for such a bracing panel. Note that dimensions for locating the ends of the angles are given to reference edges readily accessible to the erector. The angles will be held in place during welding by C-clamps. If the slight bow in the center section is considered objectionable, a welded fill can be inserted at this point.

Shop-Bolted – Field-Bolted Construction

The examples of shop-bolted – field-bolted construction shown in Figure A7-51 are based on the design shown on Figure A7-66. These are complete details, including shop assembly and shipping marks. The location of each piece in the structure may be found on the erection drawing in Figures 7-48a and 7-48b. The details of trusses to which this bracing attaches are shown in Figure A7-49.

The details on the left-hand side of Figure A7-51 show all the bracing indicated in the bottom chord plan of Figure 7-48a, except struts S4, S5 and S6. Note that these details are arranged relative to one another, just as they will be when installed in the frame. The advantages of this method are (1) the intersection points and the holes common to two or more shipping pieces can be located with one set of dimensions, and (2) the correctness of all dimensions relating to the matching of the field connections can be checked more easily and with less possibility of error.

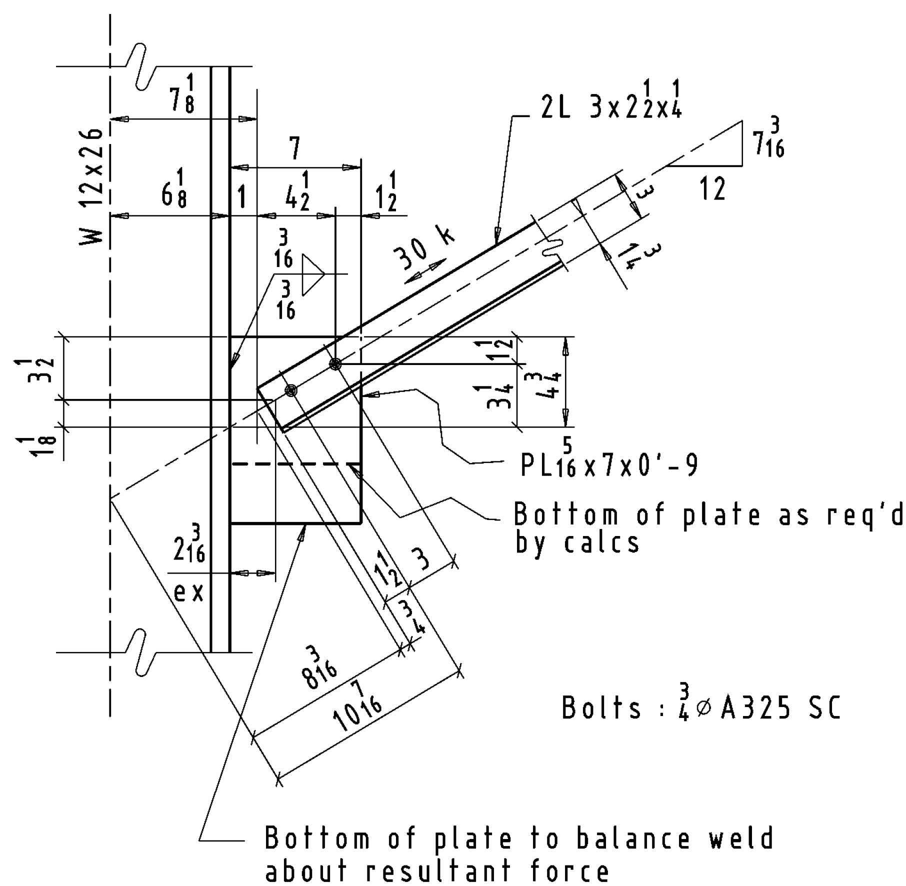


Figure 7-64. Knee brace detail.

Note that the gussets to which diagonal D1 and struts S1 and S2 will attach in the field are detailed with the trusses (see plates bc, bd and bf in Figure A7-49) instead of with the struts, as is the case with plates h and k on strut S3. Although some fabricators might have preferred to include all gussets with the bracing, a study of erection problems indicated that plates bc, bd and bf would serve a better purpose if used on the trusses to stiffen the splice point of the bottom chord during erection. Earlier in this chapter mention is made of the problems and hazards sometimes encountered in the assembling and handling of trusses. Because struts S3, S4, S5 and S6 are on the centerline of the building and not at a truss splice point, the transfer of their gussets to the trusses would serve no purpose.

Diagonals D3 constitute the vertical bracing in the side walls and attach to the columns. D4 is the sway frame diagonal which, with bottom strut S3 and ridge purlins RS1, makes up intermittent braced bays at the truss centers.

Bracing in the plane of the top chord of the trusses consists of $^3/_4$-in.-diameter rods, R1 and R2, shown in the upper right-hand corner of Figure A7-51. Note that this detail is shown as a line diagram and that no attempt is made to

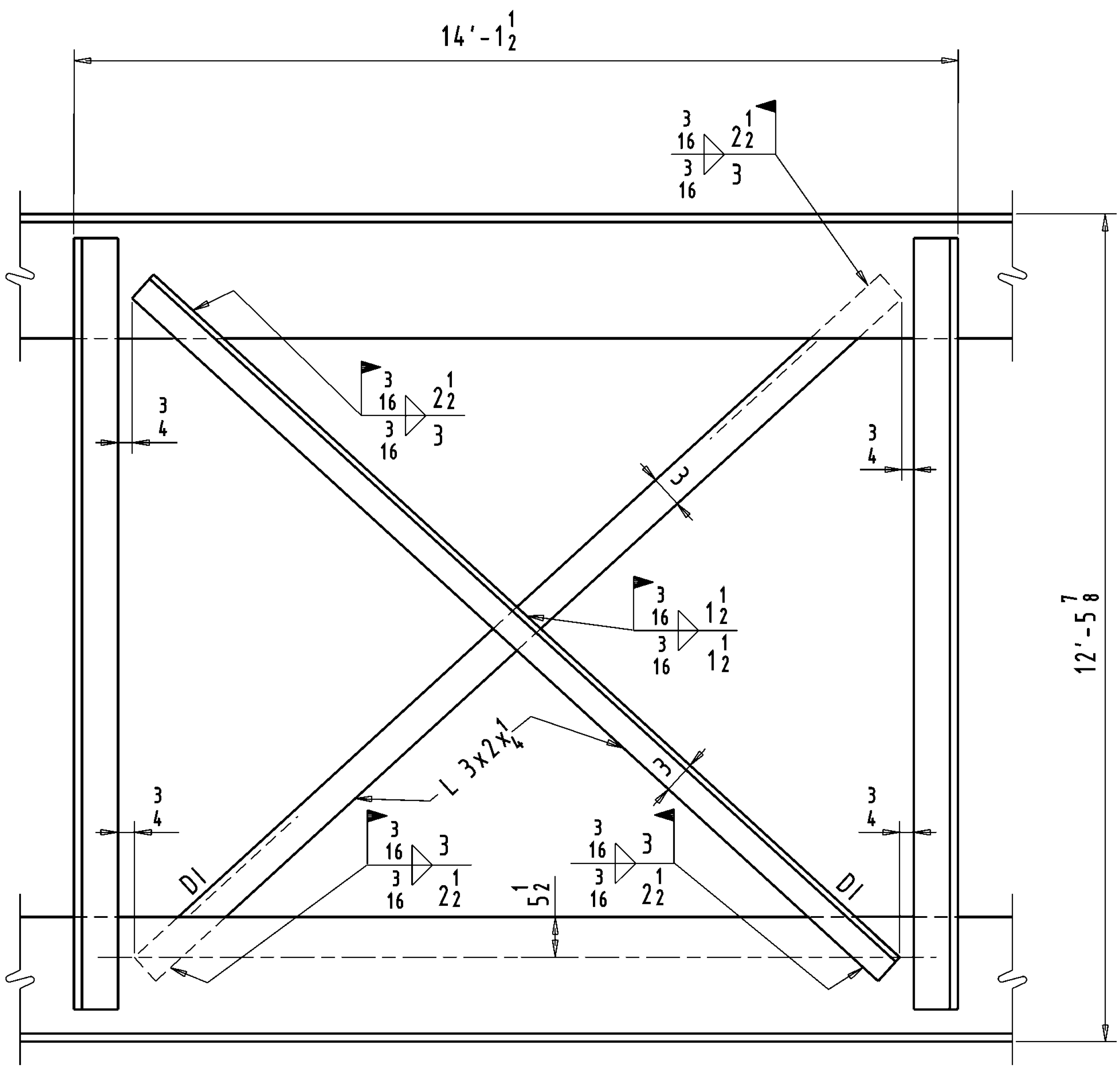


Figure 7-65. Field assembly diagram for a bracing panel.

show thickness of material, thread symbols or the shape of the nuts and turnbuckles. The standard nut called for will be the same used with A307 bolts. Dimensions of the turnbuckles are standard for a given diameter of rod and may be found in the *Manual*, Table 15-5.

Calculations of bevels, lengths between working points, pretension in diagonals and spacing of stitch fasteners are the same as discussed earlier in this chapter for welded construction. Note that the stitch fasteners in the struts employ washers as intermittent fills and that these are spaced to meet the requirements of compression members. The stitching of members S1 and S2 serves a double purpose, as the fasteners not only provide the attachment required by AISC *Specification*, but also makes possible the shop assembling of a pair of angles, otherwise loose, as a single shipping piece.

The detail of struts S4 and S5 shows the method used in making combinations to avoid repeating details. Angle s is used on both struts, but plates t are assembled only to strut S4. Observe that the notes accomplishing this are positive, not negative, in form.

The extent to which cross-bracing members can be fabricated alike, by rotating them 180° about their axes of symmetry for erection, may be seen from a study of Figures 7-48a and A7-51.

SKEWED, SLOPED AND CANTED FRAMING

Often the steel detailer will be required to detail connections for beams that are not level and do not connect to their supports at right angles. Such a beam may be inclined to a supporting member in various directions. Depending upon the relative angular position a beam assumes, the connection may be classified as skewed, sloped or canted. The steel detailer is referred to the *Manual*, Part 10 for a thorough discussion of these types of connections, the manners in which they are presented on shop drawings and the generally accepted practices for fabricating them.

BUILT-UP FRAMING

Most of the study of shop details thus far has been related to beam and column framing, trusses and bracing using rolled structural members. For buildings that require large clear spans and heavy loadings, rolled beam shapes may not be adequate to meet design requirements. In such cases built-up structural members fabricated from plates and shapes are used.

Crane Runway Girders

The shop details for the crane runway girders called for in Figure A7-52 are shown in Figure 7-67. The design drawing shows the type of girder-to-column connection, the size of connection material, the number of fasteners and tie-backs, and other pertinent information such as the need for end stiffeners.

On the construction documents, the owner's designated representative for design specifies the material to be used for the several components of the girder and the sizes of welds or bolts to attach them. In this case, the shop has the choice of using E70XX (manual shielded metal arc) and F7XX-EXXX (submerged arc) welding for the girder. Other shops may prefer to use ER70S-X (gas metal arc) or E7XT-X (flux cored arc) welding. Shops equipped with submerged arc welders probably would realize economy in the automatic or semi-automatic welding of the long runs required to attach the channel to the beam.

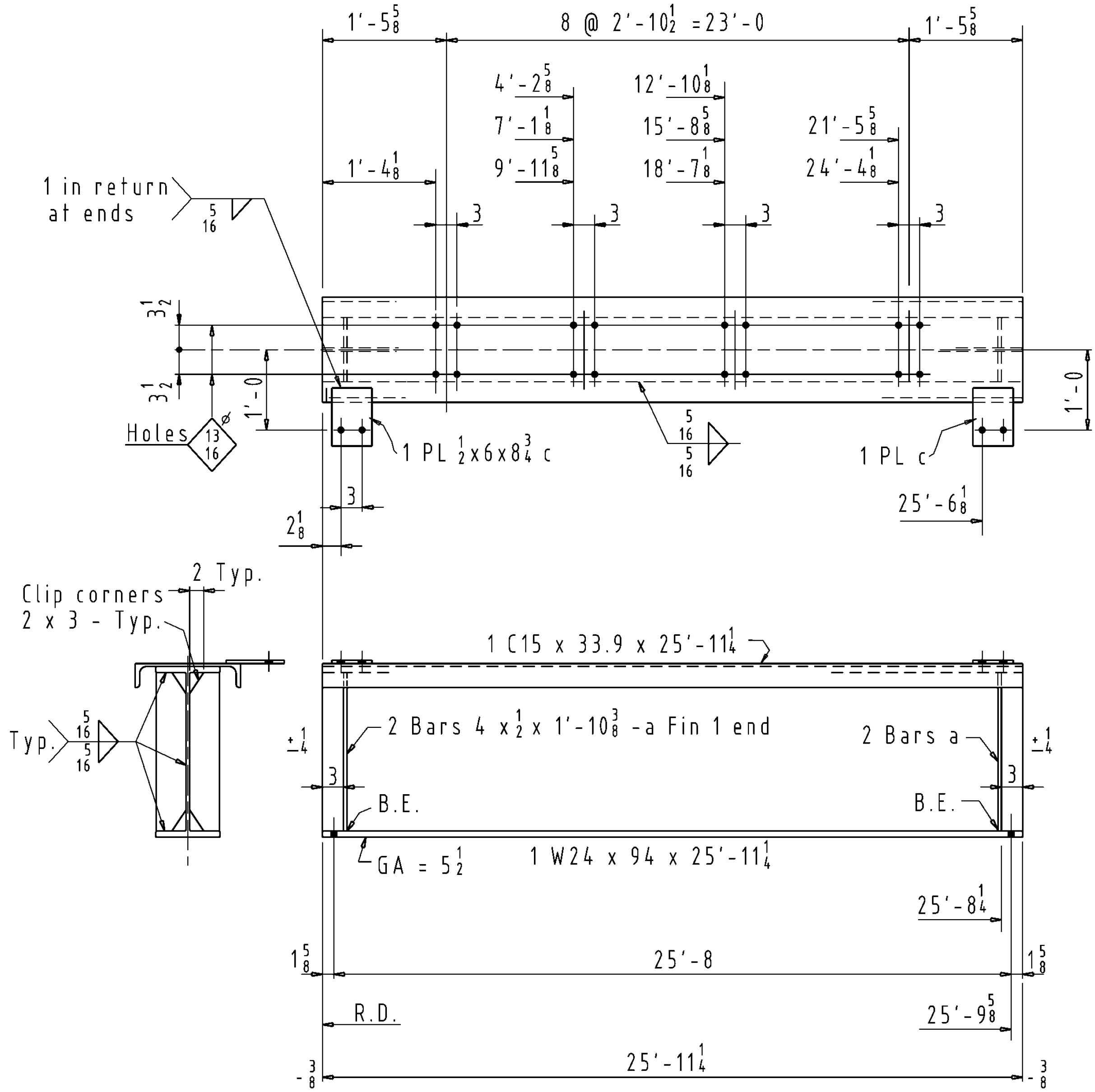
Figures 7-68a and 7-68b show two types of crane runway girders. Figure 7-68a shows a girder comprised of a W-shape with a channel welded to the top flange. Figure 7-68b illustrates the case when heavy crane loads are to be supported. Such cranes frequently require the strength of a built-up runway girder, reinforced at the top flange by a horizontal girder.

The crane runway girders of Figure A7-69 show the economy of design and detail usually realized when shop welded construction is employed for built-up girders. Note that the flange and stiffener angles necessary for shop bolted work have been replaced by plates and bars with a resultant savings in material and fabrication costs. The $1^1/_{16}$-in. holes in the top flange are for bolted crane rail clips. Had the designer called for welded studs or had field-welded clamps been used, these holes would not be required. Other holes in the top flange are for temporary erection bolts used to hold a horizontal girder in place during field welding.

The details of two different girders are shown in Figure A7-69. Girder 103CG2 has special end details to provide seats for adjacent girders which happen to be shorter and shallower. Note the use of line X-X in this combination of sketches. Without this device and the explanatory note, detailing girder 103CG2 at full length would have been necessary.

Note that $^1/_4$-in. fillet welds are used in connecting the $^3/_4$-in.-thick bottom flange plate to the $^1/_2$-in.-thick web plate. This weld size, which was determined by the owner's designated representative for design ($^3/_{16}$ in. is the minimum permitted by AISC *Specification* Section J2.2b), develops the force between the bottom flange plate and the web plate. The weld must be continuous even though intermittent welds might have adequate available strength.

For a built-up crane runway girder, the joint between the top flange and web must be capable of transferring wheel load concentrations from the crane wheels. In Figure A7-69, Note "B" stipulates that the web and flange must be in tight contact before welding. This is accomplished by preparing the edge of the web with a mechanically guided torch to provide the smooth straight edge necessary for continuous, tight contact. If tight contact does not exist, the flange-to-web welds must be designed to transfer concentrated loads from the crane



CRANE GIRDER CG1

General Notes :
Spec: AISC - Latest edition
Mat'l : ASTM A 992(W shapes) & A36 (all other)
Welds: E70XX or F7XX - EXXX
Holes: 15/16 ⌀ unless noted.
Paint: As per Specs.
All holes are for high-strength bolts.
No paint within 3" of holes for these bolts.
B.E. Denotes bearing end.

Figure 7-67. Shop details for the crane runway girders called for in Figure A7-52.

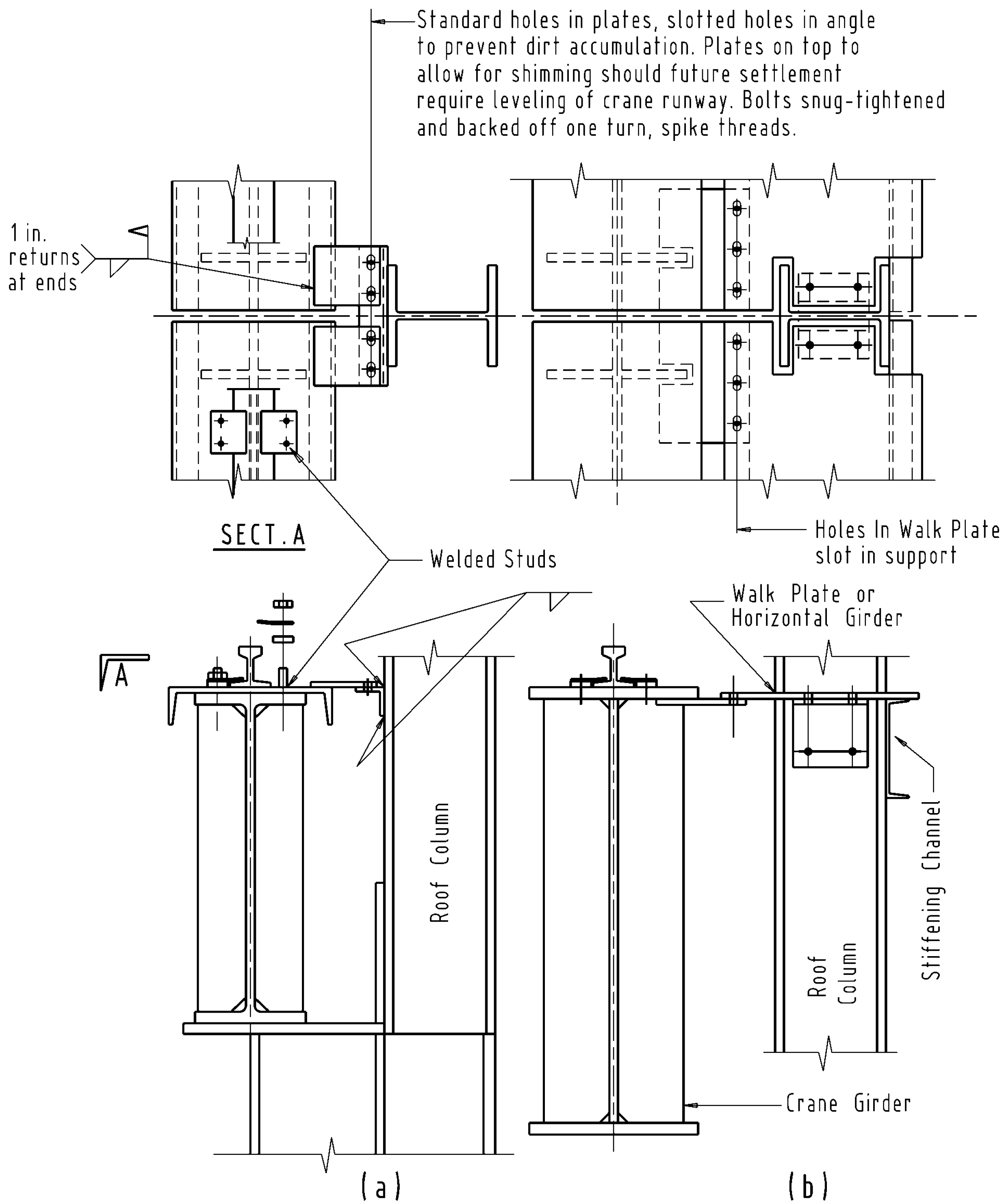


Figure 7-68. Two types of crane runway girders.

wheels. Additionally, transverse bending of the top flange under crane loads has led to fatigue failures when flange-to-web joints were fillet welded in the past. As a result of the service demands on these welds, complete-joint-penetration groove welds are required for top flange-to-web joints of built-up crane runway girders.

In the general notes, the shop is given the option of using either shielded metal arc welding (E70XX) or submerged arc welding (F7XX-EXXX). Anticipating the probable use of submerged arc welding on the flange-to-web joints, the decision was made to clip the corners of the stiffeners, 1 in. × 1 in., to permit their assembly after completion of the full length welds. Here, too, as in the example in Figure 7-67, some shops may elect to use flux core arc or gas metal arc welding.

In addition to the previous discussion of crane runway girder details, the following features should be given attention by the steel detailer:

- Various means are employed for fastening the rails to the girders as shown in the *Manual*, Part 15. The designer must supply typical details for each particular case. Attention is called to Figure 7-68a, where welded studs take the place of through bolts in holding the filler and bent clamp. This will save punching or drilling of girder flanges, but provision must be made for the upset or "flash" that occurs at the base of the stud. Oversize or countersunk holes in the filler bar will permit seating of the assembly. Normally, studs are used when insufficient room exists on the girder flange to punch or drill gages.
- Crane stops are always provided on each girder at both ends of a building. These are shock-absorbing devices used to stop the crane before it travels too far. A design must be supplied for this because it affects the details of the end girders.
- If crane stops are attached to the girders and the rail ends abut to the face of the stop, the total length of rail is shortened. This must be taken into account in ordering rails and splice bars because, usually, no fabrication takes place on crane rails. For light-duty rails, where the stops are clamped to the rails, the rails are ordered to the full length of the runway. Medium- and heavy-duty crane rails are ordered usually with "tight joints." A discussion of this and other data on crane rails, clamps, splices and fasteners is given in the *Manual*, Part 15. Rails are normally ordered as two runs of the total length necessary. The stipulation frequently is added that not more than one rail in each run shall be less than the standard length (either 33 or 39 ft). Rail joints should be staggered on opposite sides of the runway and should not coincide with girder joints. This is accomplished by ordering one odd length piece for each line of rails, which are placed at opposite ends of the runway.
- Crane runway girders often require punching for brackets (usually supplied by others) to support the electric conductors from which the crane draws its power. The designer should obtain this information from the electrical contractor and supply it to the steel detailer.
- At some point along the runway, a ladder is furnished for the crane operator to use to reach the cab. A working platform is also provided at about crane rail level for use by millwrights when inspecting or overhauling the crane. The steel detailer should obtain complete information about the connections for these details.
- Crane runway girders are detailed without regard to locations of crane rail splices. When high-strength bolts are used to hold rail clamps, holes in the top flange shall be spaced to provide the greatest duplication of crane runway girders. The bolts must be fully tensioned.
- When several girders are alike except for slight variations in length (not exceeding 1 in.), some fabricators prefer to make the basic girders identical and keep spacing of web holes and attachments the same. This can be accomplished by:
  - Varying the gages on end connection angles.
  - Adding filler bars to the end connections. If fillers are preferred, their use must be approved by the designer.
- At their bearings over column cap plates, runway girder webs must be square with the flanges.

COLUMNS

As may be seen in Figures A7-66 and A7-52, industrial building columns will vary over a wide range, from the simple roof supports for a storage shed to the highly complex double- and triple-shaft columns supporting multiple crane runways and high roofs in heavy steel mill buildings. Each application is a unique problem for both the designer and the structural steel detailer. The examples given here are limited to light and medium construction, but the treatment of details and connections applies to most columns used in this type of work.

Roof Columns—Light Work

The columns detailed in Figure 7-70 are typical for those used in light buildings that have no traveling cranes. These columns and the members they support are shown in the design of Figure A7-66 and are located on the erection drawing of Figure 7-48a. The combination of all the columns for this job on a single drawing was practical because the only difference in them lay in the vertical bracing connections. The absence of bracing connecting to column C1 and the reversal of

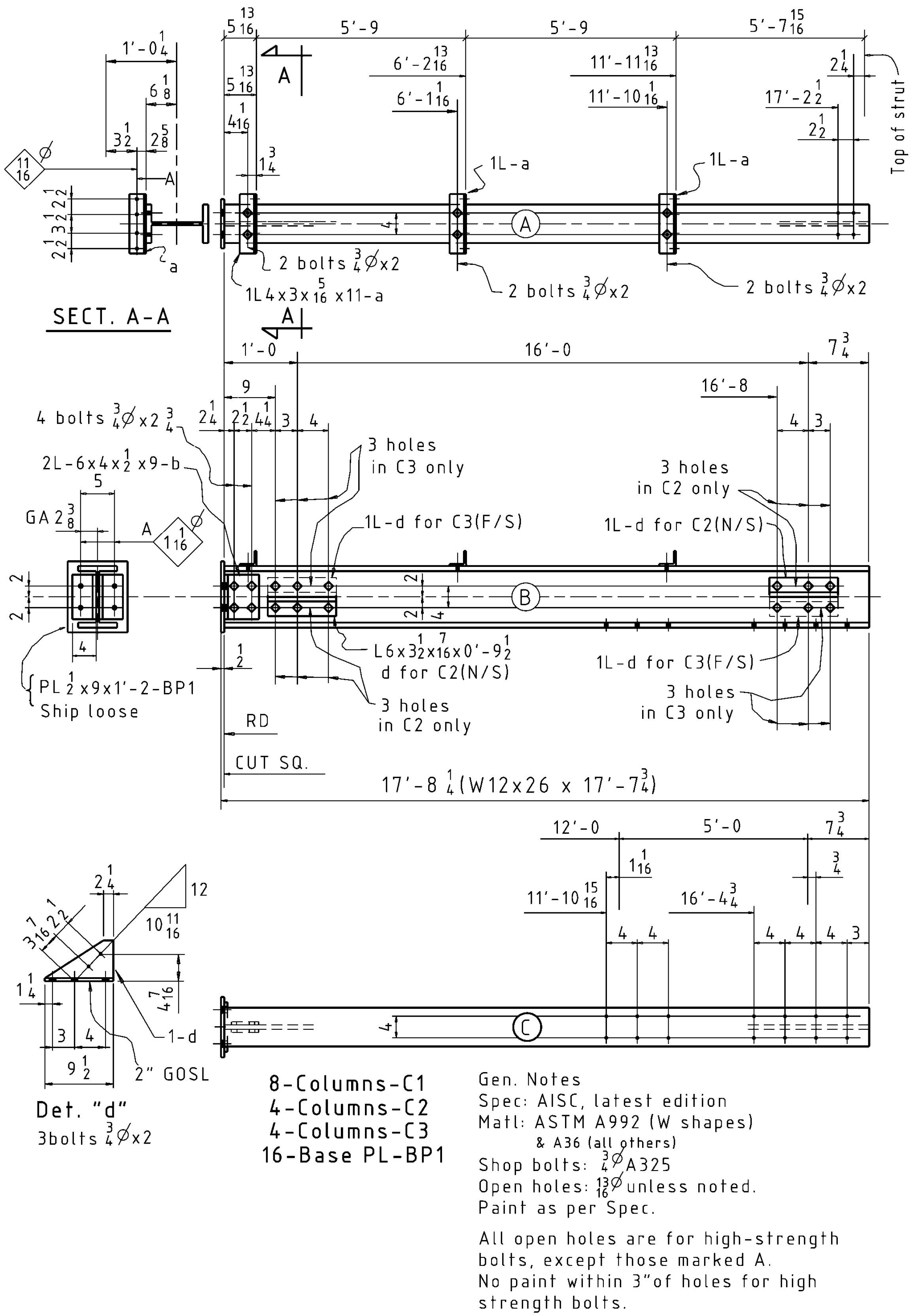


Figure 7-70. Typical detailed columns for those used in light buildings that have no traveling cranes.

bracing connection angles on columns C2 and C3 were noted, readily, in the web view.

Crane and Roof Columns

The columns shown detailed in Figure A7-10 complete the treatment of the various members of the single bay of framing on the design sheet of Figure A7-52. Most of the connections to this column have been discussed previously in this chapter and have been developed to some degree in the layouts and details relating to this particular design.

In practice much of the information required to detail the column is assembled on various free-hand sketches and layouts before the shop drawing is started. Work sheets, such as Figures 7-55 and 7-57, are not finished shop drawings, but they help to organize the work and are of considerable value to other steel detailers who may be preparing shop details of the several connecting members concurrently.

The arrangement of the column on the sheet follows the method given for tier building columns discussed previously. In this case the length of column and the amount of detail required horizontal placement on the sheet. All location dimensions refer to the finished bottom of the column shaft. Dimensions are made continuous, but extension dimensions are given to girt clips and certain hole groups as an aid to the detailing group checker and to the shop inspector. Note the absence of face or direction marks on the flange and web faces. The characteristic shape of these columns ensures their erection in the proper position.

ROOF AND WALL FRAMING

Purlins

A purlin is a horizontal longitudinal member that rests on the top chords of roof trusses or the tops of sloping roof beams (rafters) usually to support sheets of roofing material. It may be designed as a channel (see Figure A7-66) or a W-section (see Figure A7-52). The type and spacing of purlins is a design consideration depending on roof dead and live loads, as well as the limiting lengths of sheeting to be employed. When channels are used, the ridge purlin is placed as close to the peak of the truss as possible to shorten the connection to the purlin on the opposite side of the centerline (see Figure 7-71). This also serves to reduce the overhang of the roof sheeting where it extends beyond the purlin to the edge.

The location of ridge purlins and the spacing of field fasteners is done, conveniently, by a scaled layout as shown in Figure 7-72. This purlin is connected to the chord by means of a clip angle. Earlier in this chapter, the ridge purlins were shown to act as struts in resisting wind loads. For this application, specifications often require that high-strength bolts be used for the connection of the clip angle on the strut to the truss chord. Although typical purlin connections may be made with A307 bolts, many fabricators choose to use high-strength bolts rather than having to handle different sizes of bolts on a job and having to drill or punch different sizes of holes.

At the mid-point of the purlins, the design drawing shown in Figure A7-66 calls for a line of $^5/_8$-in. sag rods to prevent the purlins from deflecting laterally down the sloping roof plane. To be effective the pull exerted by the sag rods must be carried across the roof ridge so that it can be balanced by a corresponding pull on the opposite side of the ridge. This is done by using one of several different connections shown in Figure 7-71. Ridge purlins also are fastened together at other points along their length to increase their transverse stiffness and, thus, permit them to be more effective as struts.

The ridge struts are shown detailed in Figure 7-73. Each pair of channels is fastened together in the shop with special connection angles. The end holes are matched to those in the purlin clip angles ab (Figure A7-49). An open hole is provided in each channel for the string of sag rods. Although the sag rods are $^5/_8$-in.-diameter, $^{13}/_{16}$-in. holes, required elsewhere for the $^3/_4$-in. connection bolts, are used to avoid a change of punch size. The letter T placed near the open holes is required by some fabricators to identify sag rod holes. This is because the accuracy of their location is not as critical as it is for bolt holes.

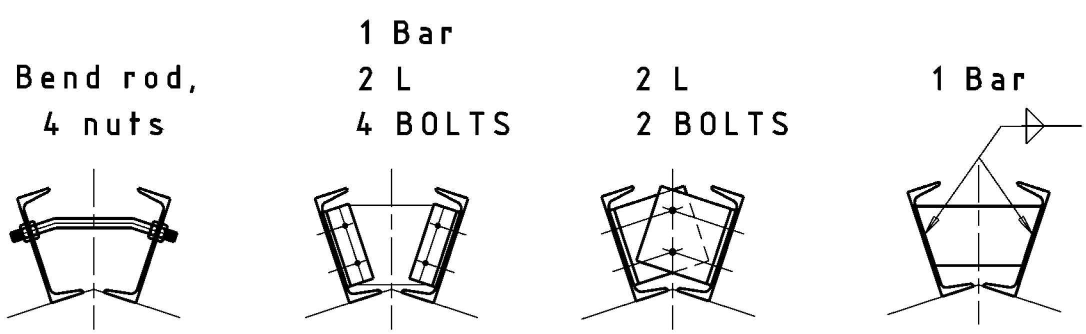


Figure 7-71. Roof ridge connections.

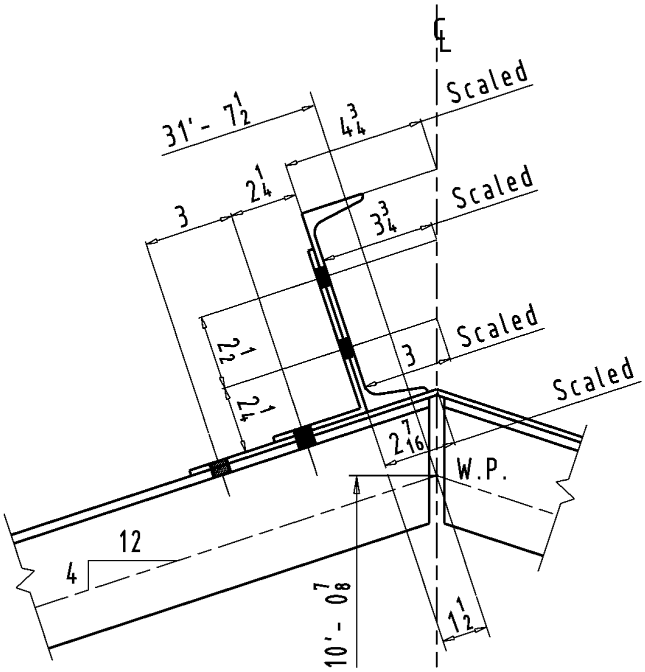


Figure 7-72. Scaled layout.

In the braced bays the two purlins at the quarter-points of the truss span shown in Figure A7-66 will be stressed in compression when the bracing rods are tightened. Because of this the designer has provided some additional lateral stiffness by adding a $2^{1}/_{2}\times2\times^{1}/_{4}$ angle to these purlins in the braced bays. When purlins are considered to act as part of the bracing system, they should be identified on the erection drawing as PS (purlin strut). Otherwise, purlins generally carry the mark P.

A typical purlin detail is shown in Figure 7-74a. The purlins in the two end bays are detailed in Figure 7-74b. The only difference is that at the gable ends the purlins project 18 in. beyond the center of the truss, as called for on the top chord plan of Figure A7-66. When a future extension to the building is anticipated, typical length purlins may be used throughout, the gable end overhang being supported by short stub purlins.

Referring to the purlins at the gable end, if they are W-shapes, request permission from the fabricator to use "blind" holes where such holes would not be objectionable. For instance, where the purlin runs over the top of a rafter or a truss top chord, punch two holes in the rafter or top chord and four holes in the bottom flange of the purlin. This reduces the

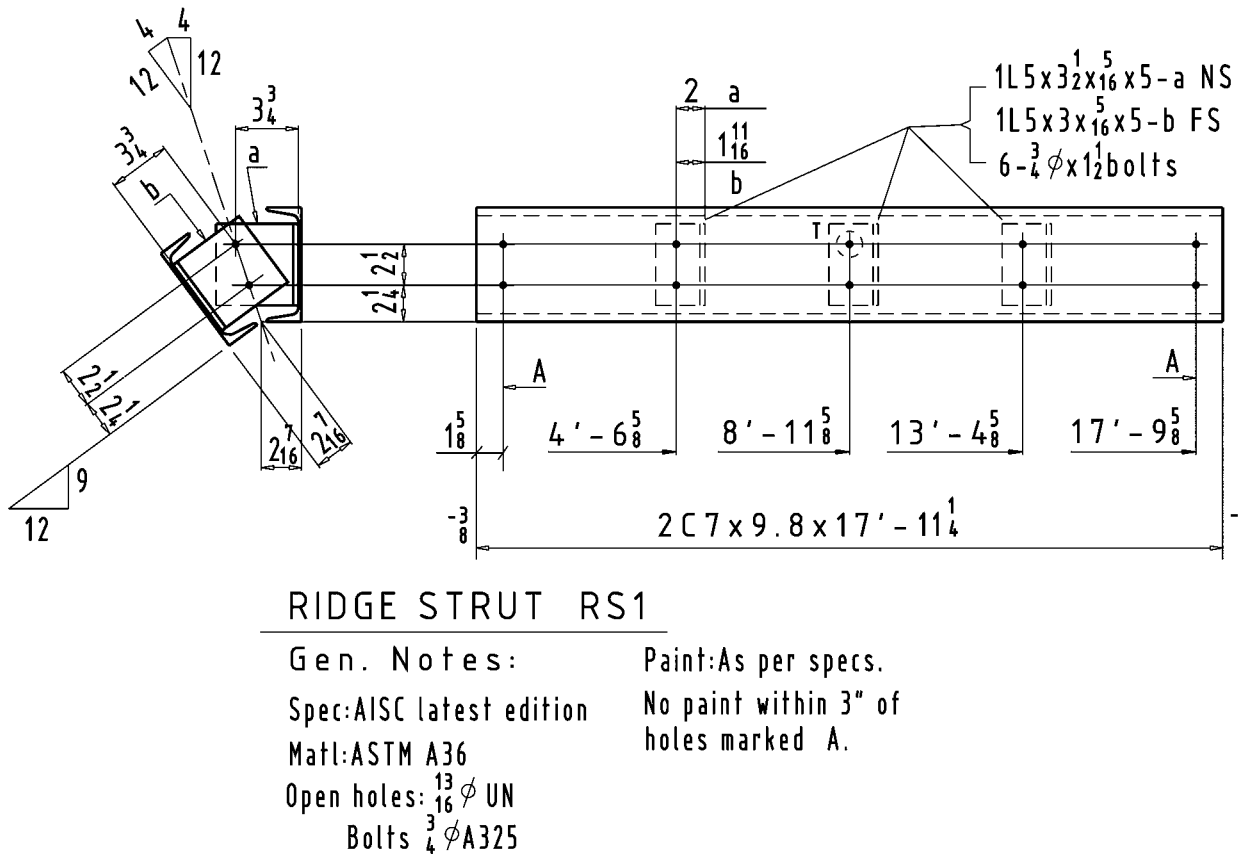


Figure 7-73. Detailed ridge struts.

chance that the holes may be punched opposite to what is desired, especially if the rafters or top chords are composed of right- and left-hand members. Also, this procedure may allow more duplication of members. Note, too, that the open holes will not be visible from the ground.

Usually, channel purlins are erected with their flanges pointing up the slope. Erectors prefer this, as the purlins are easier to erect against the connection clips. Sometimes, designers will want the channels to face down the slope so dirt, moisture and condensation will not collect in the trough of the channel.

Eave Struts

An eave strut is a longitudinal member between the tops of columns at the eaves. Being in the plane of both the horizontal and vertical bracing systems, it is depended upon to carry the compression components of forces from the diagonals in the roof and truss bottom chord bracing into the vertical wall bracing. The design of eave struts may take several forms: W-shape sections, channels paired on either side of the roof column or the simple channel and angle combination member described herein.

Figure 7-75 is a layout of the eave struts shown in Figure A7-66. Each eave strut consists of a 7-in. channel with a $7 \times 4 \times 3/8$ angle on its lower flange. Note the following points in Figure 7-75:

The upper toe of the channel receives the weight of the roof sheeting and its elevation is determined from the roof line. The $7 \times 4 \times 3/8$ angle must extend outward to align with the horizontal channel girts (C6×8.2), so the sheet metal siding will contact all of its supports in the same vertical plane. The eave strut must be connected to the column with high-strength bolts, and the connection must be matched to the column details.

Note that the roof deck sheets make contact only with the edge of the eave channel flange. With care, the screws that attach the roof sheets to the flange can be installed effectively. If the roof slope is very steep, an effective support system could be obtained by attaching an angle to the top of the channel to provide a flat surface for fastening the sheets as shown in Figure 7-76.

The shop details of eave strut ES1 (see Figures 7-48a and 7-48b) are shown in Figure 7-77. As the flange punching of the channel is limited to $^{11}/_{16}$-in.-diameter holes, the decision

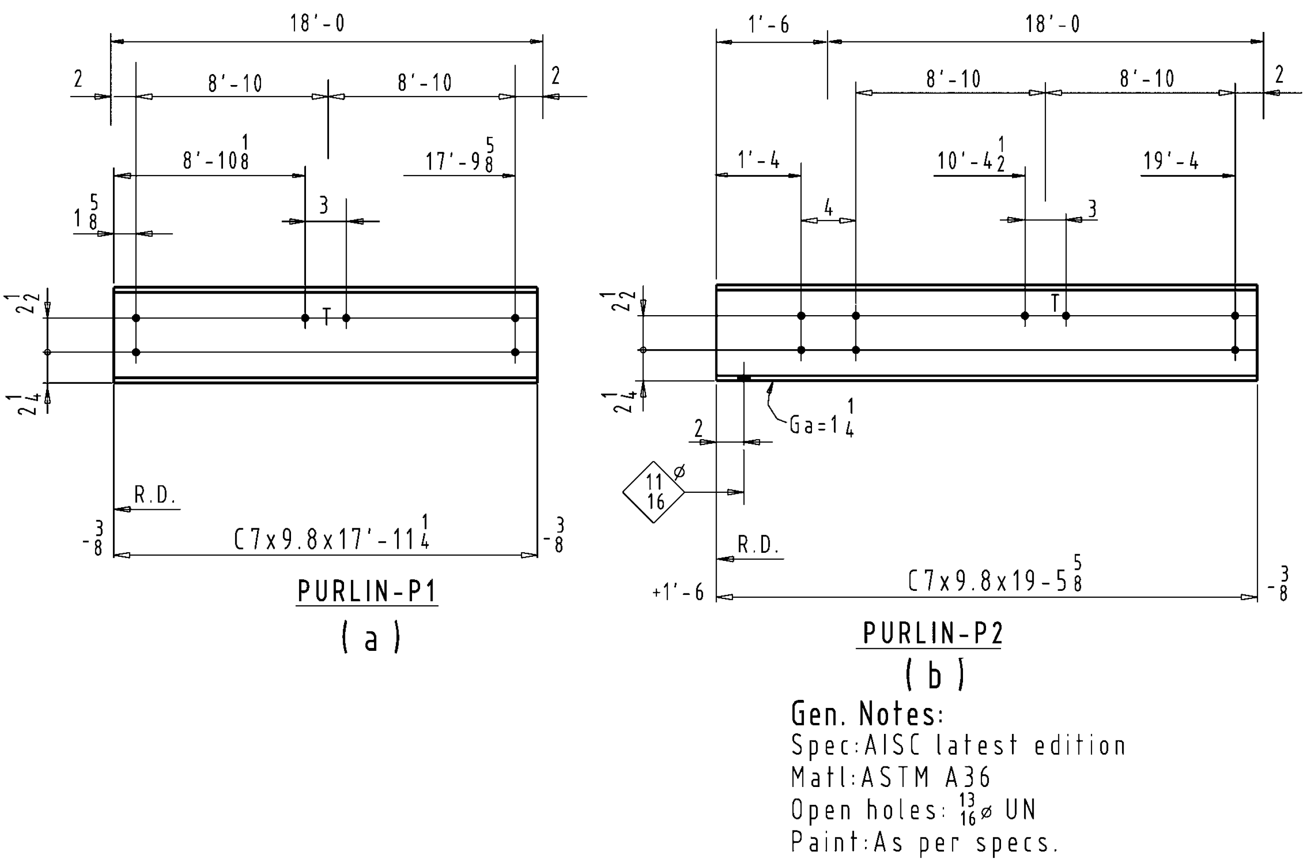


Figure 7-74. Typical purlin detail.

was made to use this punching for both the channel flange and all punching in the angle. This causes no problem as the sag rods are 5/8 in. and the gutter connections can be made up with 5/8-in. bolts.

The eave struts ES2 in the end bays are similar to eave struts ES1. Like the end purlins, they project 18 in. beyond the center of the truss (see the top chord plan of Figure A7-66).

Girt Framing

A girt is a horizontal member in the side or end of a building used to support side covering such as corrugated steel. Figure 7-78a represents an enlarged portion of the cross-section shown in Figure A7-66. Girts are spaced and the sheet metal siding is indicated. Note that, although the siding sheets are shown on 6'-0 and 6'-9 lengths, most erectors prefer full-length sheets, particularly on a small building such as this. The long sheets require less handling, and fewer fasteners are required. However, the availability of such sheets must be investigated before their use is specified. The 6-in. horizontal girts do not support the entire weight of the siding, but act to transfer the weight through the sag rods to the eave strut. Their main function is to transfer wind forces from the

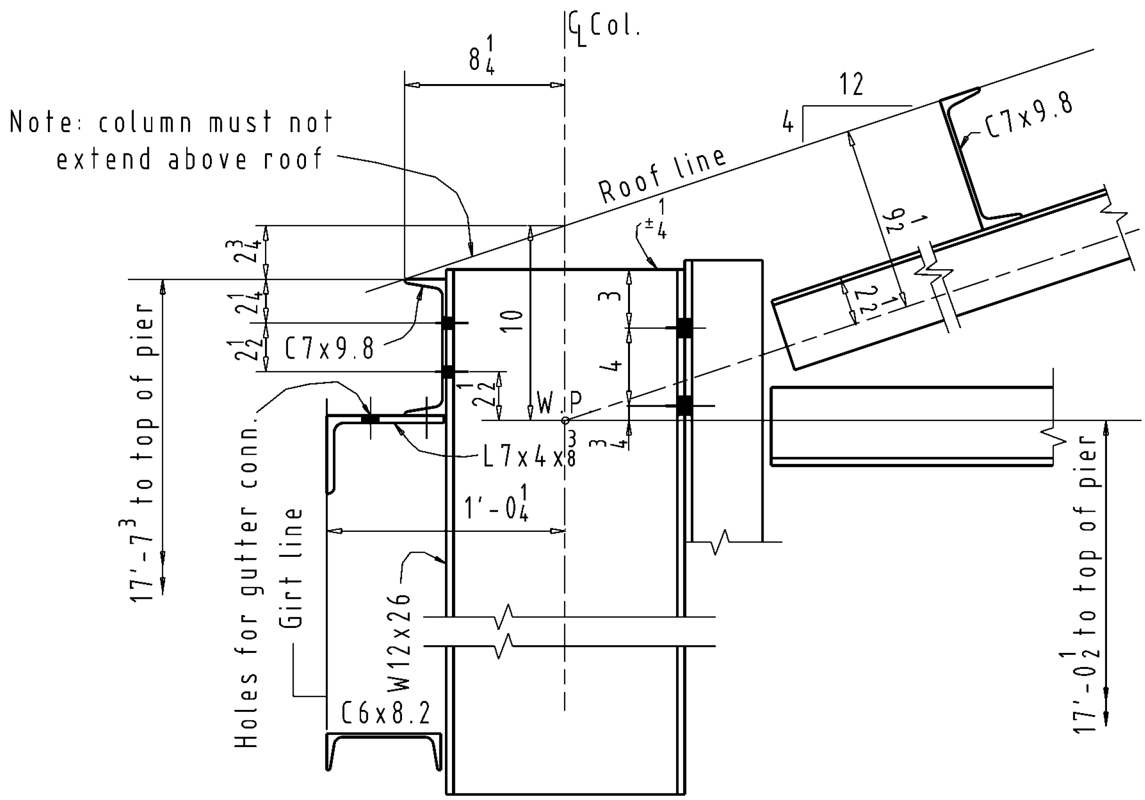


Figure 7-75. Eave strut layout.

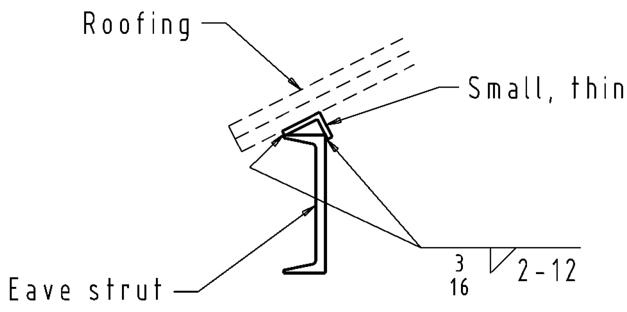


Figure 7-76. Angle attached to the top of a channel to provide a flat surface for fastening the roofing material.

siding to the columns. In general, channel girts should be placed with the flange toes down to avoid collecting dirt. Some erectors prefer that the girts rest on top of their supports (girt clips), for reasons of convenience and safety. If so and the flanges toe downward, one flange must be coped to clear the girt clip.

The drawing in Figure 7-78b shows dimensioning and a detailed layout for the connection of a girt to the W12×26 column flange. Clearance between the ends of the two adjacent girts usually is made 3/4 in. to 1 in. to provide for possible mill overrun in the length of these members. The shop details of girt G1 are shown in Figure 7-79. The girt end connections are matched to the girt clip angles a, shown shop bolted to the flange of the W12×26 columns in Figure 7-70. As for the purlins and eave struts, the end girts on the building must be extended 18 in. beyond the centerline of the col-

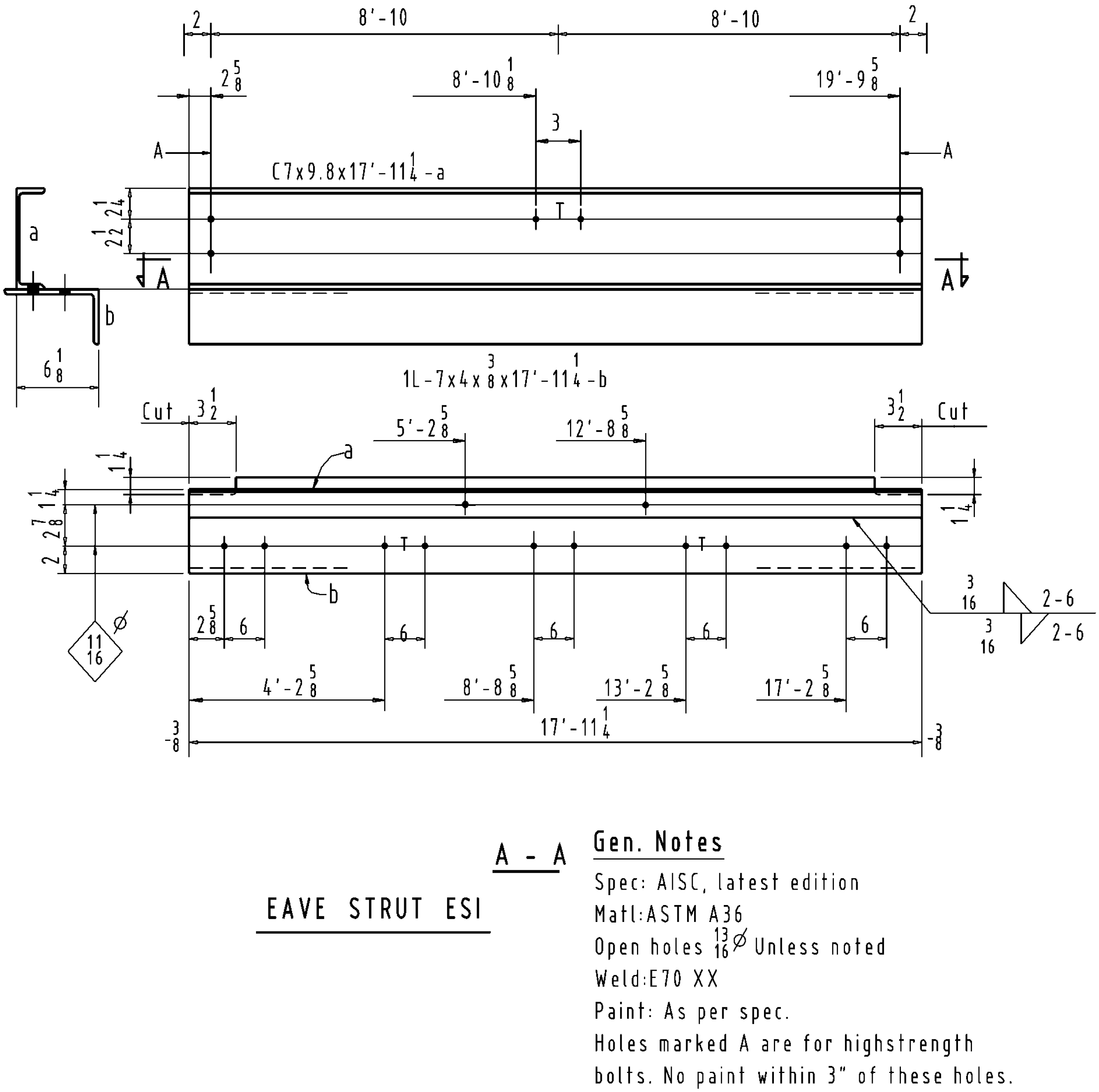


Figure 7-77. The shop details of eave strut ES1.

umn. In this case, however, consideration must be given to girts in the end wall. Should the side and end wall girts be at the same elevation, one or the other must be set back to avoid interference.

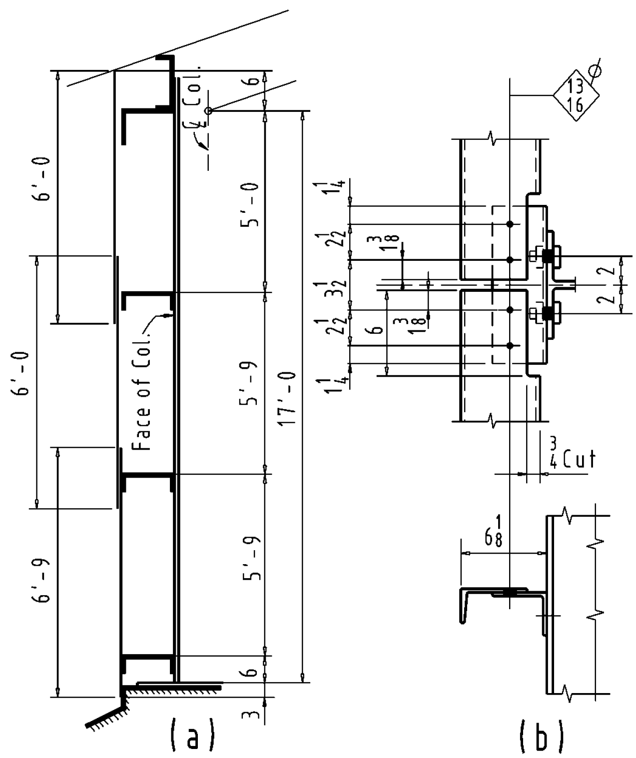


Figure 7-78. Detailed layout for the connection of a girt to a column flange.

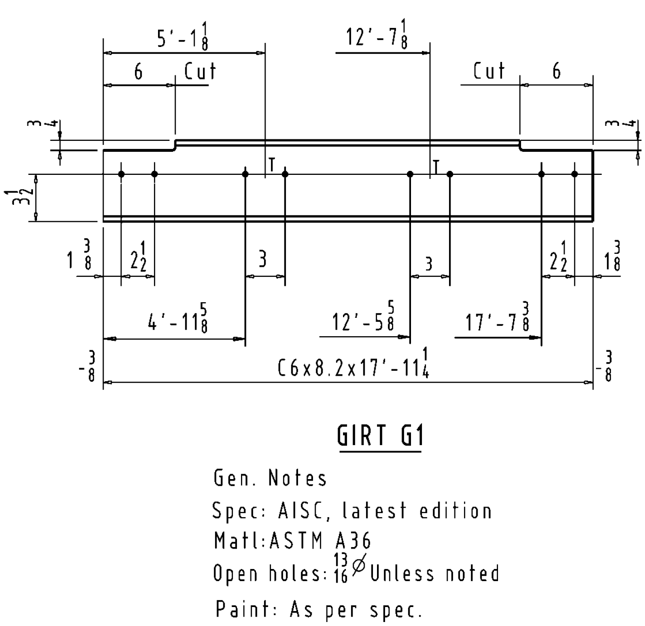


Figure 7-79. Shop details of girt G1.

Connecting girts at building corners is a worthwhile consideration. The varying positions of the sun will produce differential movement at the corners. Restraining the girts at the corners will lessen the possibility of distress in the corner flashing, loose screws and leaks of wind and water.

FIELD BOLT SUMMARY

The preparation of a field bolt summary (see Figure 7-80) should commence promptly with the final issue to the shop of drawings for a job or sequence. Using the shop drawings of shipping pieces, the steel detailer is in the best position to make a summary of the required type, quantity, size and length of field bolts, nuts and washers. The steel detailer must be aware of the importance of preparing this summary accurately. Normally, especially on large projects, it is the only source the erector has to identify the quantity, size and length of bolts required for joint assembly.

From the summary, the shop ships the required quantity of each type, size and length of bolts, nuts and washers to the field. In summarizing total requirements for field connections, an extra 2% of each bolt size (diameter and length) is furnished as specified in Section 7.8, "Field Connection Material," of the AISC *Code of Standard Practice*.

As an aid to the erector in identifying the bolts required at each joint, the following options are available:

- Showing the bolts required at each joint on the erection drawing.
- Listing the bolts required on the shop drawing of the supporting member.
- Preparing a point-to-point list. Many computer detailing programs automatically tabulate and produce such a list.

If bolts need to be installed in some joints in a particular manner by the erector, this information must be specified on the erection drawings either through the use of notes or sketches. For instance, if all bolted connections are of the type having threads excluded from the shear plane, a sketch such as that illustrated in Figure 7-81 should be included to ensure proper installation where thin material is encountered.

Figure 7-80a is a computer-generated listing of field bolts required for the "DEF" high rise job. Note that unavoidable circumstances of the project required using type A325N and A307 bolts of the same diameter. In this situation, the erection drawings will designate the locations where each type of bolt is to be installed.

Figure 7-80b represents a handwritten field bolt list. Here, too, the locations of the handful of type A307 bolts will be

Fabricator: THE BEST FABRICATORS **Page 1**
Job: DEF HIGH RISE
Field Bolt Summary

Ln		Count	Dia.	Type	Length	Head Wash.	Nut Wash.	TC
1	Summary	12	¾	A325N	5 ¼		1 Hardened	Yes
2								
3	Summary	10	¾	A325N	4 ¾		1 Hardened	Yes
4								
5	Summary	90	¾	A325N	3 ¾		1 Hardened	Yes
6								
7	Summary	40	¾	A325N	3		1 Beveled	Yes
8								
9	Summary	8	¾	A325N	2 ½		1 Hardened	Yes
10								
11	Summary	556	¾	A325N	2 ¼		1 Hardened	Yes
12								
13	Summary	1396	¾	A325N	2		1 Hardened	Yes
14								
15	Summary	2591	¾	A325N	1 ¾		1 Hardened	Yes
16								
17	Summary	2	½	A307	2	1	1 Hardened	No
18								
19	Summary	4	¾	A307	7 ¼			No
20								
21	Summary	3	¾	A307	4 ½			No
22								
23	Summary	3	¾	A307	3 ½			No
24								
25	Summary	122	¾	A307	3		1 Beveled	No
26								
27	Summary	13	¾	A307	2			No
28								
29	Summary	115	¾	A307	1 ¾			No
30								
31								
32								
33								
34								
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Figure 7-80a. Computer-generated field bolt summary.

FIELD BOLT SUMMARY

Job ABC HIGH RISE Order No. 1847

Date 8-9-2000 Material AS NOTED Sheet No. FB-1

Line	Quantity	Bolt Dia. - Type	Length		Remarks
1	1182	$\frac{3}{4}\phi$ A325 Hx. Hd.	0	$1\frac{3}{4}$	W/N & W
2	1156		0	2	
3	524		0	$2\frac{1}{4}$	
4	292		0	$2\frac{1}{2}$	
5	92		0	$2\frac{3}{4}$	
6					
7	42	$\frac{3}{4}\phi$ A325 - TC	0	$1\frac{3}{4}$	W/N & W
8	386		0	2	
9	24		0	$2\frac{1}{4}$	
10	12		0	$2\frac{1}{2}$	
11					
12					
13					
14	48	$\frac{7}{8}\phi$ A490 Hx. Hd.	0	2	W/ N & W
15					
16					
17	110	$\frac{7}{8}\phi$ A490 - TC	0	2	W/N & W
18	156		0	$2\frac{1}{4}$	
19	56		0	$2\frac{1}{2}$	
20	92		0	$2\frac{3}{4}$	
21					
22	24	$\frac{3}{4}\phi$ A307 Hx. Hd.	0	7	W/NUT ONLY (NO WASH.)
23	12		0	9	
24					
25					
26					

Figure 7-80b. Handwritten field bolt summary.

Nominal bolt diameter db,in.	Min. thickness t of ply closest to nut to exclude threads from shear plane, in.
3/4	1/4
7/8	1/4
1	3/8
×Values shown assume one 5/32-in. thick washer is present. If washer is not present, increase minimum thickness by 1/8-in.	

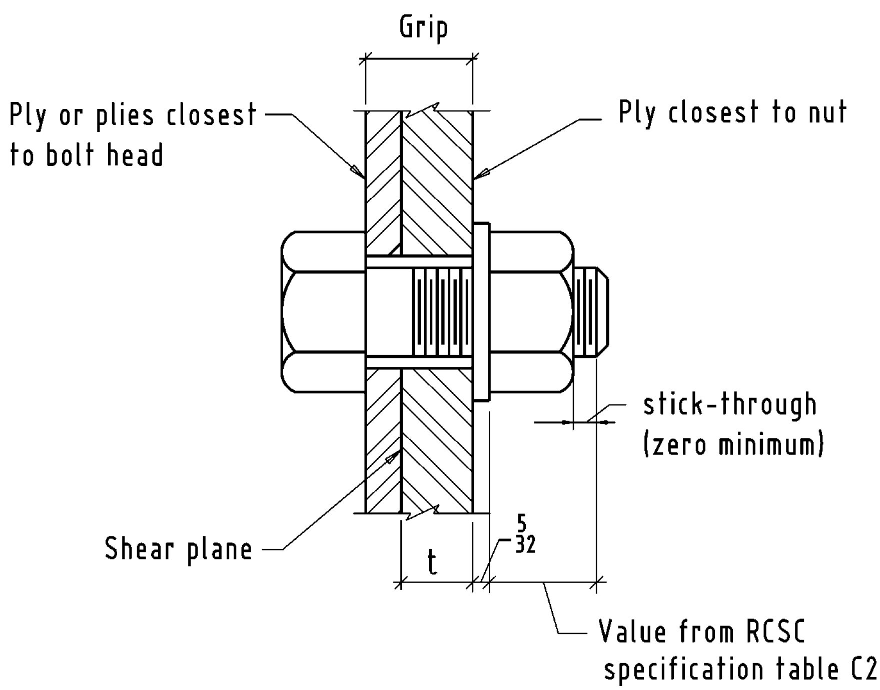


Figure 7-81. Sketch showing threads excluded from the shear plane.

shown on the erection drawings. On this list, the "TC" refers to tension control bolts.

NONSTRUCTURAL STEEL ITEMS

Section 2.2 of the AISC *Code of Standard Practice* (Other Steel, Iron or Metal Items) lists items that are considered "Nonstructural-Steel," although they may be manufactured from steel. However, some of these items closely resembling the items of "Structural Steel" listed in Section 2.1 may be included in the contract of the steel fabricator. When a detailing group is retained by the fabricator to perform the preparation of shop drawings for a job, the group should be made aware of these special items to be included in the work. Often, too, these special items will be presented on design plans other than the structural steel design drawings. When instructed in the construction documents, the steel detailer will need to refer to other types of plans such as architectural, HVAC, foundation, mechanical, etc.

Also, the fabricator may be required to furnish items such as nonsteel bearings as part of the contract. These products may be combinations of steel and bronze or steel and Teflon as produced by specialty manufacturers. A steel detailer may be required to furnish generic shop drawings of such items for the fabricator's use in procuring them. The information for such drawings usually can be obtained from catalogs supplied by these manufacturers.

DETAILING ERRORS

Errors made in the drawing room are costly in terms of wasted material and man-hours expended to correct them in the shop and in the field. Utmost care must be exercised by the steel detailer in ensuring that the information given on a drawing (shop or erection) is accurate. Drawing room errors can spell the difference between the fabricator's making a profit or not. The following are some of the more common errors found on shop and erection drawings.

Dimensional

The total of a series of incremental dimensions (for instance, giving spacing between groups of holes in a beam web) does not agree with the extension dimension to one or more groups. Also, the bottom hole of a line of holes in the web of a beam (the holes being dimensioned from the top of the beam) is too close to the bottom fillet to permit acceptable attachment of a fitting.

Bills of Material

The billed weight and/or size of a rolled shape disagrees with the description shown on the shop drawing. Also, the width and/or length billed for a plate does not match the shop detail drawing. Another error by the steel detailer is when a piece is billed in the shop bill larger than the size of material ordered for it. The steel detailer has the responsibility to ascertain that the material ordered is adequate to make the piece. If the material is undersize, the steel detailer must prepare a material change order to provide the correct size or length. Another error occurs when the quantities of pieces (assembling or shipping) are wrong.

Missing Pieces

This is caused when the steel detailer fails to produce a shop drawing for each piece required on a job.

Clearance for Welding

In manual arc welding, the operator or welder must position the electrode (a flux coated rod approximately 14 to 18 in. long and up to $^3/_8$ in. diameter) in such a way that the nearest point of contact with the base metal is at the far end of the rod. Having positioned the electrode with its far end in close proximity to the weld joint, the welder lowers a head-protective hood. Observing the work through the hood's protective window of dark glass, the welder strikes the arc and manipulates the electrode to build up the full-weld cross-section. To deposit a satisfactory weld, the operator must have sufficient room to manipulate the electrode and must be able to see the root of the weld with the protective hood in position.

All things being equal, the preferred position of the electrode when welding in the horizontal position would be one in a plane forming an angle of 30° with the vertical side of the fillet being laid down. However, in order to avoid contact with some projecting part of the work, this angle (angle X in Figures 7-82a and b) may be varied slightly. A simple rule used by many fabricators to ensure adequate clearance for the passage of the electrode in horizontal fillet welding is that the root of the weld shall be visible to the operator and that the clear distance from the weld root to a projecting element, which might otherwise obstruct passage of the electrode, shall be at least one-half the height of the projection—distance y/2 in Figure 7-82b.

A special case of minimum clearance for welding with a straight electrode is illustrated in Figure 7-83 (which shows a beam as it would lie on the skids with its web in a horizontal position). In this case, the governing obstruction is the inside of the flange. Assuming a $^1/_2$-in. setback for the end of the beam and a $^3/_8$-in. outside diameter of electrode, this distance would not be less than $1^1/_4$ in. for an angle width, w, of 3 in., nor less than $1^5/_8$ in. when w equals 4 in.

One technique used by fabricators is to cut the end (noted optional cut in Figure 7-83) of the connection angle to a bevel and, thereby, gain additional clearance. The width of the overhanging flange is a major factor in determining how much room is required for welding. Welded connections of this type in the web of a column are difficult, particularly, because of the "boxing" effect created by the projecting flanges of the column.

Another clearance that is critical to the deposition of fillet welds is the "shelf" on which it is to be placed. Figure 7-84 shows the minimum recommended shelf for normal-size fillet welds made with the shielded metal arc welding (SMAW) process. Submerged arc welding (SAW) would require a wider shelf to contain the flux, although sometimes this is provided by clamping auxiliary material to the member. This also applies to welded column splices, as shown in the *Manual*, Part 14, Case IV.

The steel detailer must not only consider clearances required to manipulate the welding electrode, but must also

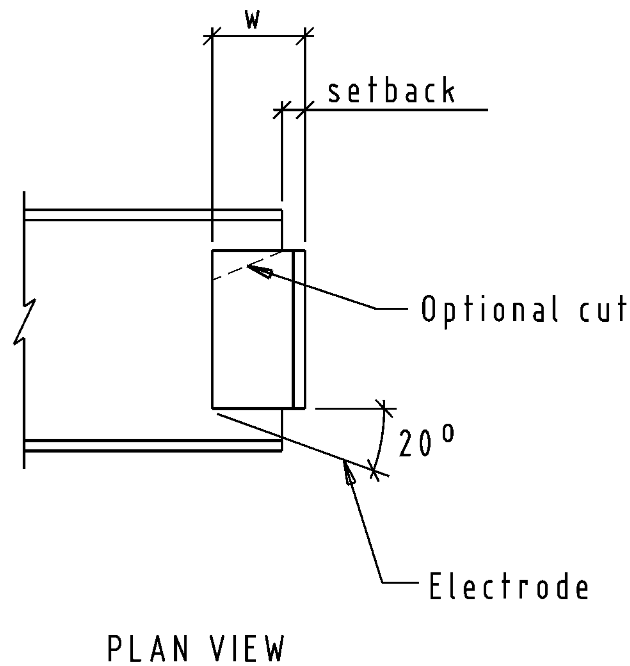


Figure 7-83. Welding clearance.

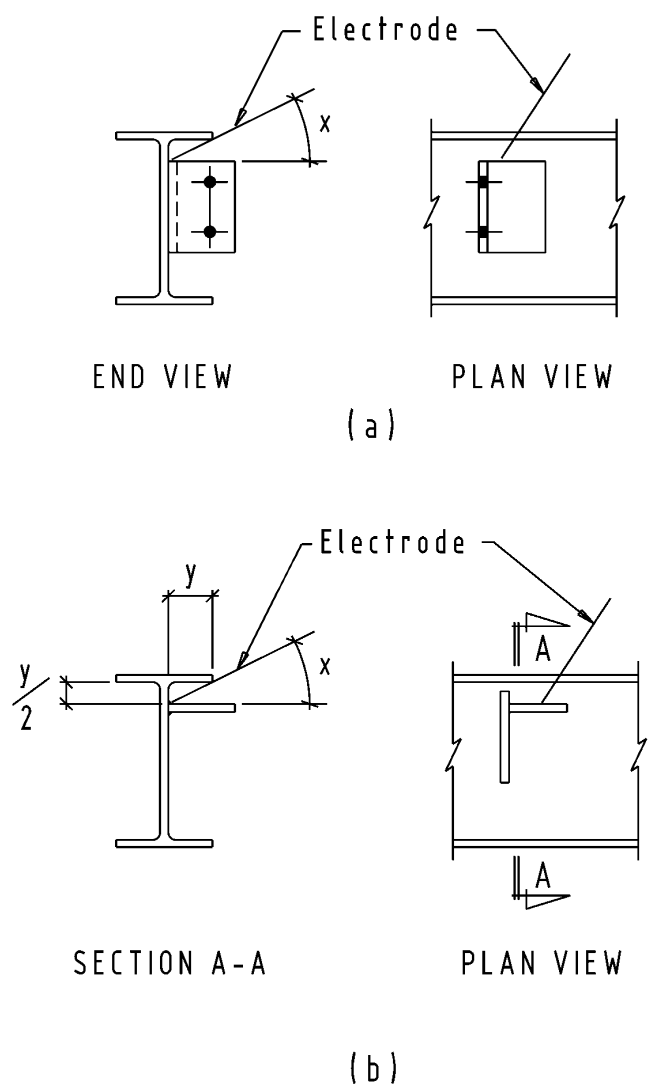


Figure 7-82. Preferred welding position.

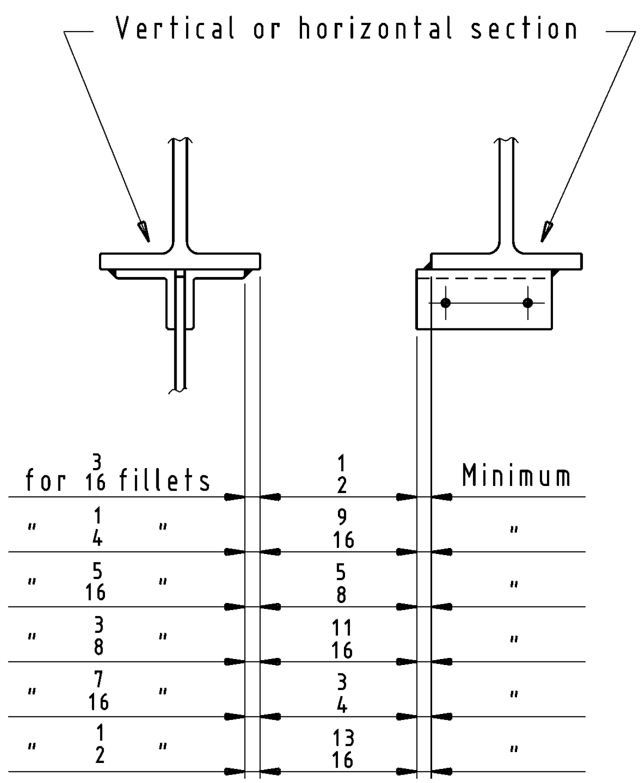


Figure 7-84. Minimum recommended shelf for normal-size fillet welds made with the SMAW process.

provide adequate space to permit depositing the required fillet weld. In Figure 7-85 welds are shown along the toes of two 3×3×5/16 connection angles. Section A-A shows that the nominal dimensions of the connected part provide only 1/8 in. of surface width on the column flange for each weld. If the details call for 3/16 in. or 1/4 in. fillet welds, obviously the welds cannot be made. The detail shown in Figure 7-85 is not a good detail, even if the column flange is 6 3/4 in. wide, because any slight deviation in the actual dimensions of the work due to rolling or fabrication tolerances would result in an undersized weld.

Clearance for Bolting

The assembly of high-strength bolted connections requires clearance for entering and tightening the bolts with an impact wrench. The clearance requirements for conventional high-strength bolts are shown in Table 7-16 of the *Manual*, Part 7. Similarly, for tension control (TC) bolts, the entering and tightening clearances are given in Table 7-17 of the *Manual*. Violation of these clearance requirements could result in un-erectable situations, creating costly rework in the field.

Flanges of beams seated on angles attached to column webs may have to be cut to clear not only the column flanges themselves, but also the projecting fasteners used on column flange connections. Heads of shop-installed bolts attaching a connection on the opposite side of a column web may also interfere with erection.

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

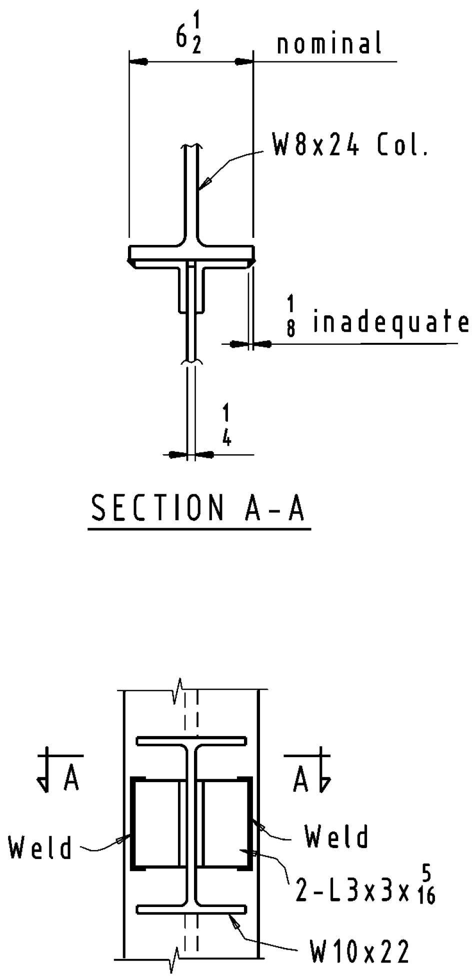


Figure 7-85. Improper connection detail.

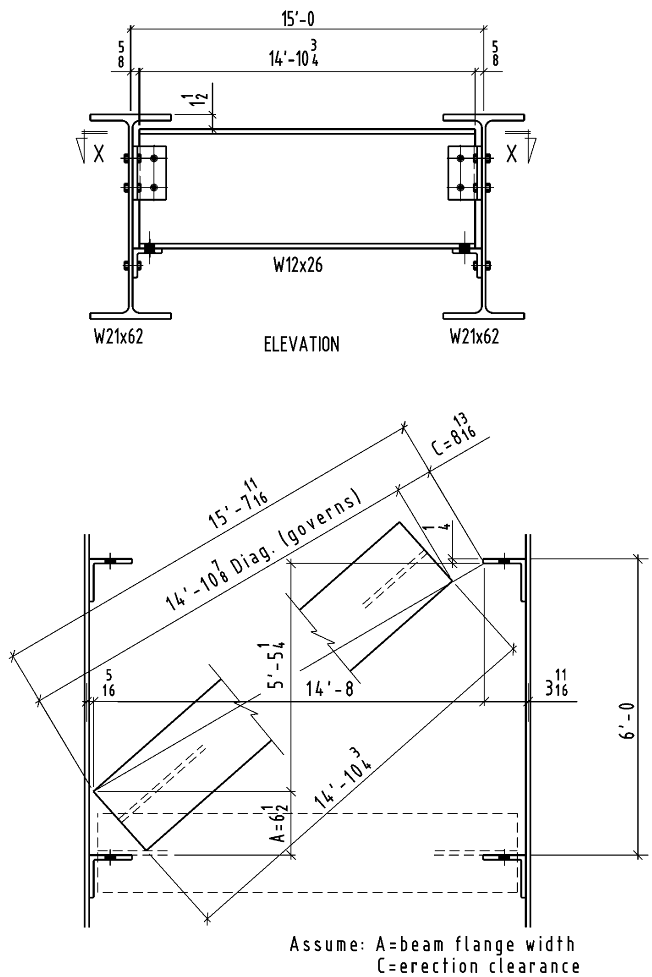


Figure 7-86. Clearance available for erecting seated filler beams where the side-angle connections are to be shop fastened to the supporting girder beams.

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 temporarily. While suspended in the crane sling, the beam is tilted until its ends clear the edges of the column flanges. Then, it is rotated back into a horizontal position and landed on the seats. The greatest diagonal length (B) of the beam should be about 1/8 in. less than the face-to-face distance between column webs. Also, it must be such as to clear any obstructions above, otherwise the obstructing detail must be shipped bolted for temporary removal. To allow for possible overrun in length of plain punched beams, the shop should be given definite instructions on the overrun that can be tolerated without trimming.

As the mathematical solution of erection clearance problems may require several steps and may become time consuming, solving the more complicated problems by a graphical method is expedient. In this method, an accurately scaled layout is prepared showing supporting beams, columns or girders with their center-to-center locations and all flanges or detail fittings that might cause interference. A second layout of the member to be erected is made to the same scale on a separate piece of tracing paper.

By placing a layout of the member to be erected over the layout of the supporting framing, the member can be positioned and moved past interferences to simulate its actual

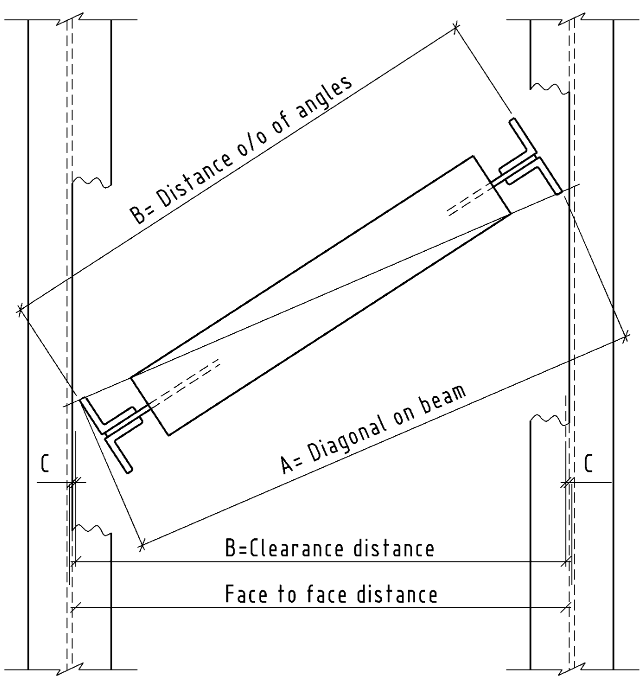


Figure 7-87. Study of erection clearances for the case of framed filler beams.

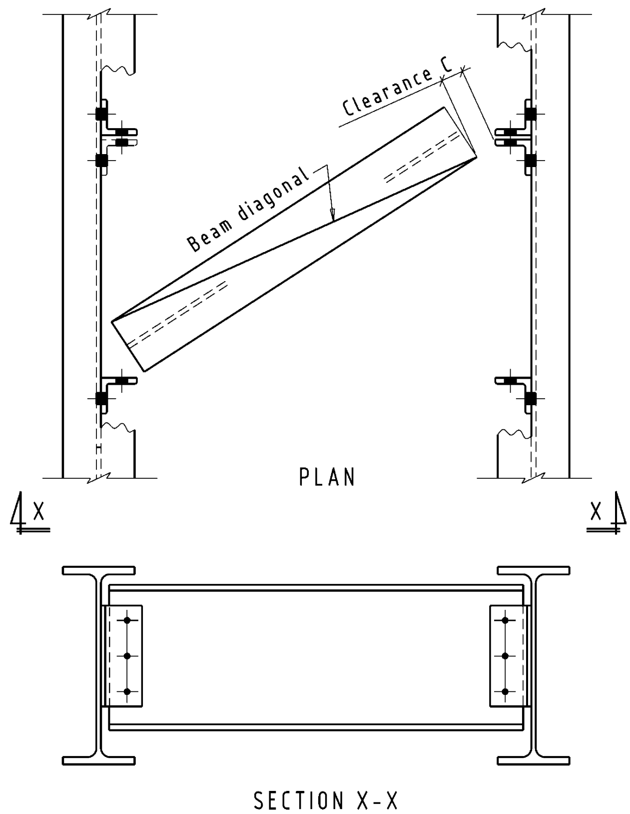
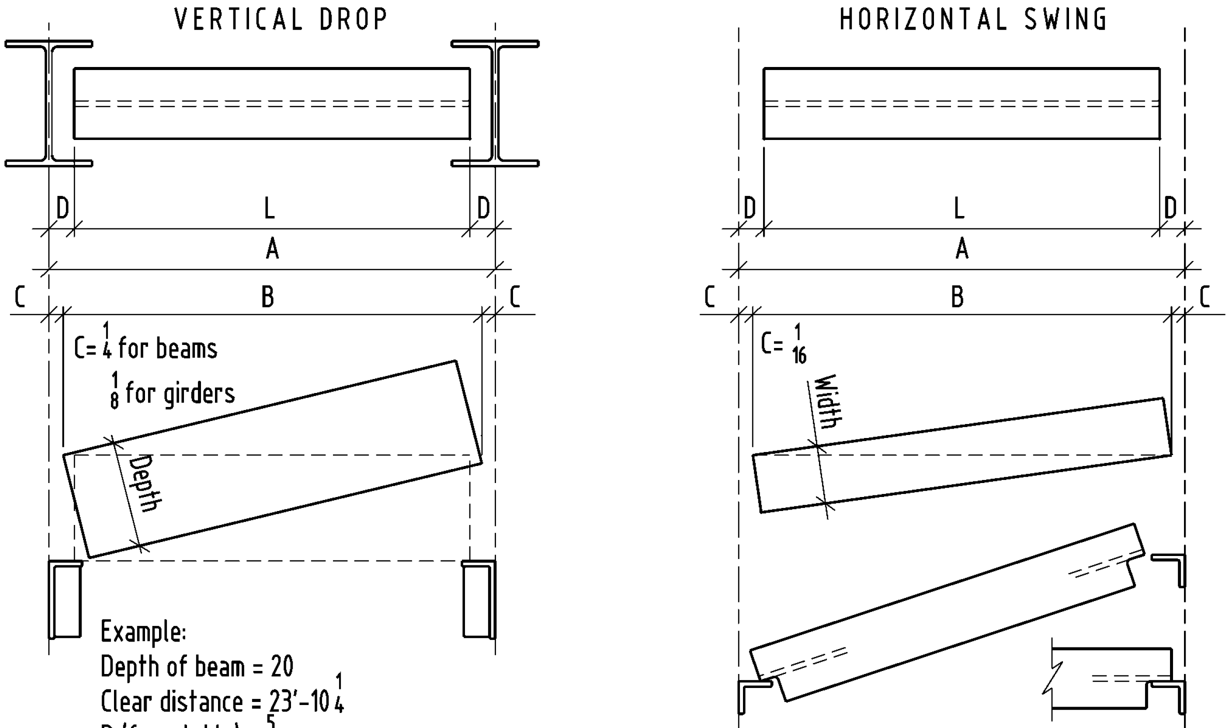


Figure 7-88. Study of erection clearances for the case of framed filler beams.

Table below gives clearance distance A in feet, that will allow piece to be erected without tilting columns, or moving members to which it connects.



Example:
Depth of beam = 20
Clear distance = 23'-10 1/4
D (from table) = 5/8
Length (A-2D) = 23'-9

Arrange detail so that piece will swing clear of stiffeners or clip angles.

	Actual Depth	D: 5/16	3/8	7/16	1/2	9/16	5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
BEAMS	8	22	11	8	6	5								
	9	28	14	10	7	6	5							
	10	34	17	13	10	7	6	5						
	12	49	25	17	13	10	9	7	5					
	14		33	22	17	14	11	9	7	6				
	15		38	26	19	16	13	10	8	7	6			
	16		43	29	22	18	15	11	9	8	7	6		
	18			37	28	22	19	14	11	10	8	7		
	20			45	34	27	23	17	14	12	10	9	8	7
	21			50	37	30	25	19	15	13	11	10	9	8
	24				49	39	33	25	20	17	14	13	11	10
	27				61	49	41	31	25	21	18	16	14	13
	30				76	61	51	38	31	26	22	19	17	16
	33					73	61	46	37	31	27	23	21	19
	36					87	73	55	44	37	31	28	25	22
GIRDERS	36 1/2				75	64	56	45	38	32	28	25	23	21
	42 1/2					87	76	61	51	44	38	34	31	28
	48 1/2						97	79	66	57	50	44	40	36
	54 1/2							100	83	71	62	56	50	46
	60 1/2							123	102	88	77	68	62	56

Actual Width	D: 1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8
8	22	11	8	6	5				
9	28	14	10	7	6	5			
10	34	17	12	9	7	6	5		
12	49	25	17	13	10	9	7	7	6
14	66	33	22	17	14	11	10	9	8
16	86	43	29	22	18	15	13	11	10
18	109	55	37	28	22	19	16	14	13
20	134	67	45	34	27	23	20	17	15

A= Clear distance, metal to metal. Mill overrun and interfering heads should be considered.
B= Diagonal length of piece.
C= Clearance
D= Deduction = C+ (B-L) 1/2
L= Length of piece = A-2D

Figure 7-89. Various beam and girder framing clearance conditions and tabulated combinations of length, width, depth and clearances that will permit erection without interference.

travel during erection. Any interferences should be noted and steps taken to eliminate them, as discussed previously. This graphical method enables the steel detailer to tilt the member and move it laterally and vertically at the same time. Frequently, it demonstrates an easy solution to a clearance problem, which on first inspection appears to be extremely difficult or impossible.

Figure 7-89 provides a table of various beam and girder framing clearance conditions and tabulates combinations of length, width, depth and clearances that will permit erection without interference. The steel detailer should verify a few tabulated results with actual calculations to understand the limitations that can apply to such a table. As noted in the foregoing discussions, the effect of any projecting element may establish a new clearance line.

The possibility of an interference between flanges of beams framing to adjacent faces of a column must be investigated. If interference is found, it can be eliminated by cutting the flange of one of the beams. In Figure 7-90, beams A and B are to be flush top. As beam B must be cut to clear the flange of the column, this cut, logically, is extended to clear the flange of beam A (Figure 7-90a), instead of requiring a cut on both beams (Figure 7-90b). No useful purpose is served by making the more complicated cut shown in Figure 7-90c, although such a cut would provide the necessary erection clearance also.

Other Detailing Errors

Listed here are some additional common detailing errors. The list does not complete all the possible errors that can occur in preparing shop and erection drawings.

- The number of bolt holes in a member does not match those in its supporting connection.
- Bolt holes are the wrong diameter.
- Gages between lines of holes in flanges of W- or similar shapes do not fit the width of the flange. The gage may be too narrow so as to encroach upon the fillet of the shape or too wide so that the holes are too close to the edge of the flange.
- Connections are omitted.
- Copes on beams are missing, unnecessary, too small or too large. Deep or very long copes may require web reinforcing of the beam.
- Wrong type of steel is used.
- Wrong weld profile is used.
- On erection drawings, the north direction is indicated improperly or missing.
- Combining welding and bolting when such is unnecessary or improper (contrary to the AISC *Specification*).
- Reversing slopes. This is especially a cause for error when the slope is close to 45°.
- Improper presentation of rights/lefts, as-shown/opposite hand, right-hand/left-hand, thus/reverse, all of which are discussed in Chapter 4.
- Placing incorrect shipping marks on the erection drawings.

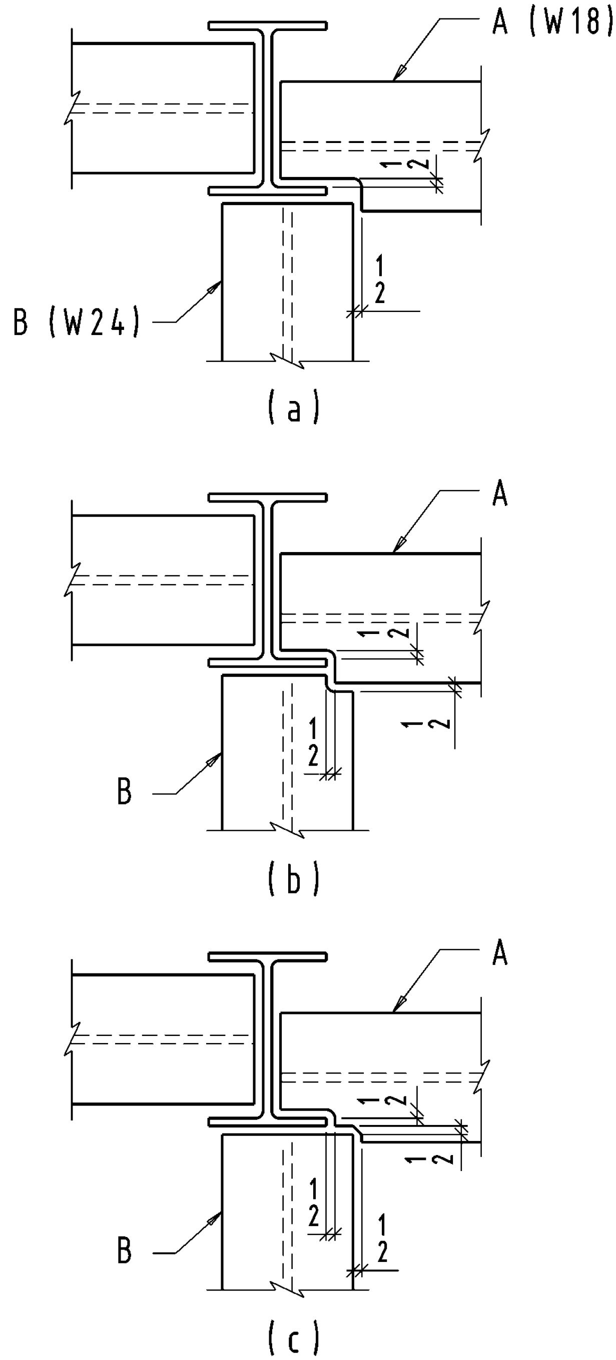


Figure 7-90. Flush-top beams A and B.

- Omitting bearing stiffeners in beam webs, if required.
- Omitting required stiffener and/or doubler plates in column webs at moment connections.
- Painting areas of members on which field welding is to be performed.
- Painting members to be fireproofed or embedded in concrete or masonry.
- Welding symbols are not shown correctly.
- Painting the tops of members which receive field applied shear studs.

CHAPTER 8

DETAILING QUALITY CONTROL AND ASSURANCE

The quality control and assurance procedures that are commonly employed in the preparation and maintenance of shop and erection drawings.

The first quality control initiative should be an adequate scope list of inclusions, adequate job standards and fabrication/erection preferences for the steel detailer to incorporate into any job.

Detailing organizations have two means available to them by which they may become certified as committed to top quality practices and procedures, ensuring the use of recognized quality assurance procedures in their offices. Independent firms whose primary business is the production of shop detail drawings may volunteer for certification through the Quality Procedures Program of the National Institute of Steel Detailing. A fabricator's in-house drawing room becomes certified as part of the certification of the entire plant through the voluntary AISC Certification Program for Fabricators. These programs are devised to confirm to the construction industry that a certified structural steel fabricating plant has the personnel, organization, experience, procedures, knowledge, equipment, capability and commitment to produce fabricated steel to the required level of quality for a given category of structure. For participating structural steel detailing organizations, these programs function to establish practices and procedures that ensure the translation of the intent of contract documents into shop and erection drawings that meet the requirements of the client.

An independent audit of a detailing group is conducted on site. The audit of the group requires the satisfaction of a specified amount of criteria, as required by the evaluation checklist. A firm becomes certified as having quality assurance procedures if it receives a minimum number of points required on the checklist. The evaluation is based upon written documents, verbal interviews with members of the detailing organization and visual examination.

CHECKING

The basic function for controlling the quality of the output of a detailing firm is the process of checking. Following preparation of advance bills of material by a steel detailer, copies of the bills are given to a checker. The checker is an individual who, by reason of experience and ability, has advanced successfully from a beginning steel detailer to a more responsible position. The checker reviews the bills for accuracy in quantities, shape nomenclature, description of material, lengths, finish requirements (such as for columns or heavy base plates), material specification and any special requirements. Also, the checker reviews the contract documents to ascertain that all the material to be included in the fabrication contract has been listed on the advance bills.

Similarly, after structural members are detailed on a shop drawing, a copy of the drawing is given to a checker, who reviews and verifies every sketch and dimension on the drawings. The checker:

- Verifies the correctness of connections in accordance with the design drawings.
- Checks material listed in the shop bill.
- Makes sure the information on the shop drawing is presented clearly.
- Ascertains that all the steel included in the contract has been detailed.
- Initials the drawings.

By checking all details, the intent is to ensure that errors have not been made, that standard detailing procedures have been followed and that the shop and erection drawings are in full compliance with the contract documents. The AISC *Code of Standard Practice* says that the fabricator is responsible for the transfer of the information contained in the contract documents into accurate and complete shop and erection drawings and information for the fit up of parts in the field. These responsibilities are clearly outlined in the AISC *Code of Standard Practice* Section 4.2.

Both steel detailers and checkers are assigned to teams under a drafting project leader, who plans and organizes work assignments and supervises production to meet a prearranged schedule—one that is planned to fit into the shop production program. The shop must, in turn, coordinate its work schedule with the erection program at the construction site.

The detailing manager supervises the drafting project leaders, and through them the checkers and steel detailers. In small detailing groups a checker may assume the duties of a drafting project leader and also be responsible for the operation of the department. Similarly, a small number of experienced steel detailers, organized as a detailing group, are able to prepare shop drawings and exchange them with each other for checking. Still, one of them will perform the duties of drafting project leader and detailing manager for purposes of administration and record keeping.

BACK-CHECKING

When a checked copy of an advance bill, shop drawing or erection drawing is returned to the steel detailer who prepared it, the steel detailer is expected to review all the comments and corrections placed on it by the checker. In other words the steel detailer checks the checker. If the steel detailer disagrees with a checker's comment or correction, that individual should discuss the matter with the checker to resolve any differences of opinion on how an item of material on an advance bill or information on a shop or erection drawing should be presented. The steel detailer is as much responsible for the accuracy of the shop drawing as is the checker. Once any differences of opinion are resolved, the steel detailer makes the necessary corrections and returns it to the checker for final review and the signature of the checker in the space provided on the advance bill or shop drawing. The intent of the system for checking and back-checking is to ensure that the documents are as accurate as possible prior to issuing to the fabricator and the erector.

APPROVAL OF DRAWINGS

After shop detail drawings have been completed, checked and back-checked, prints, reproducibles and/or electronic files of the drawings must be submitted for approval before shop fabrication operations begin. This applies to all shop and erection drawings. As a rule, this approval is given by the owner's designated representatives for design and construction per AISC *Code of Standard Practice*, Paragraph 4.4.

In preparing shop and erection drawings, any discrepancies discovered in the design plans or specifications by the group must be referred to the owner's designated representative for construction through the communication channels established for the project, as provided in AISC *Code of Standard Practice*, Paragraph 3.3. While it is the fabricator's responsibility to report any discrepancies that are discovered in the contract documents, it is not the fabricator's or the detailer's responsibility to discover discrepancies, including those that are associated with the coordination of the various design disciplines. The quality of the contract documents is the responsibility of the entities that produce those documents. Because of the contractual relationships that are often involved, the referral of discrepancies must be made at the earliest possible moment. Instructions to resolve the discrepancies must be received before proceeding further with the affected portion(s) of the work. The responsibility for any extra costs that may result from such discrepancies rests with the owner as provided for in AISC *Code of Standard Practice*, Paragraph 9.3.

Prior to submission of shop and erection drawings for approval, the fabricator must notify the owner's designated representative for design or owner's representative of intentions to request changes to connection details described in the contract documents.

Except for small orders involving relatively little work, the fabricator's drawings often are submitted for approval in sequences (or blocks, or divisions). For example, for multi-story buildings, plans and details may be submitted for one tier (two floors) at a time. Prompt approval of the shop and erection drawings is essential to meet work schedules and delivery dates. The fabricator allows approval time as arranged with the owner's designated representatives for design and construction or per AISC *Code of Standard Practice* Section 4.4. The drafting project leader is required to keep a record showing the dates on which shop and erection drawings are sent for approval and the dates on which they are returned. In conjunction with the detailing manager, the drafting project leader must maintain a close watch over this aspect of the work.

When the approving agency returns prints bearing notations for corrections, the indicated changes must be checked. These changes may affect other work in progress in the drafting room or work already released for fabrication. If found to be in order, the changes should be made promptly and corrected drawings should be issued to the shop. Sometimes, the approver misunderstands the intent of a detail or note on a fabricator's drawing and adds an erroneous correction comment. When this occurs or if a noted correction cannot be made, the detailing manager must contact the approver (directly or through project communication channels) to resolve the problem. If required, new copies of corrected drawings are resubmitted for approval.

Acceptance by the owner's designated representatives for design and construction indicates that authority's approval of the suitability of the details and interpretation of the design documents and of the design strengths of the connections. However, the correctness of dimensions on shop detail drawings and the general fit-up of parts furnished by the fabricator to be assembled in the field remain the responsibility of the fabricator and the steel detailer.

FIT CHECK

A detailing group, at its discretion or by the fabricator's request, may do a fit check soon after the final shop drawings for a job or sequence are sent to the shop for fabrication. A fit check is a partial checking of the shop drawings to ensure the proper connection of the members in the field. It should constitute an assurance to the fabricator that all connections will match as detailed, copes and gages are dimensioned accurately, hole sizes and locations are correct, clearances are adequate, and overall lengths are correct. Fit checking is done by an experienced checker. It should be completed prior to the start of fabrication, even though template, layout and minor detail preparation may have begun.

MAINTENANCE OF RECORDS

The primary reason for keeping records is to provide a running history of the progress of each piece of material in the job so that its status can be traced quickly. Good records provide documentation of revisions and other events, aid in the determination or justification of extras and back-charges, and furnish supporting data in the unpleasant event of litigation. Another reason for maintaining records is to provide assurance to a client that a detailing group (independent firm or a fabricator's drafting department) is qualified to perform the work assigned to it. The detailing group must maintain records to:

- Record receipt of original and revised design plans and specifications and of other project design documents.
- Record distribution of these plans and specifications when the detailing group is required to send them to other firms performing sublet fabrication for the structural steel fabricator. Such other firms may include, but not be limited to, a steel joist supplier, miscellaneous metals fabricator, metal decking supplier, etc.
- Record each and every advance bill and material change order prepared and its status.
- Record each and every job standard sheet, layout, shop and erection drawing, field work drawing, field bolt summary, and sketch (for "request for information" or "design clarification") prepared by the group and its status.
- Record the receipt and status of each and every "extra" and back-charge (explained later) involving the steel detailer.
- Keep written records of phone conversations with other parties involved with the project as these conversations apply to the conduct of the work.
- Prepare and submit to the client on a scheduled basis a status report indicating the progress of detailing and any problems that may cause delays. The report should include the quantity of shop drawings estimated for the job, detailed, checked, sent for approval, returned from approval and issued to the shop. Also, it should include the estimated date for submitting final shop drawings to the fabricator.
- Keep faxes, e-mail and other transmissions on file.

Contract Document Control/Revisions

When the detailing group initially receives the design plans and specifications, it lists on a record form (Figure 8-1a) each plan number, the revision number on the plan and a brief description of the plan (such as Foundation Plan, First Floor Plan, Second Floor Plan, Roof Plan, Details). Some plans may have revision identifications on them at the time they are issued for construction. The record form has a series of columns with headings to list this information and the dates received. Often, the first column may be headed "DESIGN DRAWING" (or some similar designation). The second column may be headed "DESCRIPTION" to list the title of each plan. This is followed by a series of columns for listing the dates of initial receipt and revision number of designs and of all subsequent receipts of design revisions.

Revisions to the design drawings and specifications shall be made either by issuing new design drawings and specifications or by reissuing the existing design drawings and specifications. In either case, all revisions, including revisions that are communicated through the annotation of shop and/or erection drawings, shall be clearly and individually indicated in the contract documents. The contract documents shall be dated and identified by revision number. Each design drawing shall be identified by the same drawing number throughout the duration of the project, regardless of the revision.

If a detailing group is required to distribute plans and specifications to other firms, a similar record of the receipt and distribution of such information is necessary. The records are maintained by the detailing manager or the drafting project leader, depending upon the organization of the group. Transmittal letters that accompanied receipt and distribution of the documents should be kept to support the data in the records.

Revisions to design plans may be commonly enclosed in a scalloped, curved line, sometimes called a "cloud." The cloud may be given a number, usually inside a triangle, to identify it.

Another method used to alert involved parties to a design revision is through the issue of a bulletin by the owner or general contractor. This document describes in writing and, if suitable, with sketches what changes have been made to a design plan. Although the bulletin eliminates the reissue of design drawings to show the revisions, the design drawings should be revised.

Upon receiving revised designs and/or specifications, copies are made and issued to the individuals holding previously issued copies. These earlier copies should either be isolated or boldly marked VOID as they may be needed later for tracking design changes and backing up change requests (extras). Each detailing group must retain one set of designs and specifications as the "file" set. It will consist of all of the plans used to order material; to prepare shop and erection drawings; and to record information obtained from such sources as a contractor's memo, phone conversation with the fabricator or owner's designated representative for design, etc. This set of plans is the basis for determining if extras are in order and, possibly, if back-charges are appropriate. Similarly, a "file" set of specifications must be kept. Such files are retained until the project is completed and paid for

Contract Document Log-Drawings

ABC High Rise	JOB NO: 1847		Revision or Issue (R/I)									
			0		1		2		3		4	
Design Drawing	Description	Current Status	Date Rec'd	R/I Date	Date Rec'd	R/I Date	Date Rec'd	R/I Date	Date Rec'd	R/I Date	Date Rec'd	R/I Date
S-1	General notes	D	4/14	4/03	5/01	4/26						
S-2	Foundation plan	D	4/14	4/03	5/01	4/26						
S-3	First floor framing plan	D	4/14	4/03	5/01	4/26						
S-4	Second floor framing plan	D	4/14	4/03	5/01	4/26						
S-5	Third floor framing plan	D	4/14	4/03	5/01	4/26						
S-6	Roof framing plan	D	4/14	4/03	5/01	4/26						
S-7	Penthouse roof framing plan	D	4/14	4/03	5/01	4/26						
S-8	Column schedule	D	4/14	4/03	5/01	4/26						
S-9	Moment frame elevation	D	4/14	4/03	5/01	4/26						
S-10	Braced frame elevation	D	4/14	4/03	5/01	4/26						
S-11	Concrete sections and details	D	4/14	4/03	5/01	4/26						
S-12	Concrete sections and details	D	4/14	4/03	5/01	4/26						
S-13	Steel sections and details	D	4/14	4/03	5/01	4/26						
S-14	Steel sections and details	D	4/14	4/03	5/01	4/26						

Fabricator: The Best Fabricators

Contact: Ken Doe

Phone: (444)-444-4444

Fax: (777)-777-7777

General Contractor: Fantastic Building Company

Status Legend

A – Preliminary

B – Bid Set Issue

C – Mill Order Only

D – For Construction

Superb Detailers
180 Anchorbolt Drive
Baseplate, U.S.A.

Figure 8-1a. Contract document log—drawings.

Contract Document Log-Specs.

ABC High Rise		JOB NO: 1847		Revision or Issue (R/I)									
				0		1		2		3		4	
Spec. Section	No. Pgs.	Description	Current Status	Date Rec'd	R/I Date	Date Rec'd	R/I Date	Date Rec'd	R/I Date	Date Rec'd	R/I Date	Date Rec'd	R/I Date
05120	09	Structural steel	D	05/01	04/26								
05300	10	Metal decking	D	05/01	04/26								
05220	08	Joists / Joist Girders	D	05/01	04/26								

Fabricator:	The Best Fabricators
Contact:	Ken Doe
Phone:	(444)-444-4444
Fax:	(777)-777-7777
General Contractor:	Fantastic Building Company

Status Legend

A – Preliminary
B – Bid Set Issue
C – Mill Order Only
D – For Construction

Superb Detailers
180 Anchorbolt Drive
Baseplate, U.S.A.

Figure 8-1b. Contract document log—specifications.

or as required by the project specifications or fabricator, whichever is longer.

Shop and Field Document Control/Revisions

The detailing group maintains accurate files for keeping track of documents it generates. Although the format will vary from one detailing group to the next, logs contain the same information and are divided into several columns to record it. Figure 8-2 is an example of a computer-generated log, which illustrates one of several ways that information may be recorded. The first column may be headed "DRAWING NO." and lists each and every drawing number on the project prepared by the detailing group. This listing includes all erection drawings, shop drawings, field work drawings, layout drawings, field bolt summaries and sketches prepared for submission to others for clarification of design information. The second column may be headed "DESCRIPTION" and briefly describes the content of each drawing such as bracing layout, anchor rod plan, erection drawing, base plates, embeds, columns, beams, etc. Several columns follow to list the date drawings are sent to approval and returned, the dates drawings are resubmitted for approval (if necessary) and returned, the dates drawings and any revised copies are issued to the shop, the dates drawings and subsequent revisions are issued to the erector, and the dates file prints are issued to the general contractor or others who, according to the project specifications, are designated to receive them. Some logs include space to record the initials of the preparer and checker of each drawing. If the contract specifications require special submissions or transmissions to other parties, the log must reflect these as well. Letters transmitting advance bills and drawings to other parties for ordering material, fabrication, erection, file, etc. serve as back-up for the log. Most fabricators require transmittal forms to accompany issuance of advance bills and shop drawings; for other fabricators no such transmittals are needed.

In Figure 8-2, "TRANS" stands for the number of the Transmittal letter. "R" stands for "Resubmittal," which is a function of the approval status. A "Y" in the column indicates the drawing must be resubmitted; a blank indicates resubmittal is not required. "TYP" stands for Transmittal Type and indicates whether the drawings are "For Issue Only" (FIO) or "To Be Returned" (TBR).

Also, the group keeps a record of the preparer, checker and dates when advance bills of material and material change orders are issued for ordering material. Generally, advance bills are not revised for issuing to the material ordering department. Each time material is changed after the advance bills have been released for ordering, a separate change order form is prepared and issued.

When an issued shop or erection drawing is revised, the original is changed, and the changed area is normally surrounded by a numbered cloud as described earlier. This highlights the revised portion(s) of the drawing so the changed portion(s) stands out to all who read it. As is done on design plans, the revision is recorded by number and a brief description near the title block of the drawing. Some fabricators require that the cloud(s) of the previous revision is removed each time a drawing is revised, but the number of the revision remains. Thus, a drawing having been revised three times will show a cloud(s) identified by the number 3 and only the numbers 1 and 2 identifying areas where previous clouds had been. Normally, the number is enclosed in a geometrical shape such as a triangle or circle. This procedure helps to maintain the clarity of the drawing. Other fabricators leave the clouds in place so as to maintain a clear trail of revisions.

Before a shop drawing is revised, the steel detailer should verify with the shop the status of the work on the drawing. If the revision cannot be handled in the shop, the changes will need to be made at the job site by the erector—an expensive procedure and one to be avoided if possible. The field work on the revised shipping piece is communicated to the erector by showing the work on an erection drawing. Some fabricators prefer to issue a field work drawing, which is separate from the customary erection drawings and is for the sole purpose of describing the work to be done on a particular shipping piece. This system also aids in keeping track of the costs for performing the work in the field. Field work drawings are discussed in Chapter 6.

Another document retained by the detailing group is a log of "extras," which are the costs incurred by the group in doing work that is, in their judgment, beyond the scope of work originally contracted to perform. Regardless of the cause of the additional work, the steel detailer must maintain a record that should show the date and source of the change request, its description, its impact on the work of the group, the resulting cost, the date the extra was issued to the appropriate party and its status (in progress, accepted, rejected, awaiting payment, paid). It should show the time spent to order any additional material and make changes to drawings, and describe in detail the work performed on each drawing involved. The format of these documents will vary from fabricator to fabricator. The importance of carefully and accurately completing these forms immediately upon receiving a change cannot be overemphasized.

Back-charges to the detailing group may come from different sources for a variety of reasons. A log of these back-charges must be maintained. It should show the date each back-charge was received by the detailing group, the source of the back-charge, its cost, a description of the back-charge and the steel detailer's reply. The forms for recording back-charges can be kept in a file to serve as a log.

Virtually every project is designed and detailed to some extent by phone conversations. Although some of these

09/01/2000 **DRAWING ISSUE HISTORY FOR JOB 1847** **PAGE**
ABC HIGH RISE

DWR NUM/STATUS	DESCRIPTION/ISSUE INFORMATION	REV	ISSUED	DUE BY	RECEIVED	TRANS	R	TYP	PURPOSE
E-1	ERECTION PLAN-FLOOR	2							APPROVED
REVISE & RESUBMIT	Fantastic Building Co.	0	07/05/00	07/19/00	07/19/00	00002	Y	TBR	For Approval
REVISE & RESUBMIT	Fantastic Building Co.	1	07/21/00	08/04/00	08/04/00	00003	Y	TBR	For Approval
APPROVED	Fantastic Building Co.	2	07/29/00	08/12/00	08/12/00	00004		TBR	For Approval
APPROVED	Very Good Erectors	2	09/01/00			01001		FIO	For Field Use and Dist.
E-2	ERECTION PLAN-ROOF	2							APPROVED
REVISE & RESUBMIT	Fantastic Building Co.	0	07/05/00	07/19/00	07/19/00	00002	Y	TBR	For Approval
REVISE & RESUBMIT	Fantastic Building Co.	1	07/21/00	08/04/00	08/04/00	00003	Y	TBR	For Approval
APPROVED	Fantastic Building Co.	2	07/29/00	08/12/00	08/12/00	00004		TBR	For Approval
APPROVED	Very Good Erectors	2	09/01/00			01001		FIO	For Field Use and Dist.
1	Braced Frame Details	0							APPROVED
APPROVED	Fantastic Building Co.	0	07/05/00	07/19/00	07/19/00	00002		TBR	For Approval
APPROVED	T.B.F. Shop Fabrication	0	07/19/00			00501		FIO	For Fabrication
3	Braced Frame Details	0							
APPROVED AS NOTED	Fantastic Building Co.	0	07/05/00	07/19/00	07/19/00	00002		TBR	For Approval
APPROVED AS NOTED	T.B.F. Shop Fabrication	0	07/19/00			00501		FIO	For Fabrication
4	BEAM DETAILS	0							APPROVED
APPROVED	Fantastic Building Co.	0	07/21/00	08/04/00	08/04/00	00003		TBR	For Approval
APPROVED	T.B.F. Shop Fabrication	1	08/04/00			00502		FIO	For Fabrication
6	BEAM DETAILS	0							APPROVED
APPROVED	Fantastic Building Co.	0	07/21/00	08/04/00	08/04/00	00003		TBR	For Approval
APPROVED	T.B.F. Shop Fabrication	0	08/04/00			00502		FIO	For Fabrication
7	BEAM DETAILS	0							APPROVED
APPROVED	Fantastic Building Co.	0	07/21/00	08/04/00	08/04/00	00003		TBR	For Approval
APPROVED	T.B.F. Shop Fabrication	0	08/04/00			00502		FIO	For Fabrication
10	BEAM DETAILS	0							APPROVED
APPROVED	Fantastic Building Co.	0	07/05/00	07/19/00	07/19/00	00002		TBR	For Approval
APPROVED	T.B.F. Shop Fabrication	0	07/19/00			00501		FIO	For Fabrication
11	COLUMN DETAILS	0							APPROVED
APPROVED	Fantastic Building Co.	0	07/05/00	07/19/00	07/19/00	00002		TBR	For Approval
APPROVED	T.B.F. Shop Fabrication	0	07/19/00			00501		FIO	For Fabrication
14	COLUMN DETAILS	1							APPROVED
APPROVED	Fantastic Building Co.	1	07/21/00	08/04/00	08/04/00	00003		TBR	For Approval
APPROVED	T.B.F. Shop Fabrication	1	08/04/00			00502		FIO	For Fabrication

Figure 8-2. Computer-generated log.

conversations may be followed up by issuance of revised design plans, revised project specifications or bulletins, a log must be maintained of any and all such conversations dealing with the detailing of the project. The log should at least record the date of the conversation, the parties having the conversation, the subject and the conclusion/decision. As in other record forms, the amount and type of information to be listed depends on the fabricator's preference. One accepted method of maintaining such a log is by using a form (either of the steel detailer's own making or as issued by the fabricator of the project) on which the preceding information can be written. The document can then be transmitted to other interested parties for confirmation and information. Normally, if a detailing group has direct phone access to the owner's designated representative for design, it will be required to keep the fabricator informed of all conversations.

To complement the use of direct phone conversations, the communication between the steel detailer and the owner's designated representative for design can be conducted satisfactorily using faxes or email. By their use the steel detailer's query and the owner's designated representative for design's response are in writing and are available for copying to the fabricator and others as required by contract. Prior to sending a fax, the steel detailer should alert the owner's designated representative for design of a forthcoming fax to help ensure an early response.

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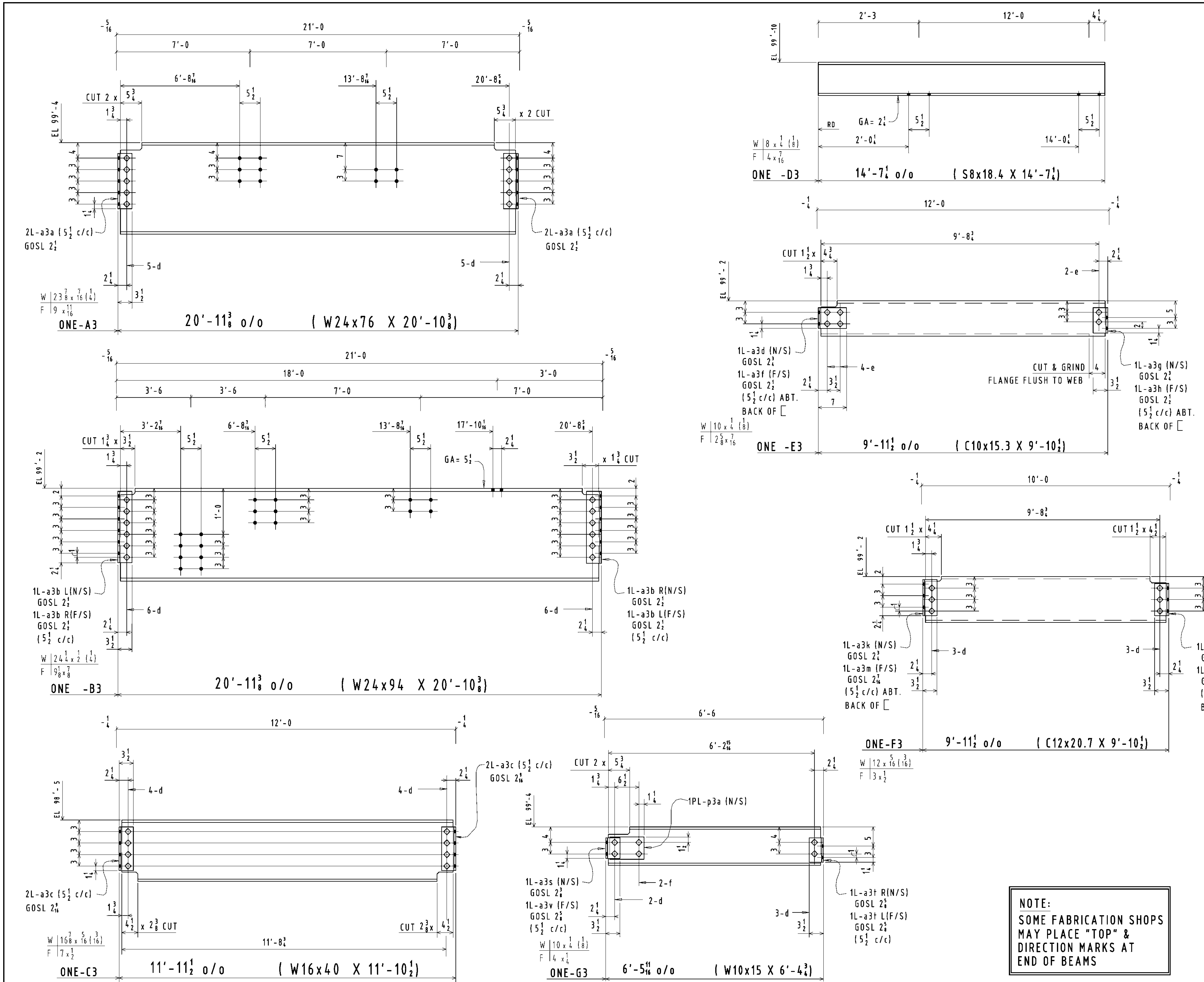
APPENDIX A
LARGE FORMAT DRAWINGS

The following section has been provided for improved drawing clarity while using this text. All pull-out drawings are numbered with respect to the chapter in which they are referenced.



NOTE:
SOME FABRICATION SHOPS MAY PLACE "TOP" & DIRECTION MARKS AT END OF BEAMS

BILL OF MATERIAL

	QUAN	MARK	DESCRIPTION	LENGTH FT	LENGTH IN	TOTAL WT.	REMARKS	MILL ORDER
1								
2								
3	ONE	A3	W24x76	20	10 3/8	1586		
4	4	a3a	L4x3 1/2 x 5/16	1	2 1/2	37		
5	10	d	3/4 Ø A325 BOLT(S)	0	2 1/4	4		
6								
7								
8	ONE	B3	W24x94	20	10 3/8	1962		
9	4	a3b R/L	L4x3 1/2 x 5/16	1	6 1/2	48	2=R; 2=L	
10	12	d				5		
11								
12								
13	ONE	C3	W16x40	11	10 1/2	476		
14	4	a3c	L4x3 1/2 x 5/16	0	11 1/2	30		
15	8	d				3		
16								
17								
18	ONE	D3	S8x18.4	14	7 1/4	269		
19								
20								
21	ONE	E3	C10x15.3	9	10 1/2	152		
22	1	a3d	L7x4 x 3/8	0	5 1/2	6		
23	1	a3f	L7x4 x 3/8	0	5 1/2	6		
24	1	a3g	L4x3 1/2 x 5/16	0	7 1/2	5		
25	1	a3h	L4x3 1/2 x 5/16	0	7 1/2	5		
26	6	e	3/4 Ø A325 BOLT(S)	0	2	2		
27								
28								
29	ONE	F3	C12x20.7	9	10 1/2	205		
30	1	a3k	L4x3 1/2 x 5/16	0	9 1/2	6		
31	1	a3m	L4x3 1/2 x 5/16	0	9 1/2	6		
32	1	a3n	L4x3 1/2 x 5/16	0	8 1/2	5		
33	1	a3p	L4x3 1/2 x 5/16	0	8 1/2	5		
34	6	d				3		
35								
36								
37	ONE	G3	W10x15	6	4 3/4	96		
38	1	a3s	L4x3 1/2 x 5/16	0	5 1/2	9		
39	1	a3v	L4x3 1/2 x 5/16	0	5 1/2	9		
40	2	a3t R/L	L4x3 1/2 x 5/16	0	6 1/2	4	1=R; 1=L	
41	1	p3a	PL 1/4 x 6	0	9	4		
42	4	d				2		
43	2	f	3/4 Ø A325 BOLT(S)	0	1 3/4	1		
44								
45								
46								
47								
48								

FIELD BOLTS	28- 3/4 Ø A325 x 2
	4- 3/4 Ø A325 x 2 1/2
PAINT	PER SPEC.
MATERIAL:	W Shapes: A992 Other: A36
HOLES	13/16 U/NOTED
ELECTRODES	
TOTAL WEIGHT	4951
BOLTS:	3/4 ØA325N

NO	DATE	DESCRIPTION	NO	DATE	DESCRIPTION

REVISIONS

Figure A1-2

DRAFTER MISD DATE 08-2000	CUSTOMER	ECKS WYE ZEE FABRICATING CO.	CONTRACT NO 3990
CHECKED AISC DATE 08-2000	STRUCTURE	MERCY HOSPITAL	DWG NO 3

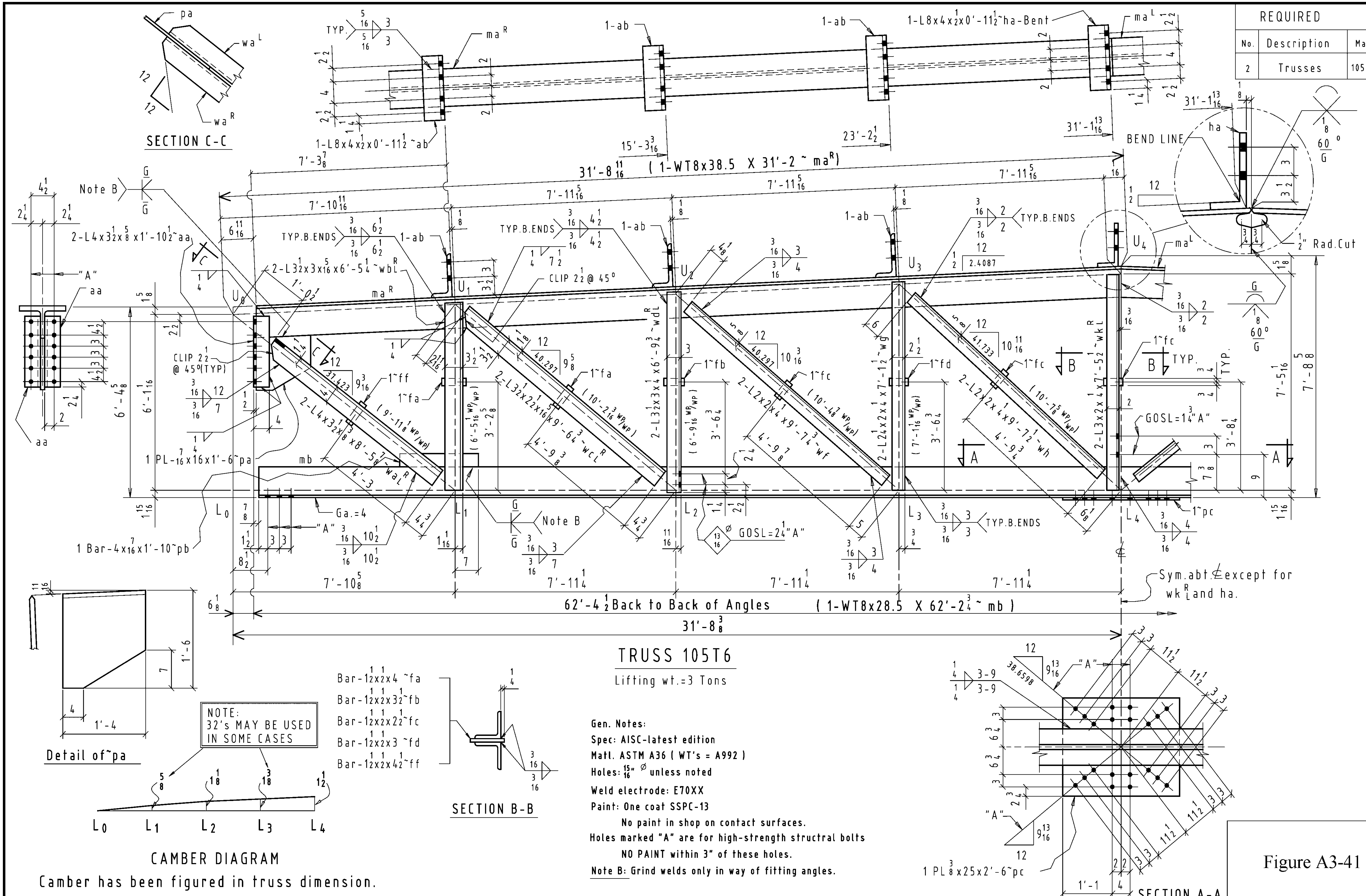
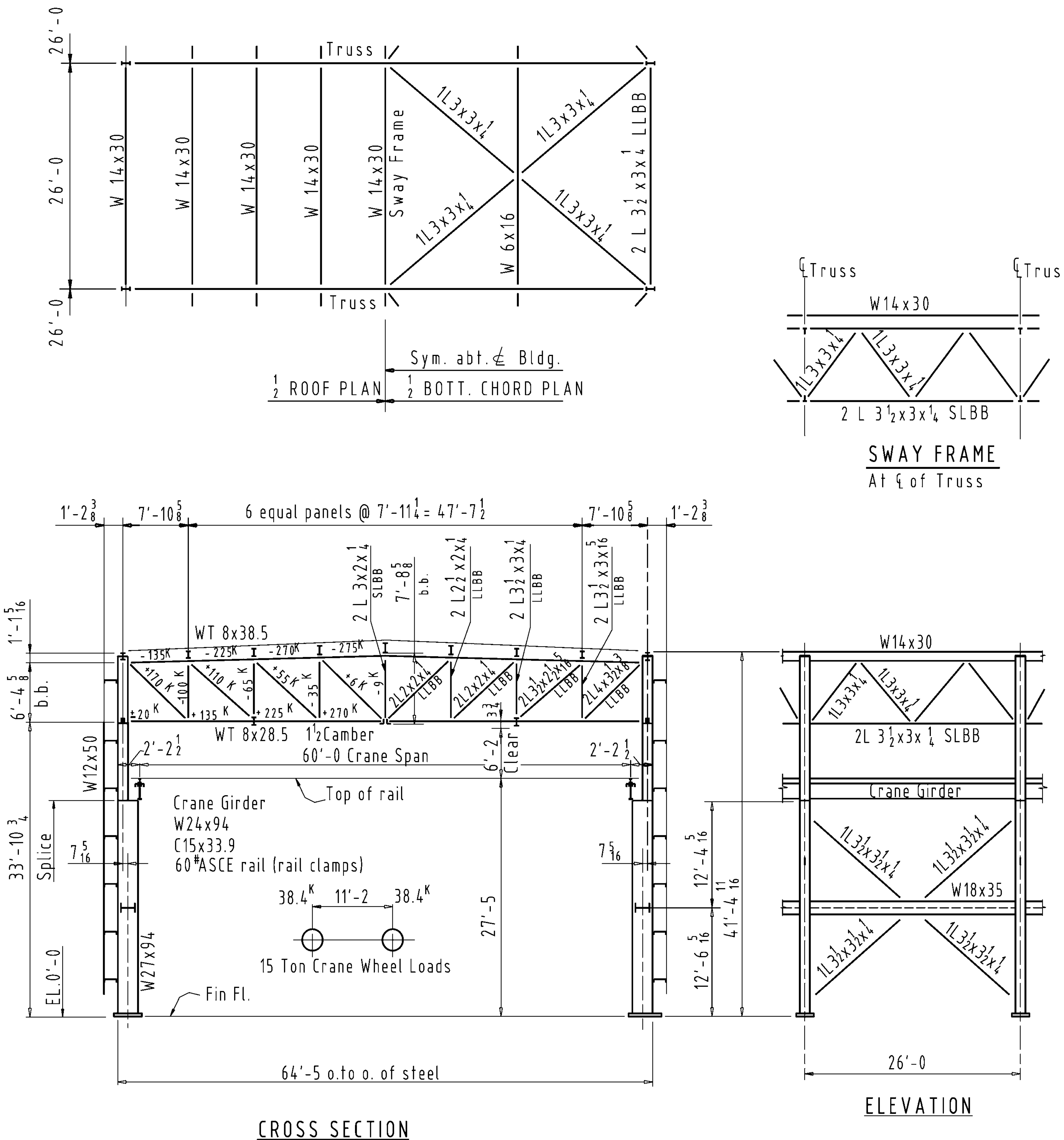
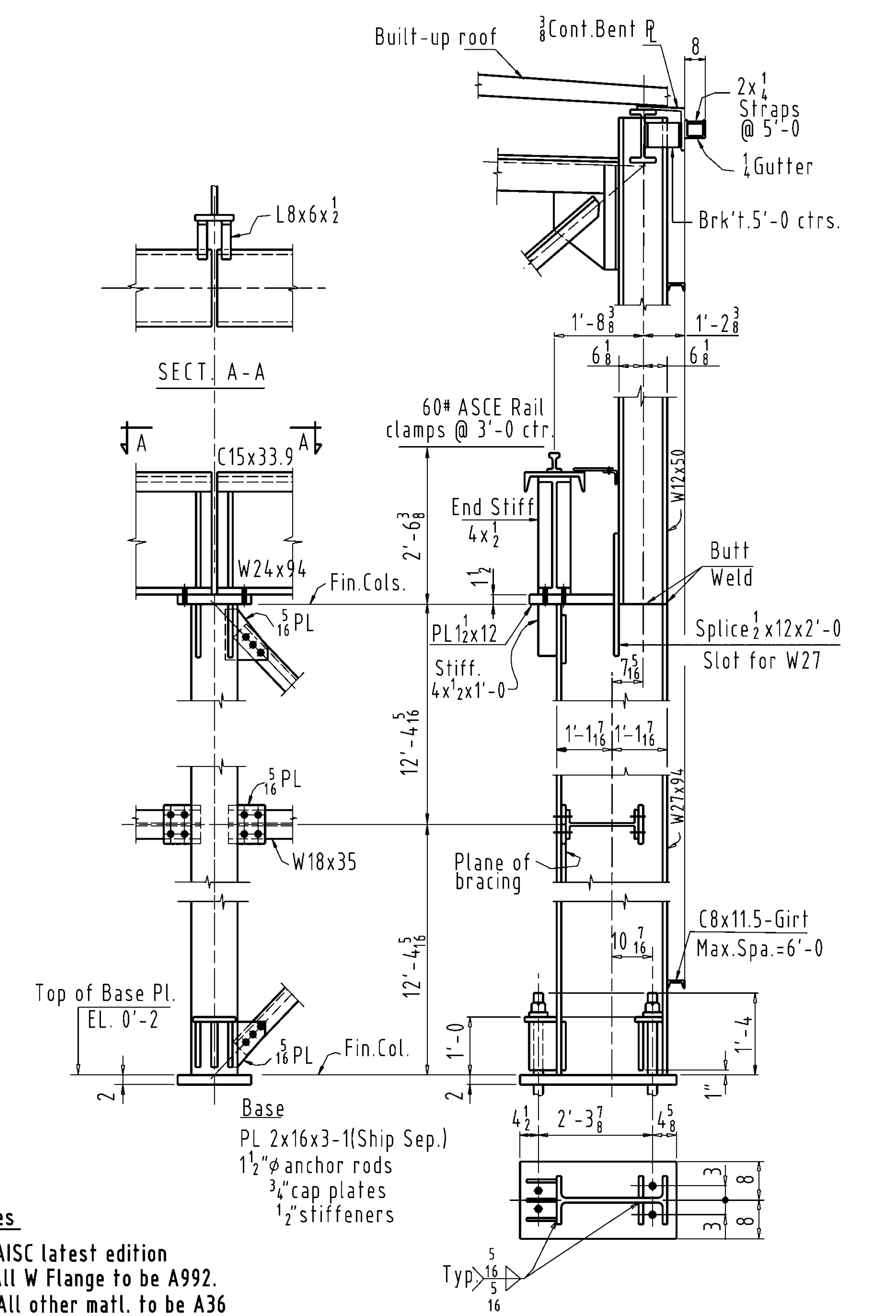

REQUIRED
No. Description Mark
2 Trusses 105T6
SECTION C-C
1-L8x4x1/2x0'-11 1/2~ab
1-L8x4x1/2x0'-11 1/2~ha-Bent
31'-8 11/16 (1-WT8x38.5 X 31'-2 ~ ma^R)
2-L4x3 1/2x5/8x1'-10 1/2~aa
2-L3 1/2x3x5/16x6'-5 1/4~wbL^R
CLIP 2 1/2 @ 45°
1 PL-7/16x16x1'-6~pa
1 Bar-4x7/16x1'-10~pb
2-L4x3 1/2x3/8x8'-5 7/8~wa^L
2-L3 1/2x2 1/2x5/16x9'-6 3/4~wc^R
2-L3 1/2x3x1/4x6'-9 3/4~wd^R
2-L2x2x1/4x9'-7 1/4~wf
2-L2 1/2x2x1/4x7'-1 1/2~wg
2-L2x2x1/4x9'-7 1/2~wh
2-L3x2x1/4x7'-5 1/2~wk^R
BEND LINE
1/2" Rad.Cut
GOSL=1 3/4"A"
GOSL=2 1/4"A"
TYP.B.ENDS
Note B
1~pc
Sym.abt.℄except for wk^R and ha.
62'-4 1/2 Back to Back of Angles (1-WT8x28.5 X 62'-2 3/4 ~ mb)
31'-8 3/8
TRUSS 105T6
Lifting wt.=3 Tons
Detail of~pa
NOTE: 32's MAY BE USED IN SOME CASES
Bar-1 1/2x2x4 ~fa
Bar-1 1/2x2x3 1/2~fb
Bar-1 1/2x2x2 1/2~fc
Bar-1 1/2x2x3 ~fd
Bar-1 1/2x2x4 1/2~ff
SECTION B-B
CAMBER DIAGRAM
Camber has been figured in truss dimension.
Gen. Notes:
Spec: AISC-latest edition
Matl. ASTM A36 (WT's = A992)
Holes: 15/16" Ø unless noted
Weld electrode: E70XX
Paint: One coat SSPC-13
No paint in shop on contact surfaces.
Holes marked "A" are for high-strength structral bolts
NO PAINT within 3" of these holes.
Note B: Grind welds only in way of fitting angles.
1 PL 3/8x25x2'-6~pc
SECTION A-A

Figure A3-41

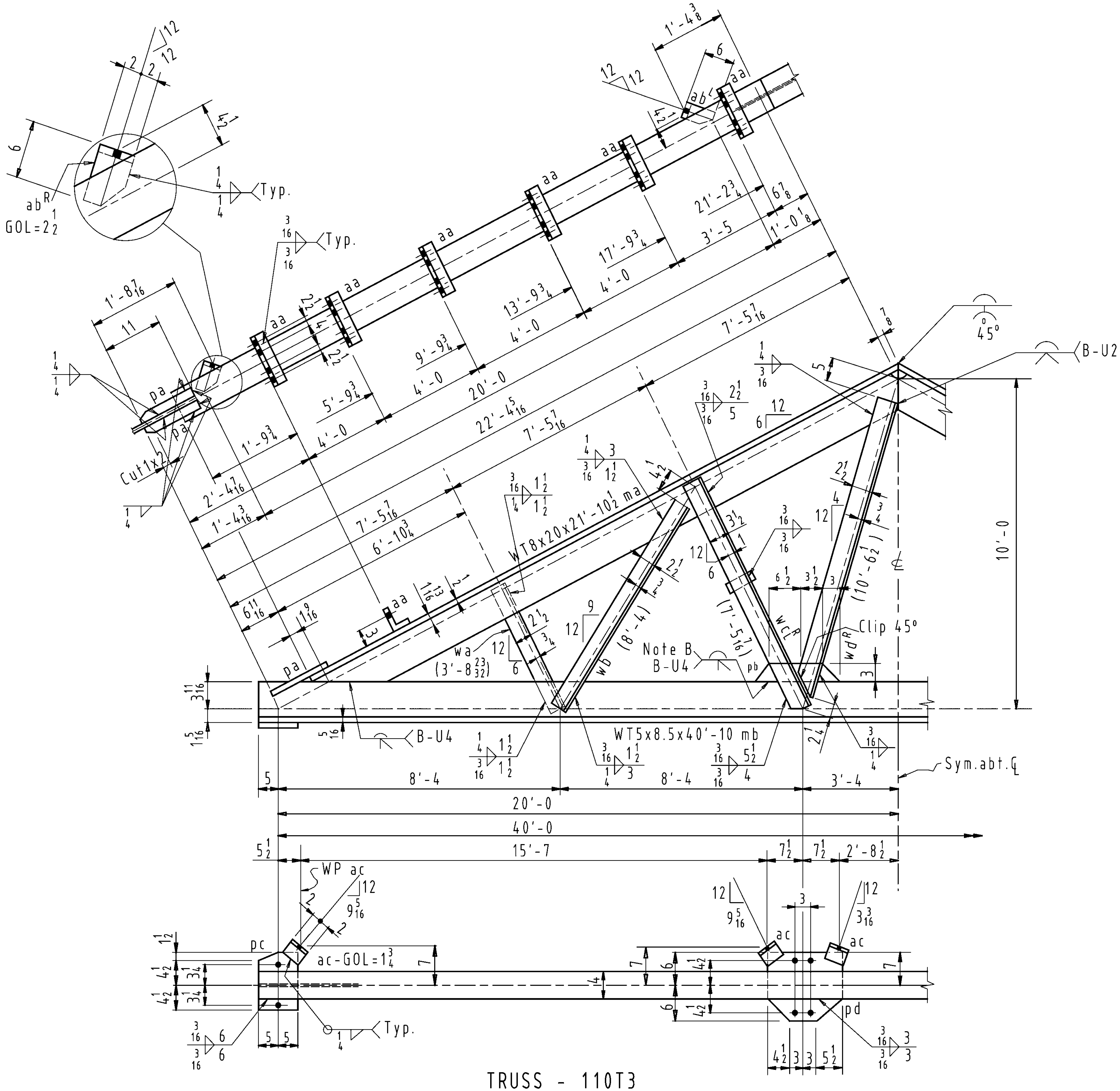




Gen. Notes

Spec. AISC latest edition
Matl: All W Flange to be A992.
All other matl. to be A36
Shop Welds: E70XX
Field Bolds: 7/8 ∅ A325-SC except purlins and girts 7/8 ∅ A325-N.
Bracing: For field connections of single angles use 3 bolts, for double angles use 2 bolts for each angle.

Figure A3-42



QUAN	MARK	DESCRIPTION	LENGTH FT	LENGTH IN	WEIGHT	MILL ORDER No.	REMARKS
ONE	110T3						
2	ma	WT8x20	21	10½			A992
ONE	mb	WT5x8.5	40	10			A992
2	wa	L2½x2x¼	3	9½			
2	wb	L2½x2x¼	7	11½			
4	wc R L	L3½x3½x5/16	7	6¼			2=R2=L
2	wd R L	L2½x2x¼	9	11¼			1=R1=L
4	pa	FL2½x⅜	0	11			
12	aa	L4x3x⅜	1	0			
4	ab R L	L6x4x½	0	4			2=R2=L
2	fa	FL1½x5/16	0	4½			
2	pb	FL3x¼	1	4			
2	pc	PL ½x10	0	10½			
2	pd	PL ⅜x12	1	4			
6	ac	L3x3x⅜	0	4			

Gen. Notes:

Spec : AISC latest edition

Matl : ASTM A36

Holes : 15/16 diam.

Welding : E70XX

Paint : One coat SSPC 13

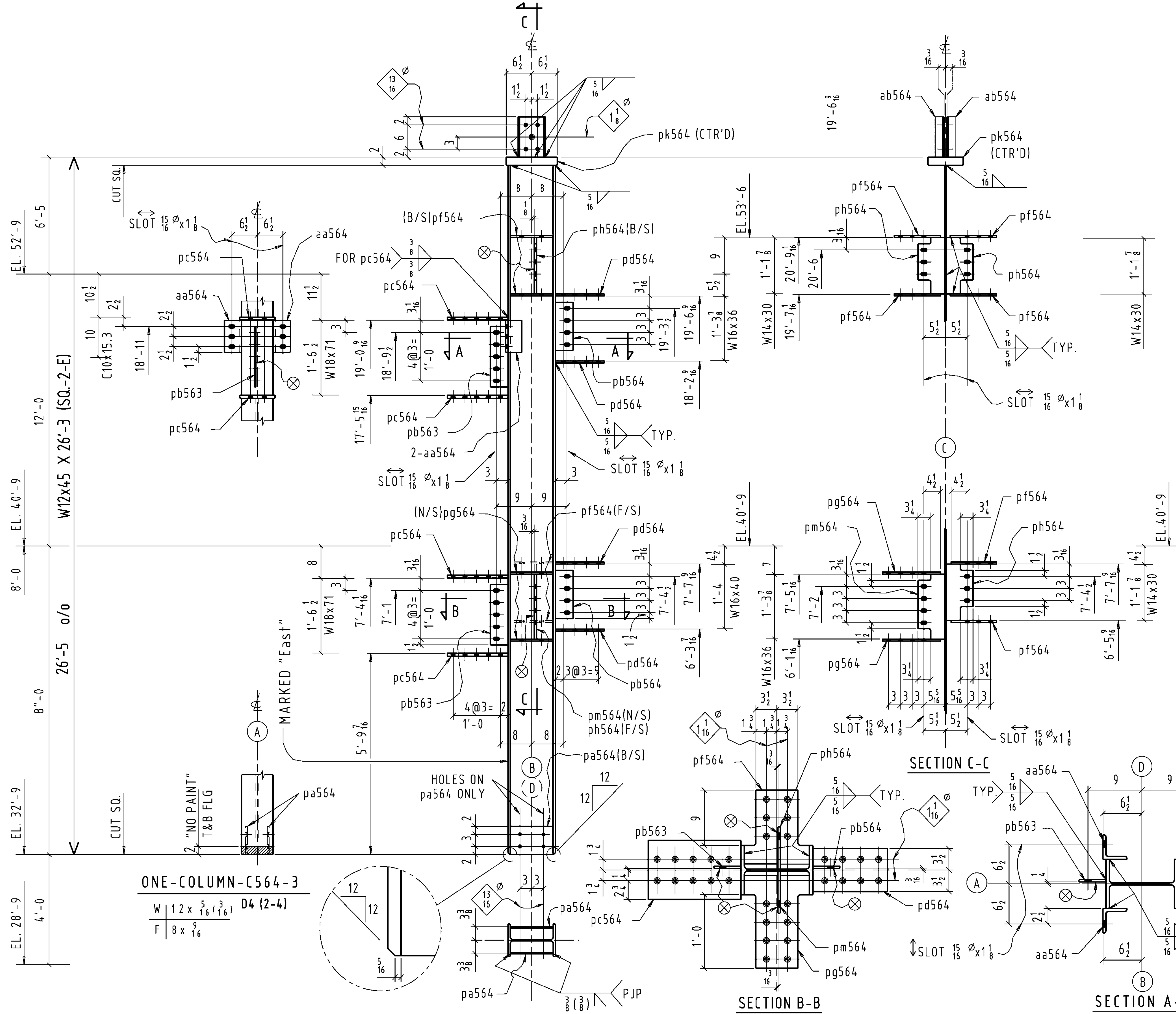
No paint on shop contact surfaces.

Note A : Gouge single U groove after fitting.

Note B : Grind welds only in way of fitting angles.

No Camber.

Figure A3-45

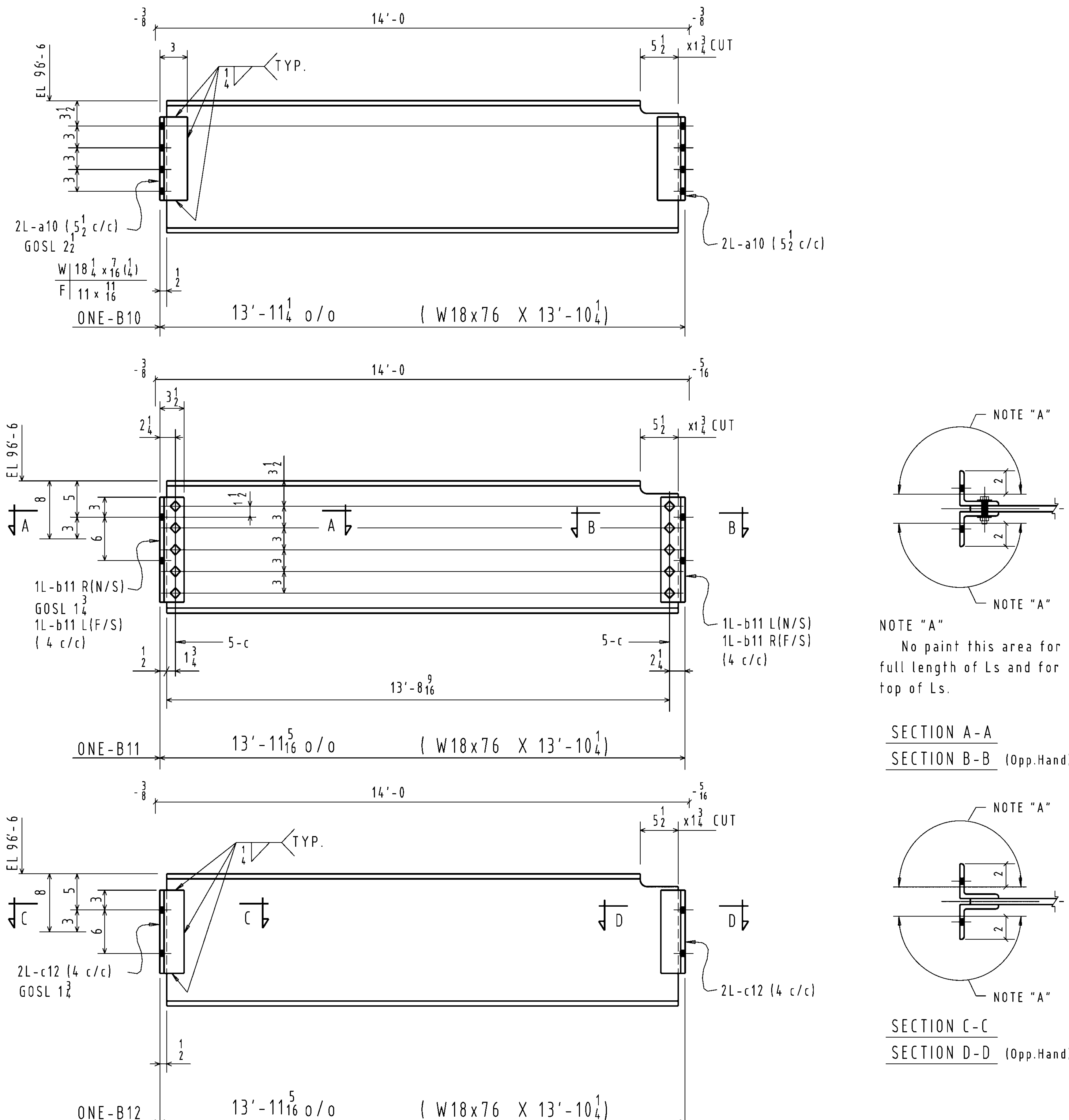


QTY	MARK	SHAPE	LENGTH FT	LENGTH IN	WGHT	GRADE	REM.
ONE	C564-3	W12x45	26	3			SQ-2-E
2	pb563	PL $\frac{3}{8}$x4$\frac{1}{2}$	1	3			
2	aa564	L4x4x$\frac{1}{2}$	0	8			
2	ab564	C7x12.25	0	10			
2	pa564	PL $\frac{1}{2}$x7	0	10$\frac{7}{8}$			C.T.F.
2	pb564	PL $\frac{3}{8}$x4$\frac{1}{2}$	1	0			
4	pc564	PL $\frac{3}{4}$x9	1	3$\frac{1}{2}$			
4	pd564	PL $\frac{1}{2}$x7	1	0$\frac{1}{2}$			
6	pf564	PL $\frac{1}{2}$x10$\frac{7}{8}$	1	0			C.T.F.
2	pg564	PL $\frac{1}{2}$x10$\frac{7}{8}$	1	3			C.T.F.
3	ph564	PL $\frac{3}{8}$x6$\frac{13}{16}$	1	2			
1	pk564	PL2x9	1	1			
1	pm564	PL $\frac{3}{8}$x6$\frac{13}{16}$	1	4			

FIELD BOLTS		
	84 - 1$^{\phi}$ A325 x 2$\frac{1}{2}$	27 - $\frac{7}{8}^{\phi}$ A325 x 2
	40 - 1$^{\phi}$ A325 x 3	10 - $\frac{7}{8}^{\phi}$ A325 x 2$\frac{1}{4}$
		4 - $\frac{3}{4}^{\phi}$ A325 x 2$\frac{1}{4}$

Gen. Notes:
Spec: AISC-latest edition
Matl: W Shape ASTM A992
All other mat'l ASTM A36
Holes: $\frac{15}{16}$" ϕ unless noted
Weld electrode: E70XX
⊗ = STRIPE or Connecting Surface.
Paint: One coat SSPC 13 except where noted "NO PAINT". No paint on shop contact surfaces. Holes marked "◇" are for high strength structural bolts. NO PAINT within 3 inches of these holes.

Figure A4-3



NOTE "A"
No paint this area for full length of Ls and for top of Ls.

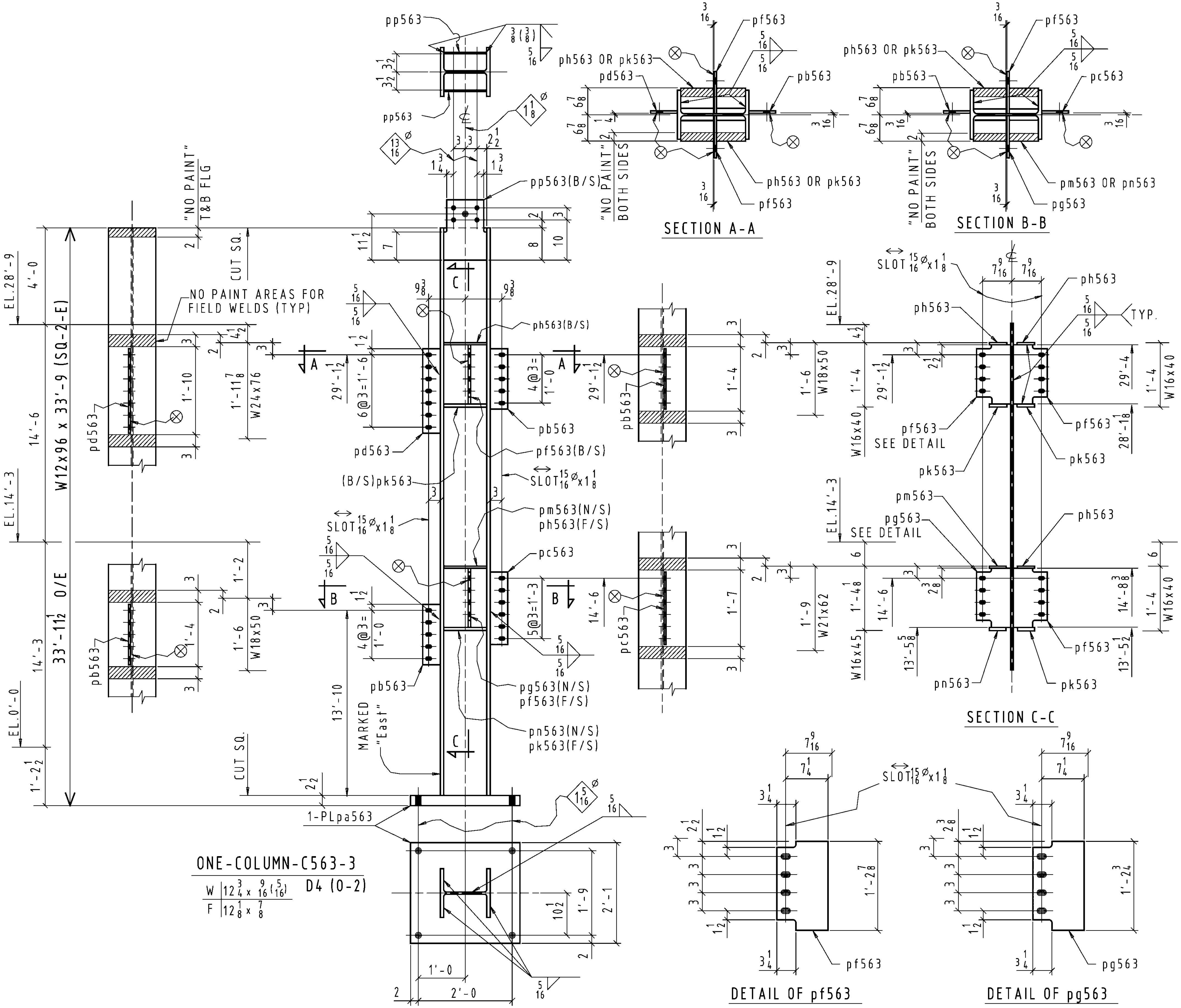
BILL OF MATERIAL

QTY	MARK	SHAPE	LENGTH FT	LENGTH IN	WGHT	GRADE	REM.
ONE	B10	W18x76	13	10 1/4			
4	a10	L4x3x5/16	1	0			
ONE	B11	W18x76	13	10 1/4			
4	b11 R/L	L3 1/2x3x5/16	1	3			2=R;2=L
10	c	7/8 ⌀A325	0	2 1/2			
ONE	B12	W18x76	13	10 1/4			
4	c12	L3x3x5/16	1	0			

FIELD BOLTS

Gen. Notes:
Spec: AISC-latest edition
Matl: W Shape ASTM A992
All other mat'l ASTM A36
Holes: 15/16 "⌀ unless noted
Shop Fasteners: 7/8 "⌀ unless noted
Weld electrode: E70XX
Paint: One coat SSPC 13 except where noted "NO PAINT".

Figure A4-6

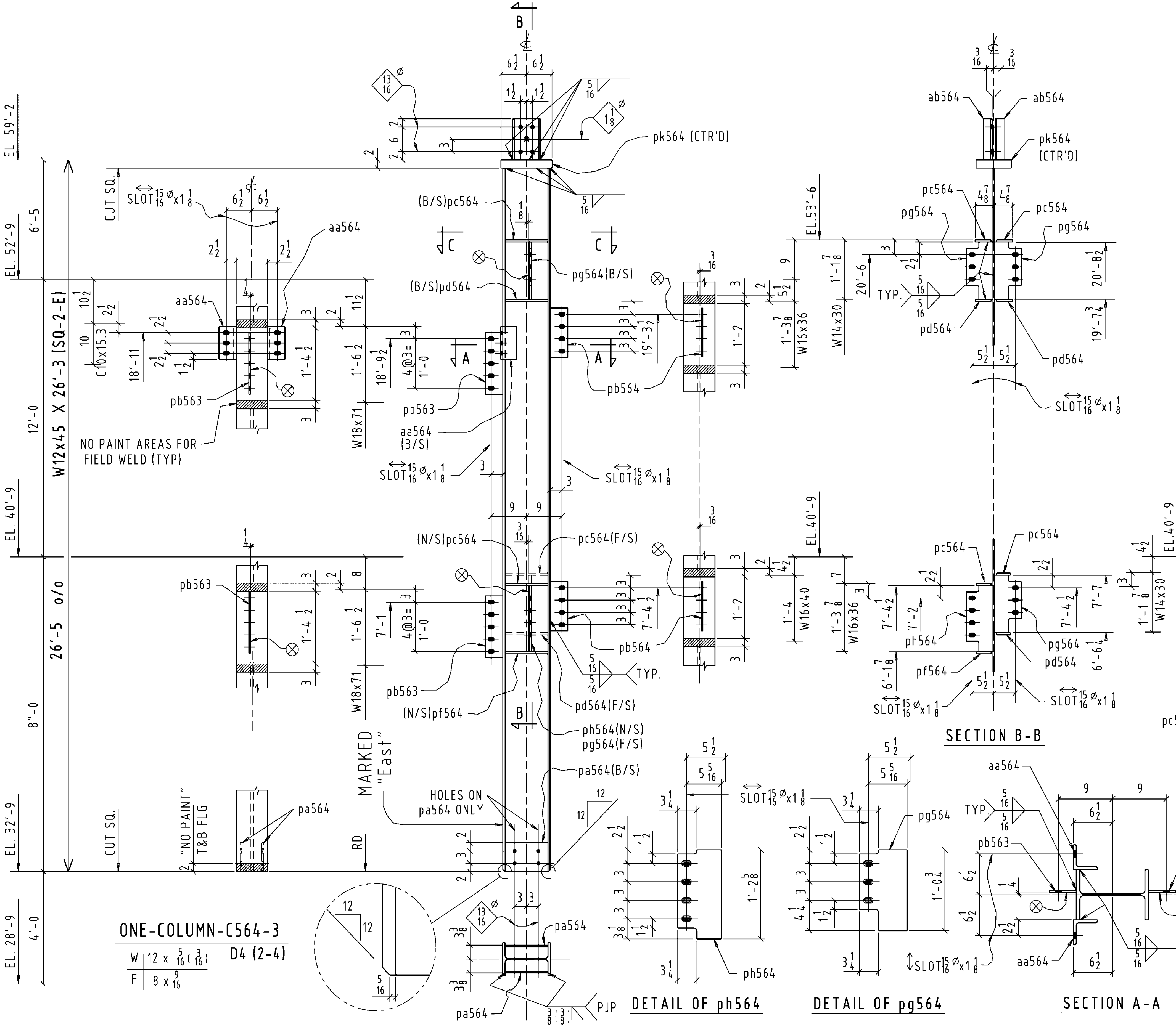


QTY	MARK	SHAPE	LENGTH FT	LENGTH IN	WGHT	GRADE	REM.
ONE	C563-3	W12x96	33	9			SQ-2-E
1	pa563	PL2½ x25	2	4			
2	pb563	PL⅜x4½	1	3			
1	pc563	PL⅜x4½	1	6			
1	pd563	PL⅜x4½	1	9			
3	pf563	PL⅜x8¾	1	2⅞			
1	pg563	PL⅜x8¾	1	2¾			
3	ph563	PL½x6	0	11			C.T.F.
3	pk563	PL¾x6	0	11			C.T.F.
1	pm563	PL⅝x6	0	11			C.T.F.
1	pn563	PL⅞x6	0	11			C.T.F.
2	pp563	PL½x11	1	3			C.T.F.

FIELD BOLTS	
	39 - ⅞ ⌀A325 x 2
	8 - ¾ ⌀A325 x 2¼

Gen. Notes:
Spec: AISC-latest edition
Matl: W Shape ASTM A992
All other mat'l ASTM A36
Holes: $\frac{15}{16}$"⌀ unless noted
Weld electrode: E70XX
⊗ = STRIPE or Connecting Surface.
Paint: One coat SSPC 13 except where noted "NO PAINT". No paint on shop contact surfaces. Holes marked "◇" are for high strength structural bolts. NO PAINT within 3 inches of these holes.

Figure A4-31



QTY	MARK	SHAPE	LENGTH FT	LENGTH IN	WGHT	GRADE	REM.
ONE	C564-3	W12x45	26	3			SQ-2-E
2	pb563	PL $\frac{3}{8}$x4$\frac{1}{2}$	1	3			
2	aa564	L4x4x$\frac{1}{2}$	0	8			
2	ab564	C7x12.25	0	10			
2	pa564	PL $\frac{1}{2}$x7	0	10$\frac{7}{8}$			
2	pb564	PL $\frac{3}{8}$x4$\frac{1}{2}$	1	0			
4	pc564	PL $\frac{1}{2}$x4	0	10$\frac{7}{8}$			C.T.F.
3	pd564	PL $\frac{5}{8}$x4	0	10$\frac{7}{8}$			C.T.F.
1	pf564	PL $\frac{3}{4}$x4	0	10$\frac{7}{8}$			C.T.F.
3	pg564	PL $\frac{3}{8}$x6$\frac{13}{16}$	1	0$\frac{3}{4}$			
1	ph564	PL $\frac{3}{8}$x6$\frac{13}{16}$	1	2$\frac{5}{8}$			
1	pk564	PL9x2	1	1			

FIELD BOLTS: 37 - $\frac{7}{8}$ Ø A325 x 2; 4 - $\frac{3}{4}$ Ø A325 x 2$\frac{1}{4}$

Gen. Notes:
Spec: AISC-latest edition
Matl: W Shape ASTM A992
All other mat'l ASTM A36
Holes: $\frac{15}{16}$"Ø unless noted
Weld electrode: E70XX
⊗ = STRIPE or Connecting Surface.
Paint: One coat SSPC 13 except where noted "NO PAINT". No paint on shop contact surfaces. Holes marked "◇" are for high strength structural bolts. NO PAINT within 3 inches of these holes.

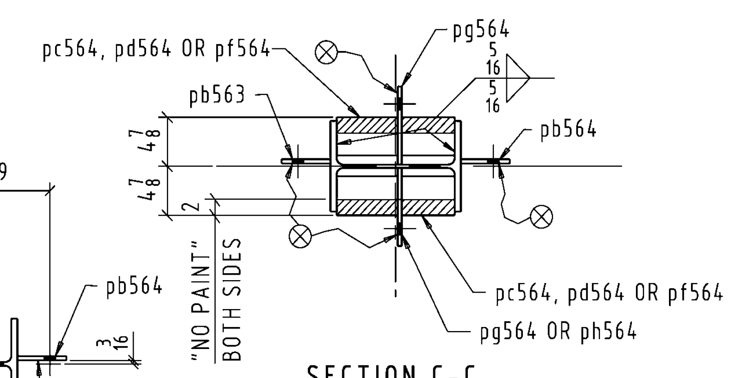
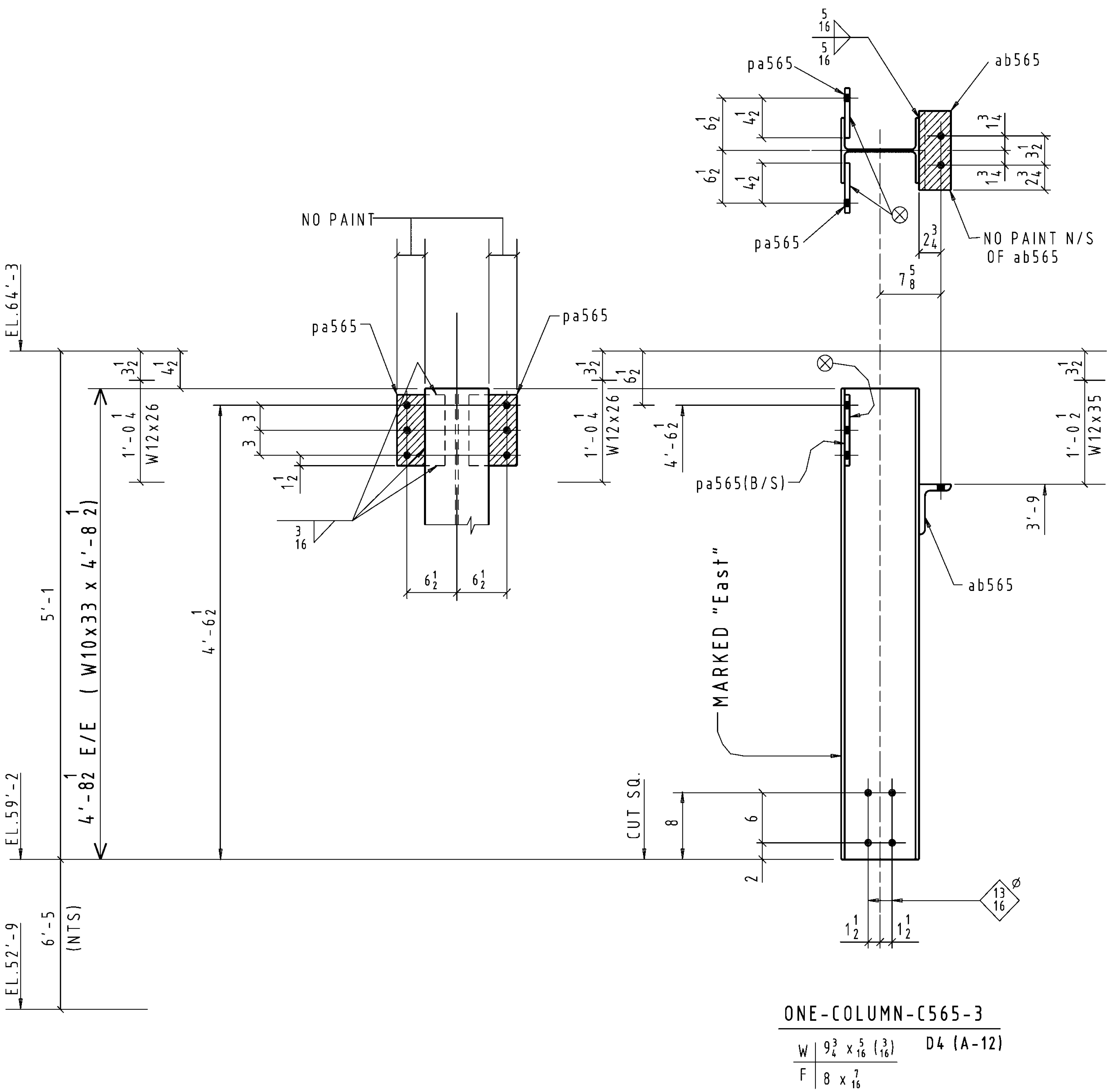


Figure A4-32

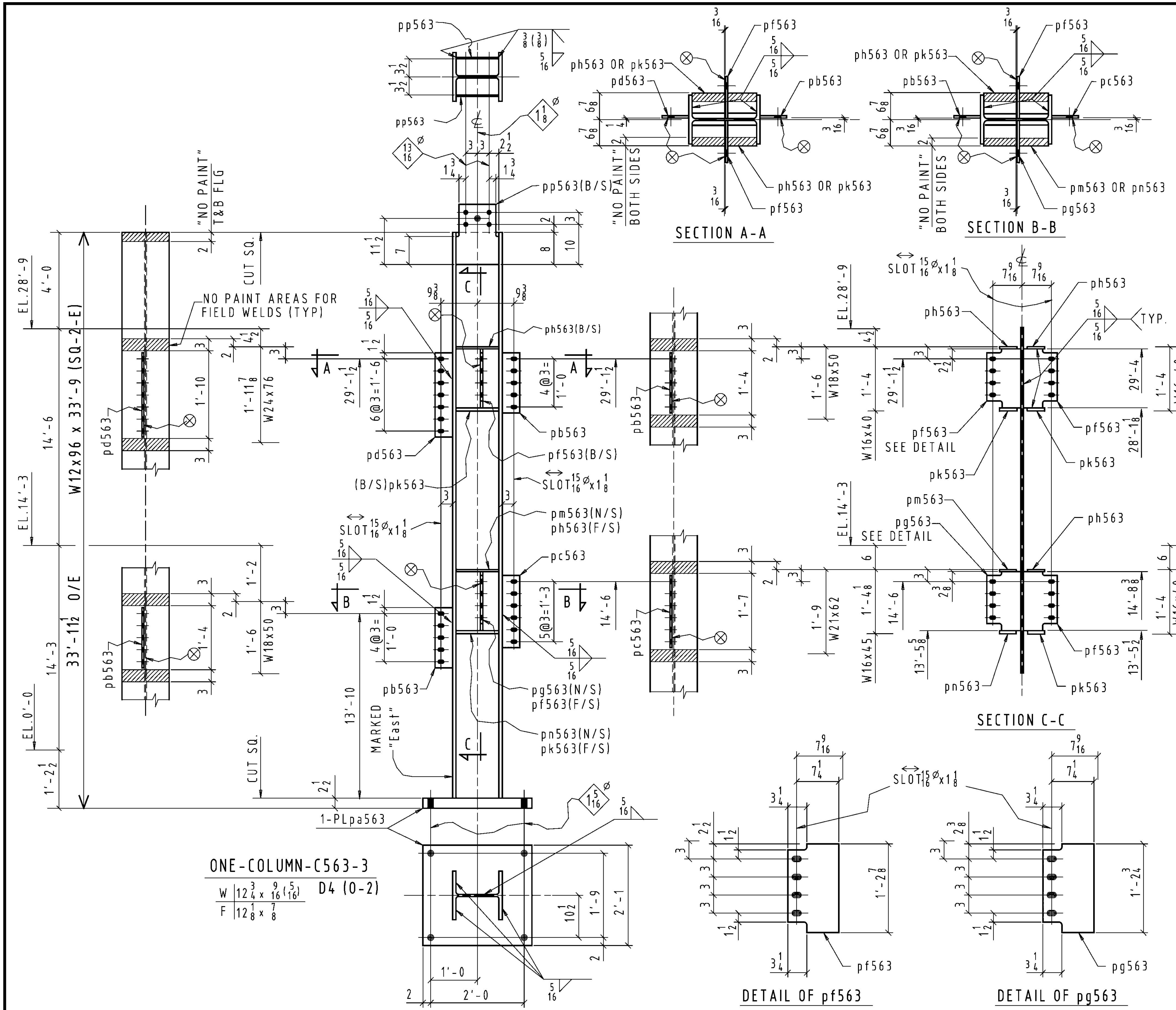


BILL OF MATERIAL							
QTY	MARK	SHAPE	LENGTH FT	LENGTH IN	WGHT	GRADE	REM.
ONE	C565-3	W10x33	4	8 1/2			SQ-1-E
1	ab565	L6x4x3/4	0	9			
2	pa565	PL5/16x6	0	9			

FIELD BOLTS	6 - 7/8 ⌀ A325 x 2	
	2 - 7/8 ⌀ A325 x 2 1/2	

Gen. Notes:
Spec: AISC-latest edition
Matl: W Shape ASTM A992
All other mat'l ASTM A36
Holes: 15/16 "⌀ unless noted
Weld electrode: E70XX
⊗ = STRIPE or Connecting Surface.
Paint: One coat SSPC 13 except where noted "NO PAINT". No paint on shop contact surfaces. Holes marked "◇" are for high strength structural bolts. NO PAINT within 3 inches of these holes.

Figure A4-33



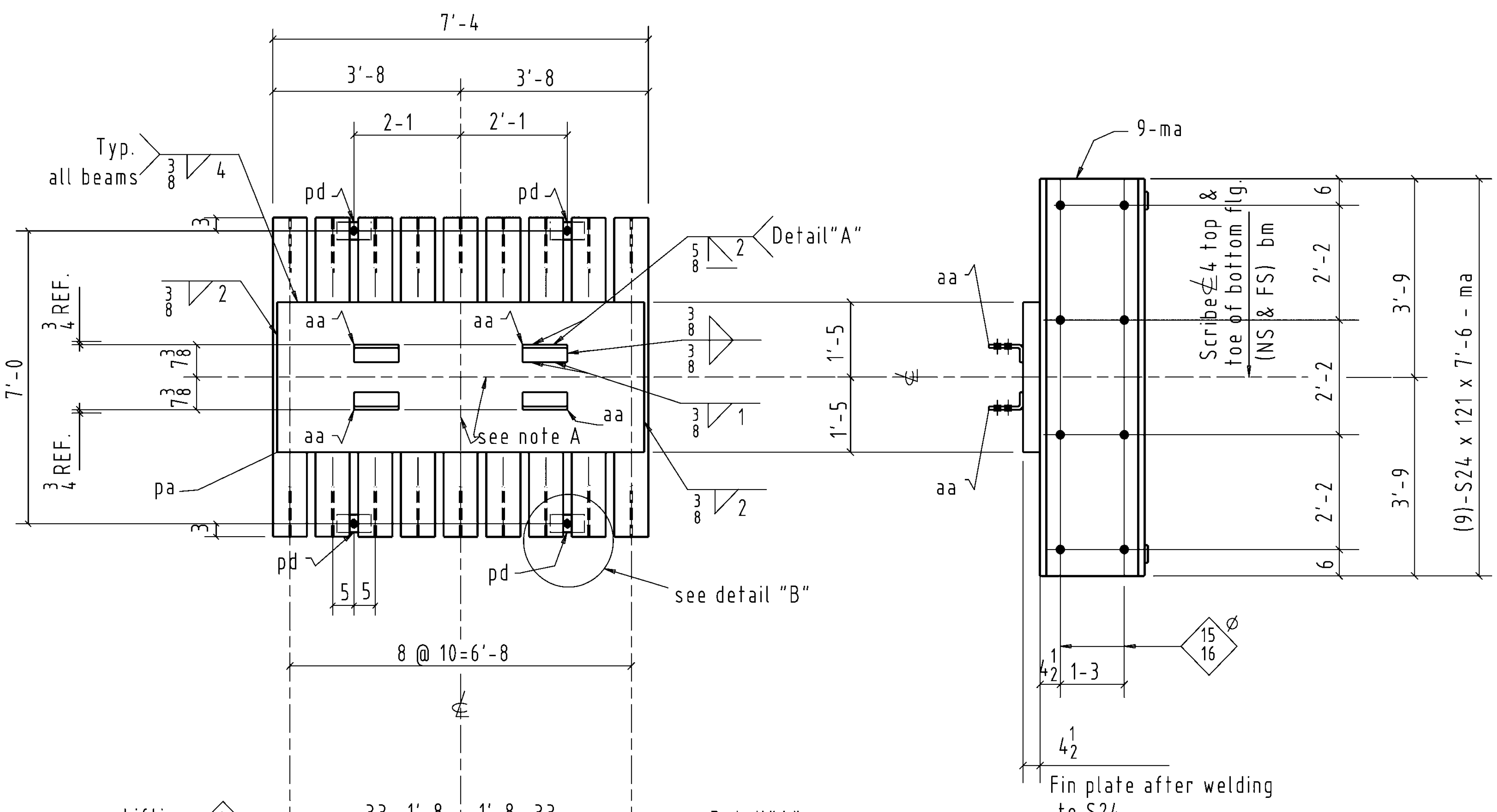
BILL OF MATERIAL

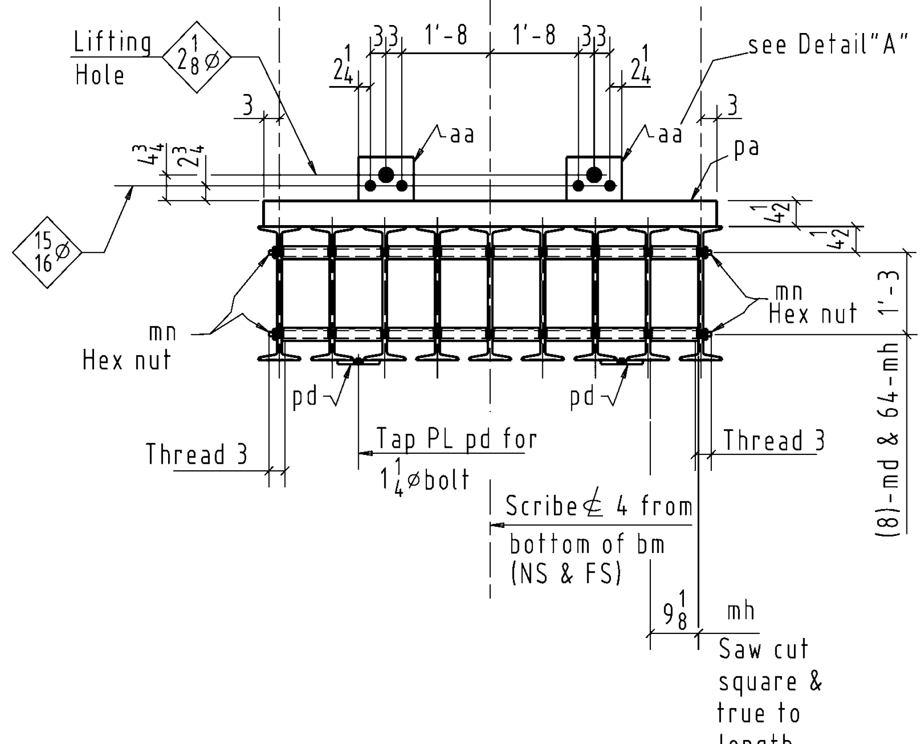
QTY	MARK	SHAPE	LENGTH FT	LENGTH IN	WGHT	GRADE	REM.
ONE	C563-3	W12x96	33	9			SQ-2-E
1	pa563	PL2½ x25	2	4			
2	pb563	PL⅜x4½	1	3			
1	pc563	PL⅜x4½	1	6			
1	pd563	PL⅜x4½	1	9			
3	pf563	PL ⅜x8¾	1	2⅞			
1	pg563	PL ⅜x8¾	1	2¾			
3	ph563	PL½x6	0	11			C.T.F.
3	pk563	PL¾x6	0	11			C.T.F.
1	pm563	PL⅝x6	0	11			C.T.F.
1	pn563	PL⅞x6	0	11			C.T.F.
2	pp563	PL½x11	1	3			C.T.F.

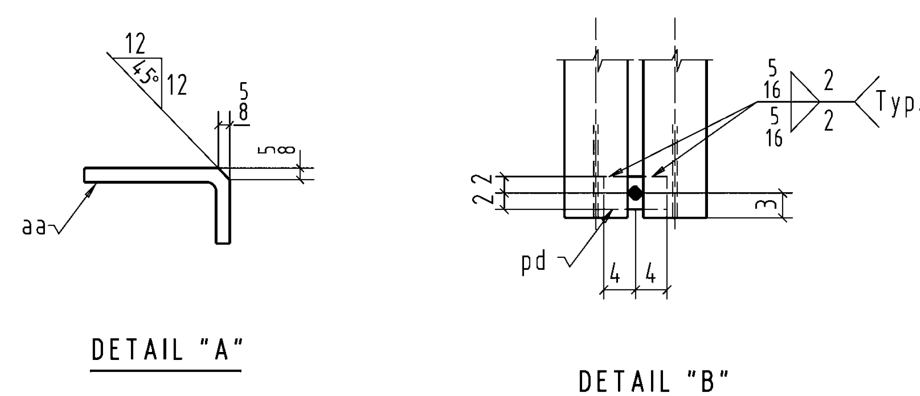
FIELD BOLTS	39 - ⅞ ⌀A325 x 2
	8 - ¾ ⌀A325 x 2¼

Gen. Notes:
Spec: AISC-latest edition
Matl: W Shape ASTM A992
All other mat'l ASTM A36
Holes: $\frac{15}{16}$"⌀ unless noted
Weld electrode: E70XX
⊗ = STRIPE or Connecting Surface.
Paint: One coat SSPC 13 except where noted "NO PAINT". No paint on shop contact surfaces. Holes marked "◇" are for high strength structural bolts. NO PAINT within 3 inches of these holes.

Figure A6-3







3 REQ'D - GRILLAGE - G81 - L.W.T.=6 TONS

BILL OF MATERIAL

QTY	MARK	SHAPE	LENGTH FT	LENGTH IN	WGHT	GRADE	REM.
3	1G81	GRILLAGE					
27	ma	S24x121	7	6			
3	pa	PL 4 1/2 x 34	7	2			
12	aa	L8x4x 3/4	0	10 1/2			
12	pd	PL 3/4 x4	0	8			
24	md	ROD 3/4 ⌀	7	0			
192	mh	PIPE 1⌀STD	0	9 1/8			T2E
48	mn	3/4 ⌀ Hex nut					CS2E

FIELD BOLTS

Note A:
Shop to scribe ℄ in both directions across top of plates and full depth on all 4 edges.

Figure A7-8

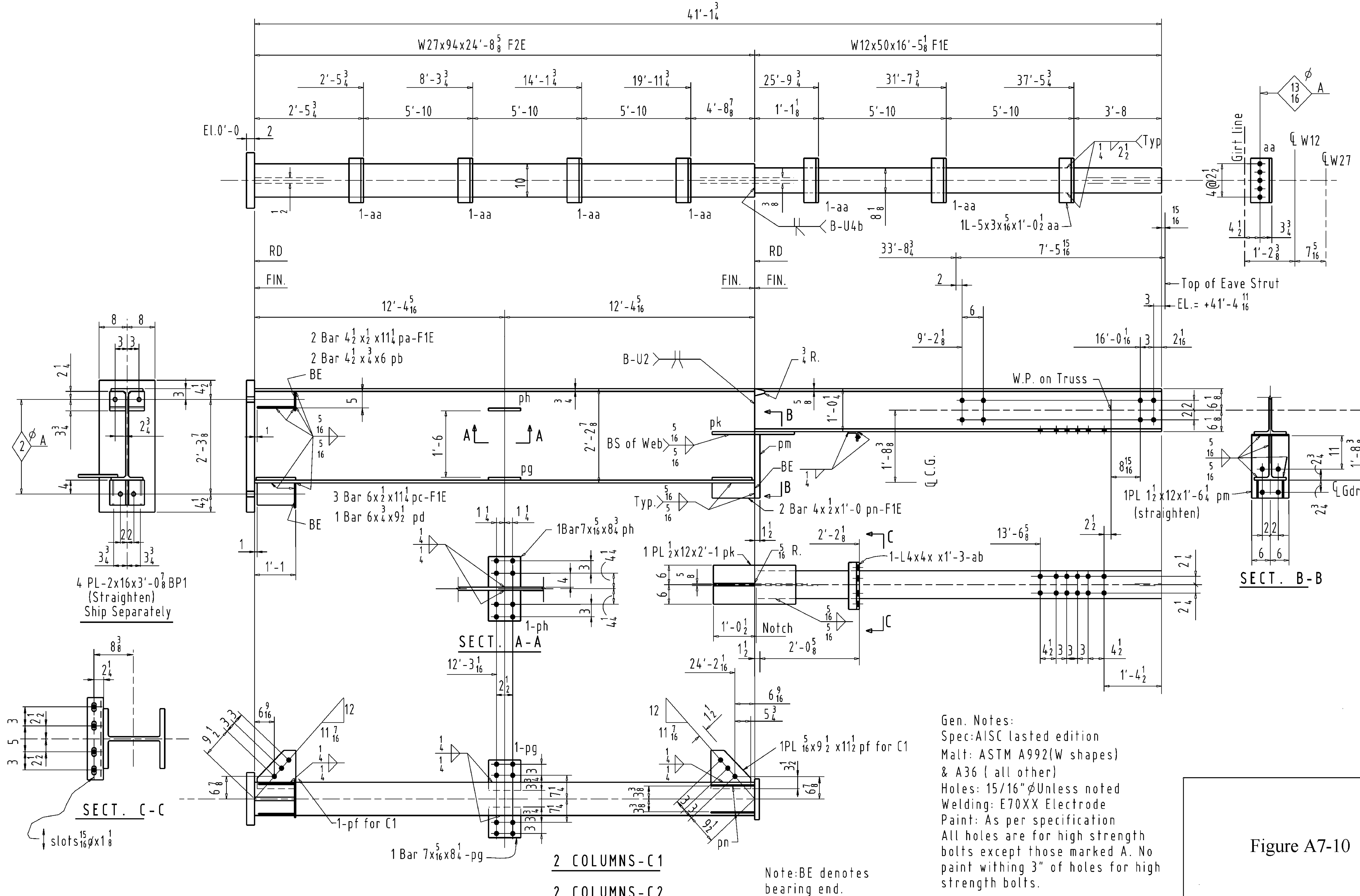
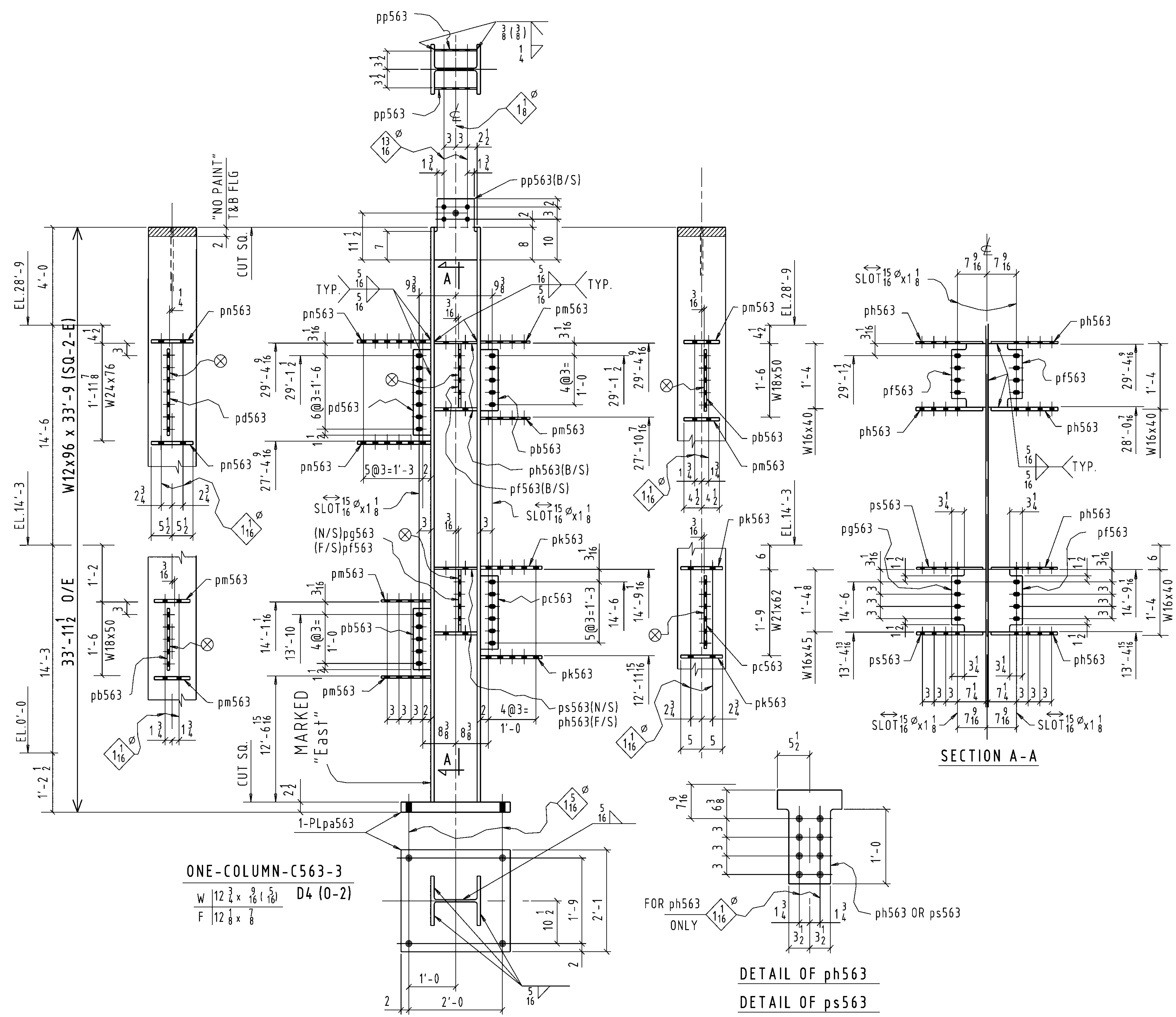

W27x94x24'-8 5/8 F2E
W12x50x16'-5 1/8 F1E
41'-1 3/4
1L-5x3x5/16x1'-0 1/2 aa
2 Bar 4 1/2 x 1/2 x 11 1/4 pa-F1E
2 Bar 4 1/2 x 3/4 x 6 pb
3 Bar 6 x 1/2 x 11 1/4 pc-F1E
1 Bar 6 x 3/4 x 9 1/2 pd
2 Bar 4 x 1/2 x 1'-0 pn-F1E
1 PL 1/2 x 12 x 2'-1 pk
1-L4x4x x1'-3-ab
1PL 1 1/2 x 12 x 1'-6 1/4 pm
(straighten)
1PL 5/16 x 9 1/2 x 11 1/2 pf for C1
1 Bar 7 x 5/16 x 8 1/4 -pg
1Bar 7 x 5/16 x 8 3/4 ph
4 PL-2x16x3'-0 7/8 BP1
(Straighten)
Ship Separately
Top of Eave Strut
EL.= +41'-4 11/16
W.P. on Truss
Girt line
SECT. A-A
SECT. B-B
SECT. C-C
2 COLUMNS-C1
2 COLUMNS-C2
Note:BE denotes bearing end.
Gen. Notes:
Spec:AISC lasted edition
Malt: ASTM A992(W shapes)
& A36 (all other)
Holes: 15/16"ϕUnless 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



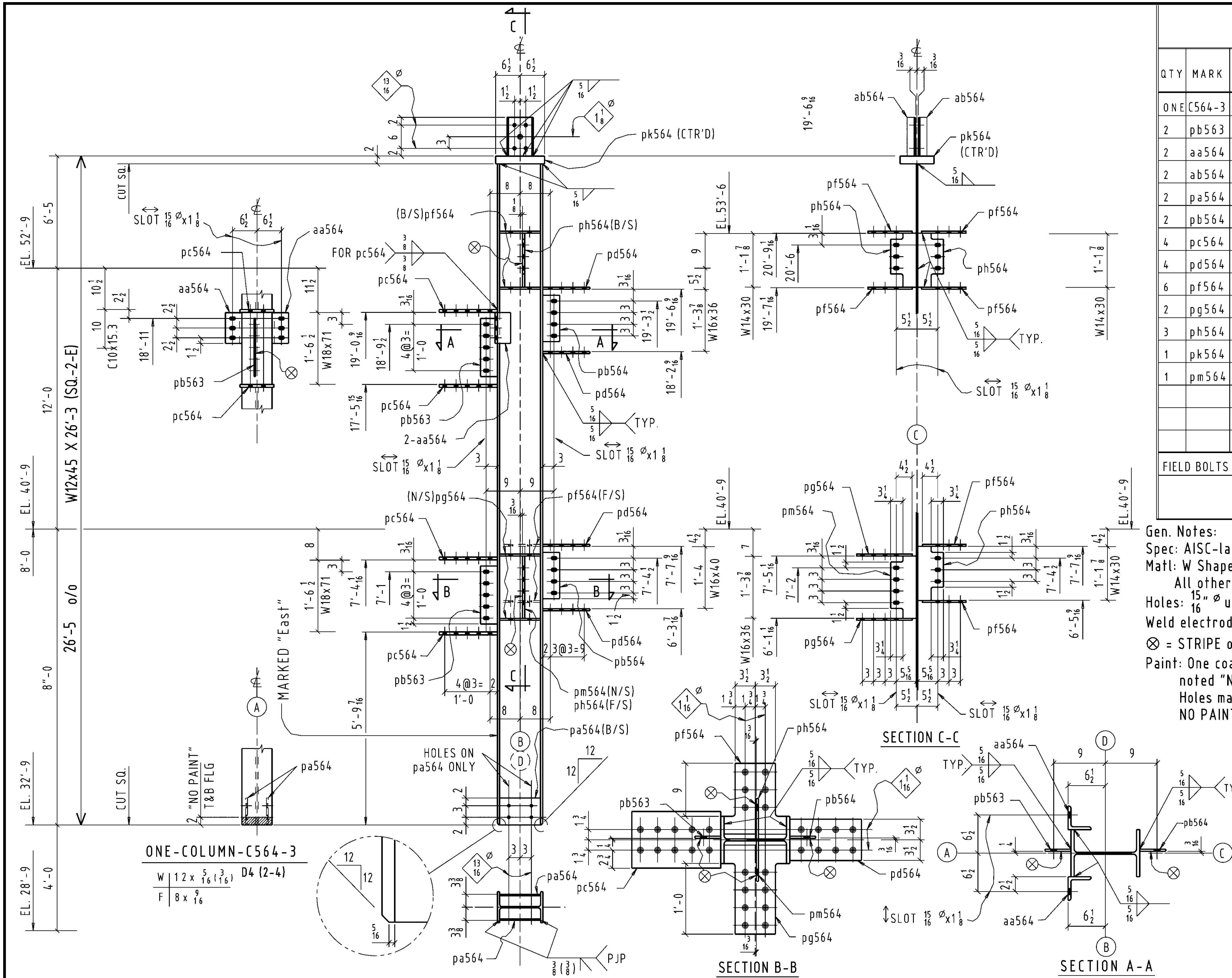
BILL OF MATERIAL

QTY	MARK	SHAPE	LENGTH FT	LENGTH IN	WGHT	GRADE	REM.
ONE	C563-3	W12x96	33	9			SQ-2-E
1	pa563	PL$2\frac{1}{2}$x25	2	4			
2	pb563	PL$\frac{3}{8}$x$4\frac{1}{2}$	1	3			
1	pc563	PL$\frac{3}{8}$x$4\frac{1}{2}$	1	6			
1	pd563	PL$\frac{3}{8}$x$4\frac{1}{2}$	1	9			
3	pf563	PL$\frac{3}{8}$x$8\frac{3}{4}$	1	$4\frac{1}{8}$			
1	pg563	PL$\frac{3}{8}$x$8\frac{3}{4}$	1	$4\frac{1}{4}$			
6	ph563	PL$\frac{1}{2}$x11	1	$4\frac{7}{8}$			C.T.F.
2	pk563	PL$\frac{3}{4}$x10	1	$3\frac{1}{2}$			
4	pm563	PL$\frac{1}{2}$x9	1	$0\frac{1}{2}$			
2	pn563	PL$\frac{3}{4}$x11	1	$6\frac{1}{2}$			
2	pp563	PL$\frac{1}{2}$x11	1	3			C.T.F.
2	ps563	PL$\frac{1}{2}$x11	1	$4\frac{7}{8}$			C.T.F.

FIELD BOLTS		
	96 - 1 ØA325 x $2\frac{1}{2}$	12 - $\frac{7}{8}$ØA325 x 2
	44 - 1 ØA325 x 3	27 - $\frac{7}{8}$ØA325 x $2\frac{1}{4}$
		8 - $\frac{3}{4}$ØA325 x $2\frac{1}{4}$

Gen. Notes:
Spec: AISC-latest edition
Matl: W Shape ASTM A992
All other mat'l ASTM A36
Holes: $\frac{15}{16}$"Ø unless noted
Weld electrode: E70XX
⊗ = STRIPE or Connecting Surface.
Paint: One coat SSPC 13 except where noted "NO PAINT". No paint on shop contact surfaces. Holes marked "◇" are for high strength structural bolts. NO PAINT within 3 inches of these holes.

Figure A7-11



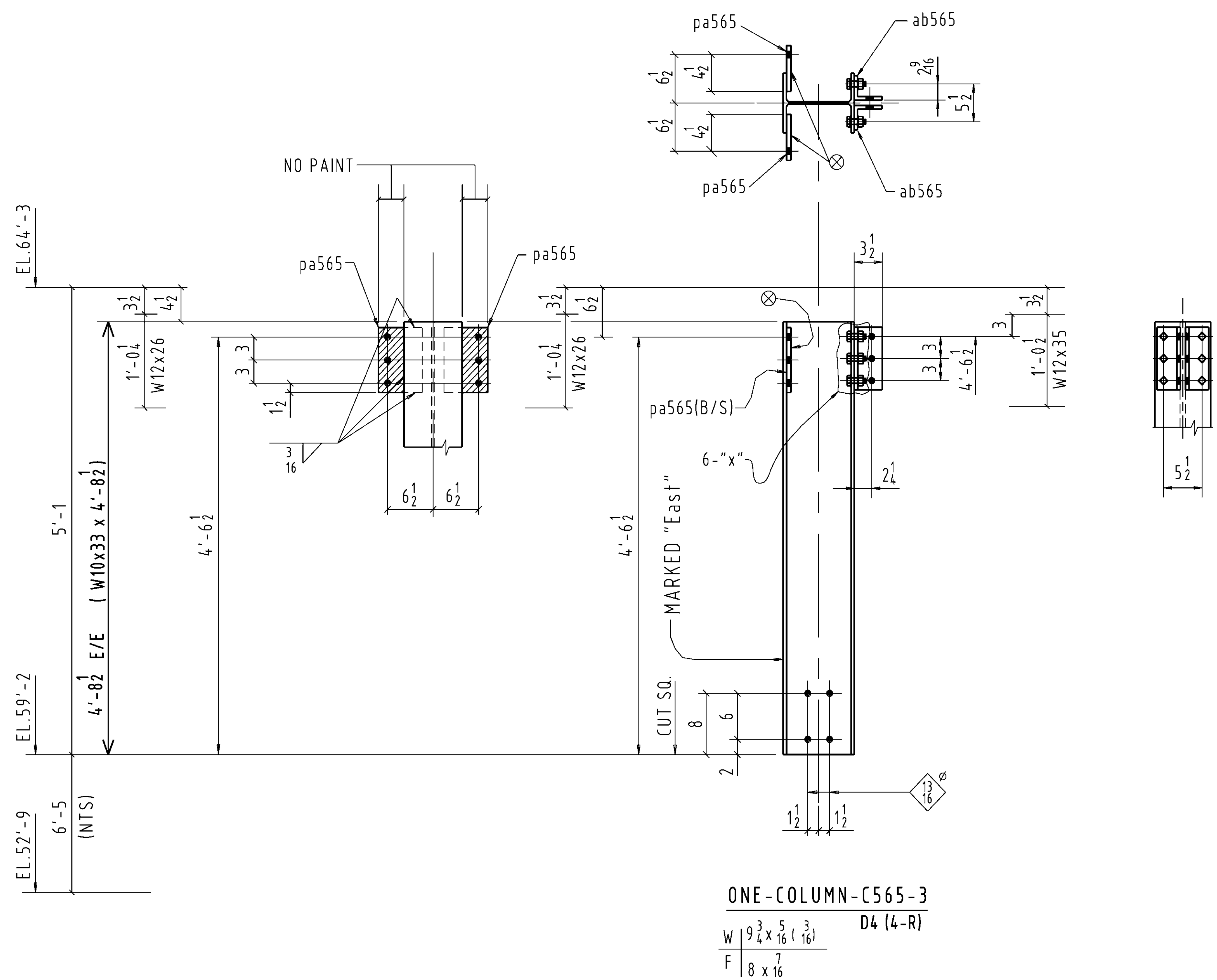
BILL OF MATERIAL

QTY	MARK	SHAPE	LENGTH FT	LENGTH IN	WGHT	GRADE	REM.
ONE	C564-3	W12x45	26	3			SQ-2-E
2	pb563	PL $\frac{3}{8}$x4$\frac{1}{2}$	1	3			
2	aa564	L4x4x$\frac{1}{2}$	0	8			
2	ab564	C7x12.25	0	10			
2	pa564	PL $\frac{1}{2}$x7	0	10$\frac{7}{8}$			C.T.F.
2	pb564	PL $\frac{3}{8}$x4$\frac{1}{2}$	1	0			
4	pc564	PL $\frac{3}{4}$x9	1	3$\frac{1}{2}$			
4	pd564	PL $\frac{1}{2}$x7	1	0$\frac{1}{2}$			
6	pf564	PL $\frac{1}{2}$x10$\frac{7}{8}$	1	0			C.T.F.
2	pg564	PL $\frac{1}{2}$x10$\frac{7}{8}$	1	3			C.T.F.
3	ph564	PL$\frac{3}{8}$ x6$\frac{13}{16}$	1	2			
1	pk564	PL2x9	1	1			
1	pm564	PL$\frac{3}{8}$ x6$\frac{13}{16}$	1	4			

FIELD BOLTS		
	84 - 1$^{\varnothing}$ A325 x 2$\frac{1}{2}$	27 - $\frac{7}{8}^{\varnothing}$ A325 x 2
	40 - 1$^{\varnothing}$ A325 x 3	10 - $\frac{7}{8}^{\varnothing}$ A325 x 2$\frac{1}{4}$
		4 - $\frac{3}{4}^{\varnothing}$ A325 x 2$\frac{1}{4}$

Gen. Notes:
Spec: AISC-latest edition
Matl: W Shape ASTM A992
All other mat'l ASTM A36
Holes: $\frac{15}{16}$" $\varnothing$ unless noted
Weld electrode: E70XX
⊗ = STRIPE or Connecting Surface.
Paint: One coat SSPC 13 except where noted "NO PAINT". No paint on shop contact surfaces. Holes marked "◇" are for high strength structural bolts. NO PAINT within 3 inches of these holes.

Figure A7-12



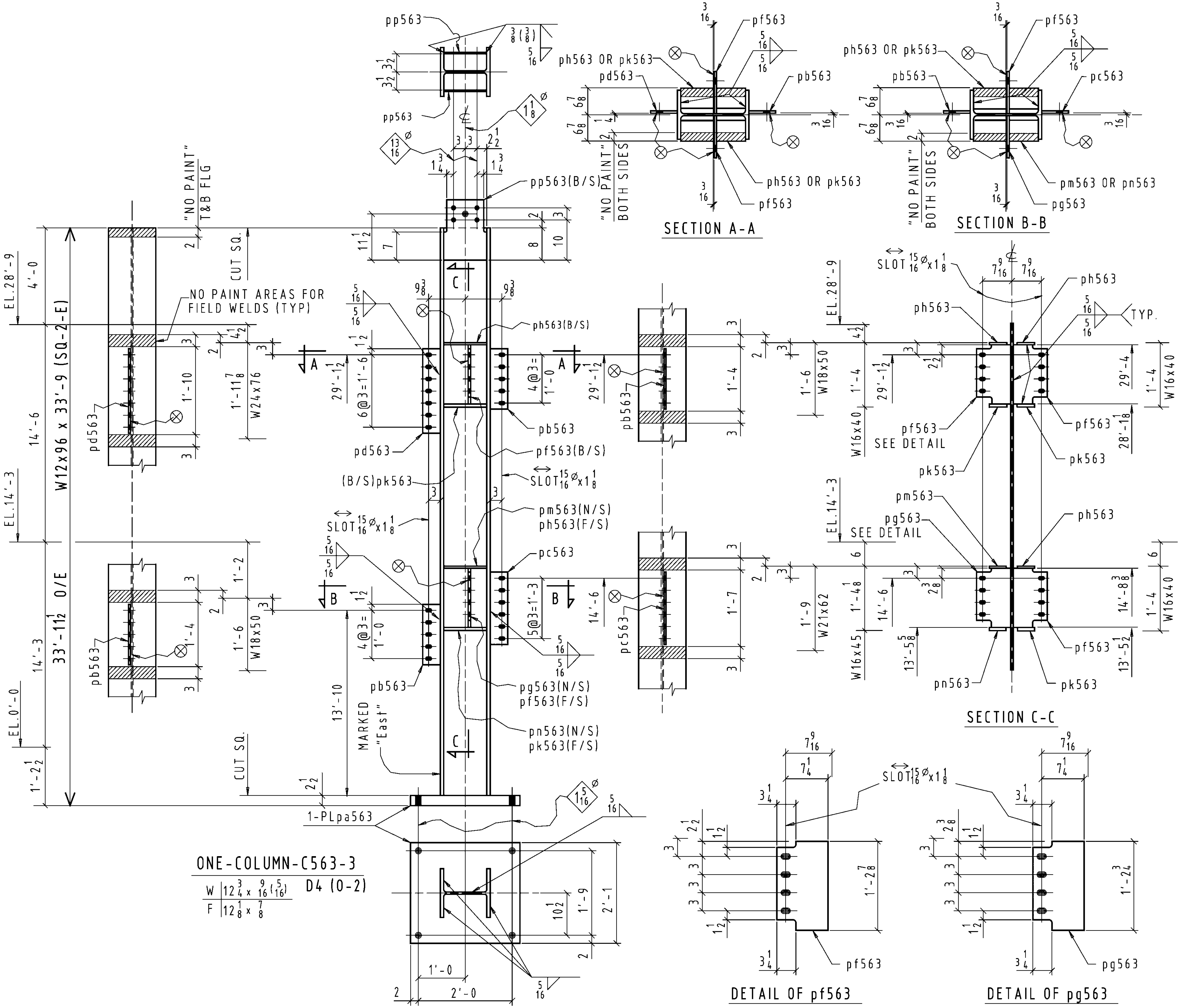
BILL OF MATERIAL

QTY	MARK	SHAPE	LENGTH FT	LENGTH IN	WGHT	GRADE	REM.
ONE	C565-3	W10x33	4	8½			SQ-1-E
2	ab565	L4x3½ x 5/16	0	9			
2	pa565	PL 5/16 x6	0	9			
6	"x"	7/8 ∅ A325	0	2			

FIELD BOLTS	
	6 - 7/8 ∅ A325 x 2
	3 - 7/8 ∅ A325 x 2 ¼

Gen. Notes:
Spec: AISC-latest edition
Matl: W Shape ASTM A992
All other mat'l ASTM A36
Holes: 15/16" ∅ unless noted
Weld electrode: E70XX
⊗ = STRIPE or Connecting Surface.
Paint: One coat SSPC 13 except where noted "NO PAINT". No paint on shop contact surfaces. Holes marked "◇" are for high strength structural bolts. NO PAINT within 3 inches of these holes.

Figure A7-16

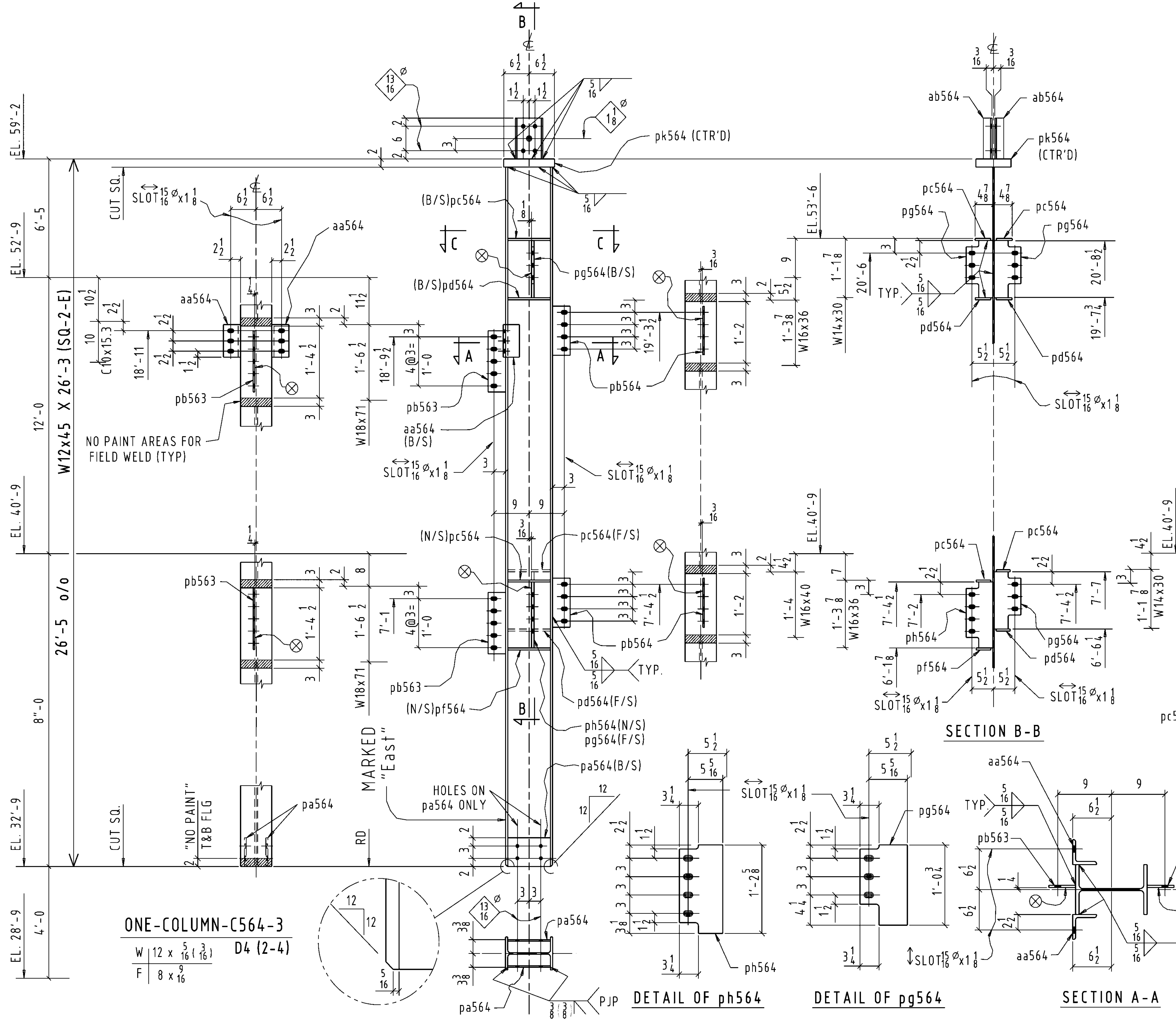


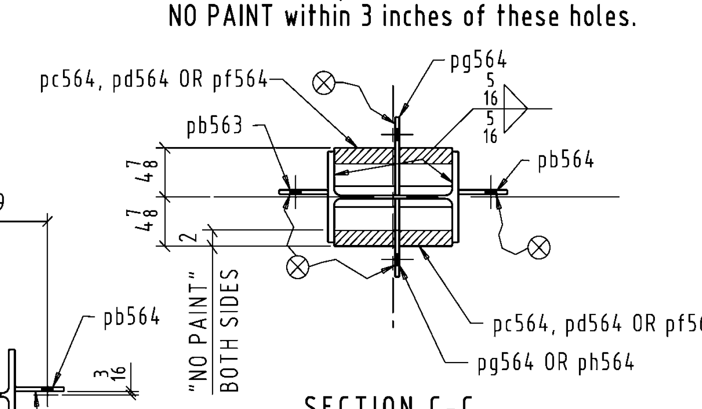
BILL OF MATERIAL

QTY	MARK	SHAPE	LENGTH FT	LENGTH IN	WGHT	GRADE	REM.
ONE	C563-3	W12x96	33	9			SQ-2-E
1	pa563	PL$2\frac{1}{2}$x25	2	4			
2	pb563	PL$\frac{3}{8}$x$4\frac{1}{2}$	1	3			
1	pc563	PL$\frac{3}{8}$x$4\frac{1}{2}$	1	6			
1	pd563	PL$\frac{3}{8}$x$4\frac{1}{2}$	1	9			
3	pf563	PL$\frac{3}{8}$x$8\frac{3}{4}$	1	$2\frac{7}{8}$			
1	pg563	PL$\frac{3}{8}$x$8\frac{3}{4}$	1	$2\frac{3}{4}$			
3	ph563	PL$\frac{1}{2}$x6	0	11			C.T.F.
3	pk563	PL$\frac{3}{4}$x6	0	11			C.T.F.
1	pm563	PL$\frac{5}{8}$x6	0	11			C.T.F.
1	pn563	PL$\frac{7}{8}$x6	0	11			C.T.F.
2	pp563	PL$\frac{1}{2}$x11	1	3			C.T.F.
FIELD BOLTS	39 - $\frac{7}{8}$ØA325 x 2						
	8 - $\frac{3}{4}$ØA325 x $2\frac{1}{4}$						

Gen. Notes:
Spec: AISC-latest edition
Matl: W Shape ASTM A992
All other mat'l ASTM A36
Holes: $\frac{15}{16}$"Ø unless noted
Weld electrode: E70XX
⊗ = STRIPE or Connecting Surface.
Paint: One coat SSPC 13 except where noted "NO PAINT". No paint on shop contact surfaces. Holes marked "◇" are for high strength structural bolts. NO PAINT within 3 inches of these holes.

Figure A7-18





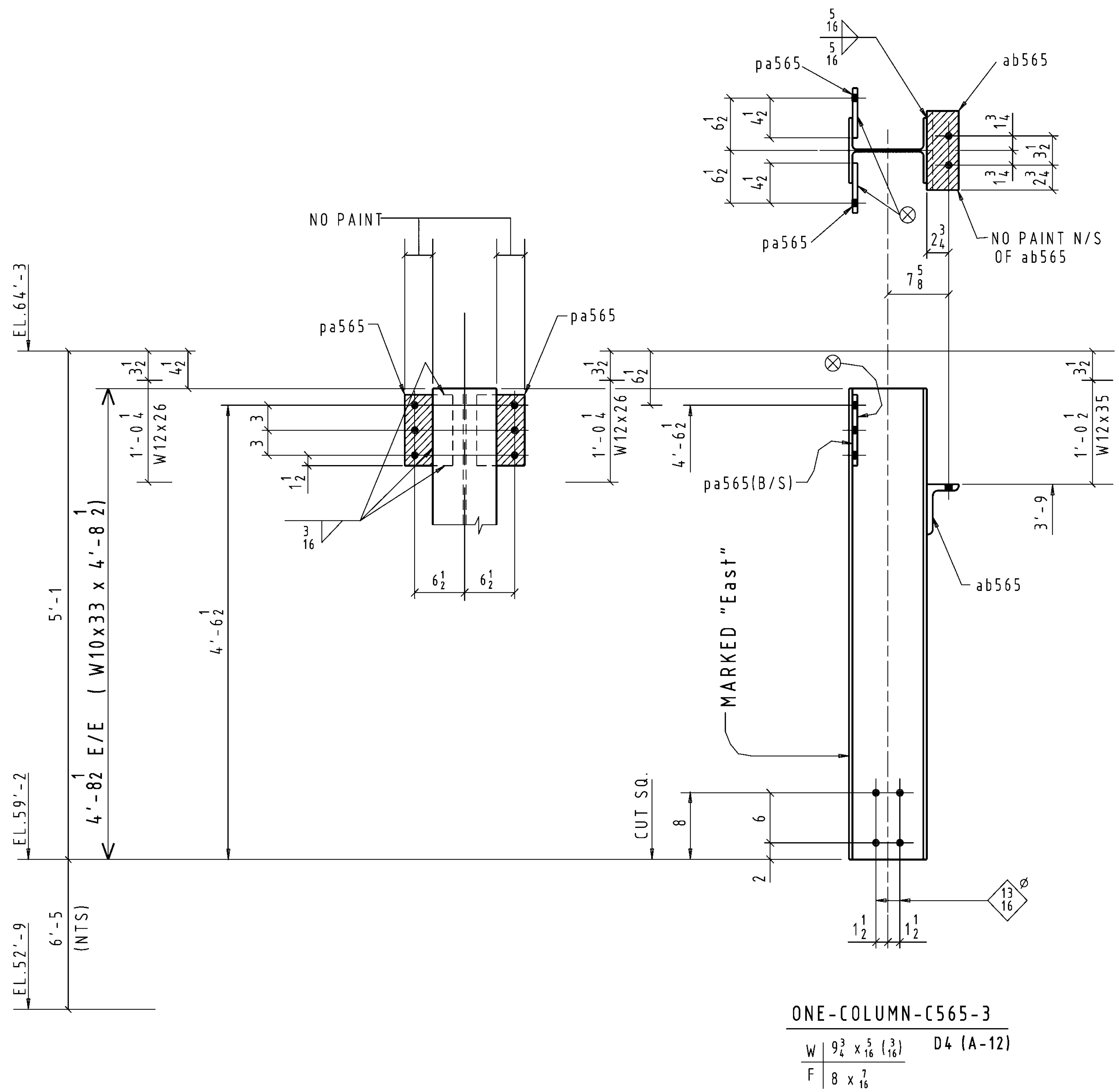
BILL OF MATERIAL

QTY	MARK	SHAPE	LENGTH FT	LENGTH IN	WGHT	GRADE	REM.
ONE	C564-3	W12x45	26	3			SQ-2-E
2	pb563	PL $\frac{3}{8}$x4$\frac{1}{2}$	1	3			
2	aa564	L4x4x$\frac{1}{2}$	0	8			
2	ab564	C7x12.25	0	10			
2	pa564	PL $\frac{1}{2}$x7	0	10$\frac{7}{8}$			
2	pb564	PL $\frac{3}{8}$x4$\frac{1}{2}$	1	0			
4	pc564	PL $\frac{1}{2}$x4	0	10$\frac{7}{8}$			C.T.F.
3	pd564	PL $\frac{5}{8}$x4	0	10$\frac{7}{8}$			C.T.F.
1	pf564	PL $\frac{3}{4}$x4	0	10$\frac{7}{8}$			C.T.F.
3	pg564	PL $\frac{3}{8}$x6$\frac{13}{16}$	1	0$\frac{3}{4}$			
1	ph564	PL $\frac{3}{8}$x6$\frac{13}{16}$	1	2$\frac{5}{8}$			
1	pk564	PL9x2	1	1			

FIELD BOLTS	37 - $\frac{7}{8}$ ⌀ A325 x 2
	4 - $\frac{3}{4}$ ⌀ A325 x 2$\frac{1}{4}$

Gen. Notes:
Spec: AISC-latest edition
Matl: W Shape ASTM A992
All other mat'l ASTM A36
Holes: $\frac{15}{16}$"⌀ unless noted
Weld electrode: E70XX
⊗ = STRIPE or Connecting Surface.
Paint: One coat SSPC 13 except where noted "NO PAINT". No paint on shop contact surfaces. Holes marked "◇" are for high strength structural bolts. NO PAINT within 3 inches of these holes.

Figure A7-19



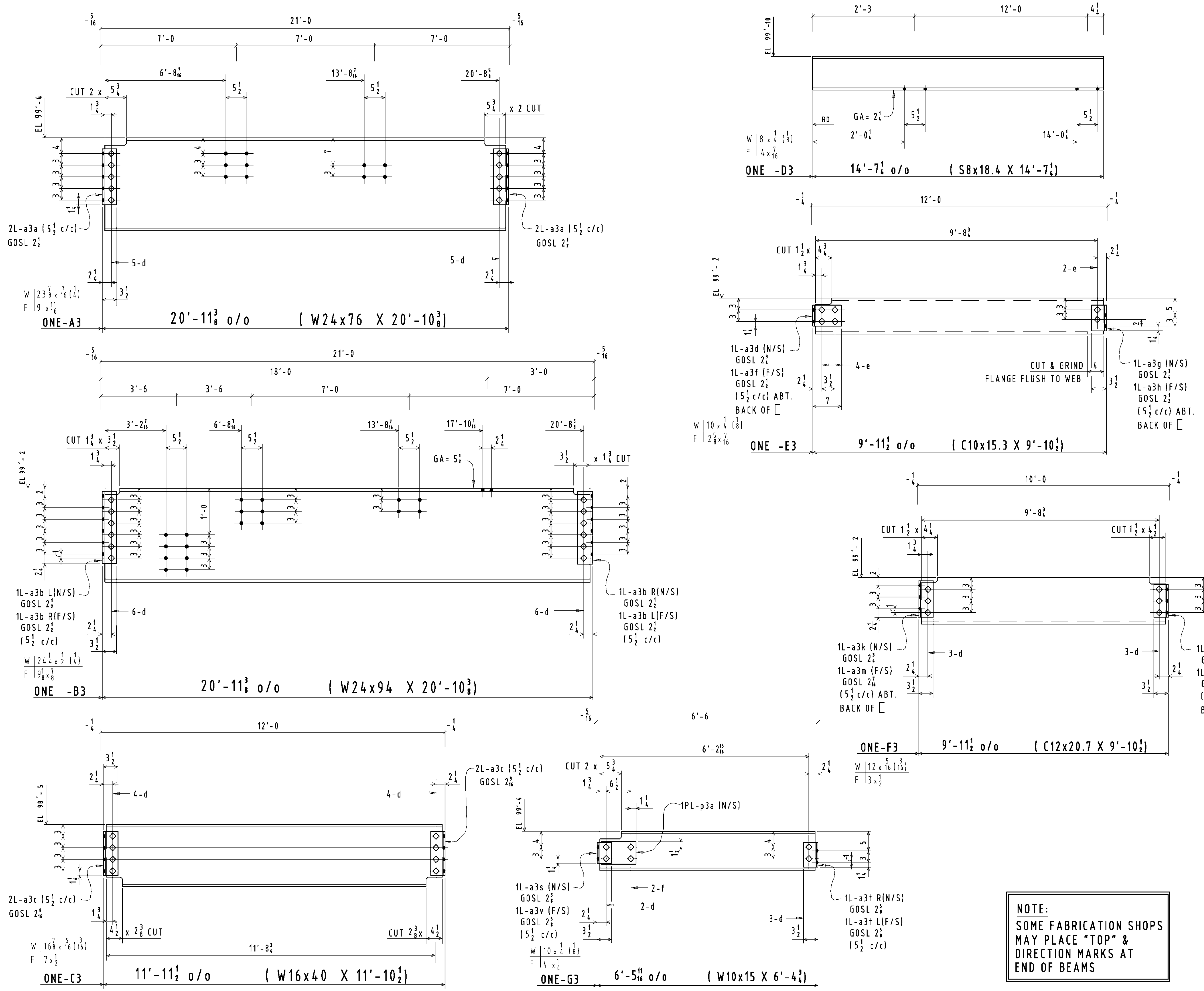
BILL OF MATERIAL

QTY	MARK	SHAPE	LENGTH FT	LENGTH IN	WGHT	GRADE	REM.
ONE	C565-3	W10x33	4	$8\frac{1}{2}$			SQ-1-E
1	ab565	$L6x4x\frac{3}{4}$	0	9			
2	pa565	$PL\frac{5}{16}x6$	0	9			

FIELD BOLTS	$6 - \frac{7}{8}\varnothing$A325 x 2
	$2 - \frac{7}{8}\varnothing$A325 x $2\frac{1}{2}$

Gen. Notes:
Spec: AISC-latest edition
Matl: W Shape ASTM A992
All other mat'l ASTM A36
Holes: $\frac{15}{16}$"$\varnothing$ unless noted
Weld electrode: E70XX
⊗ = STRIPE or Connecting Surface.
Paint: One coat SSPC 13 except where noted "NO PAINT". No paint on shop contact surfaces. Holes marked "◇" are for high strength structural bolts. NO PAINT within 3 inches of these holes.

Figure A7-20



NOTE:
SOME FABRICATION SHOPS
MAY PLACE "TOP" &
DIRECTION MARKS AT
END OF BEAMS

BILL OF MATERIAL

	QUAN	MARK	DESCRIPTION	LENGTH FT	LENGTH IN	TOTAL WT.	REMARKS	MILL ORDER
1								
2								
3	ONE	A3	W24x76	20	10 3/8	1586		
4	4	a3a	L4x3 1/2 x 5/16	1	2 1/2	37		
5	10	d	3/4 Ø A325 BOLT(S)	0	2 1/4	4		
6								
7								
8	ONE	B3	W24x94	20	10 3/8	1962		
9	4	a3b R L	L4x3 1/2 x 5/16	1	6 1/2	48	2=R;2=L	
10	12	d				5		
11								
12								
13	ONE	C3	W16x40	11	10 1/2	476		
14	4	a3c	L4x3 1/2 x 5/16	0	11 1/2	30		
15	8	d				3		
16								
17								
18	ONE	D3	S8x18.4	14	7 1/4	269		
19								
20								
21	ONE	E3	C10x15.3	9	10 1/2	152		
22	1	a3d	L7x4 x 3/8	0	5 1/2	6		
23	1	a3f	L7x4 x 3/8	0	5 1/2	6		
24	1	a3g	L4x3 1/2 x 5/16	0	7 1/2	5		
25	1	a3h	L4x3 1/2 x 5/16	0	7 1/2	5		
26	6	e	3/4 Ø A325 BOLT(S)	0	2	2		
27								
28								
29	ONE	F3	C12x20.7	9	10 1/2	205		
30	1	a3k	L4x3 1/2 x 5/16	0	9 1/2	6		
31	1	a3m	L4x3 1/2 x 5/16	0	9 1/2	6		
32	1	a3n	L4x3 1/2 x 5/16	0	8 1/2	5		
33	1	a3p	L4x3 1/2 x 5/16	0	8 1/2	5		
34	6	d				3		
35								
36								
37	ONE	G3	W10x15	6	4 3/4	96		
38	1	a3s	L4x3 1/2 x 5/16	0	5 1/2	9		
39	1	a3v	L4x3 1/2 x 5/16	0	5 1/2	9		
40	2	a3t R L	L4x3 1/2 x 5/16	0	6 1/2	4	1=R;1=L	
41	1	p3a	PL 1/4 x 6	0	9	4		
42	4	d				2		
43	2	f	3/4 Ø A325 BOLT(S)	0	1 3/4	1		
44								
45								
46								
47								
48								

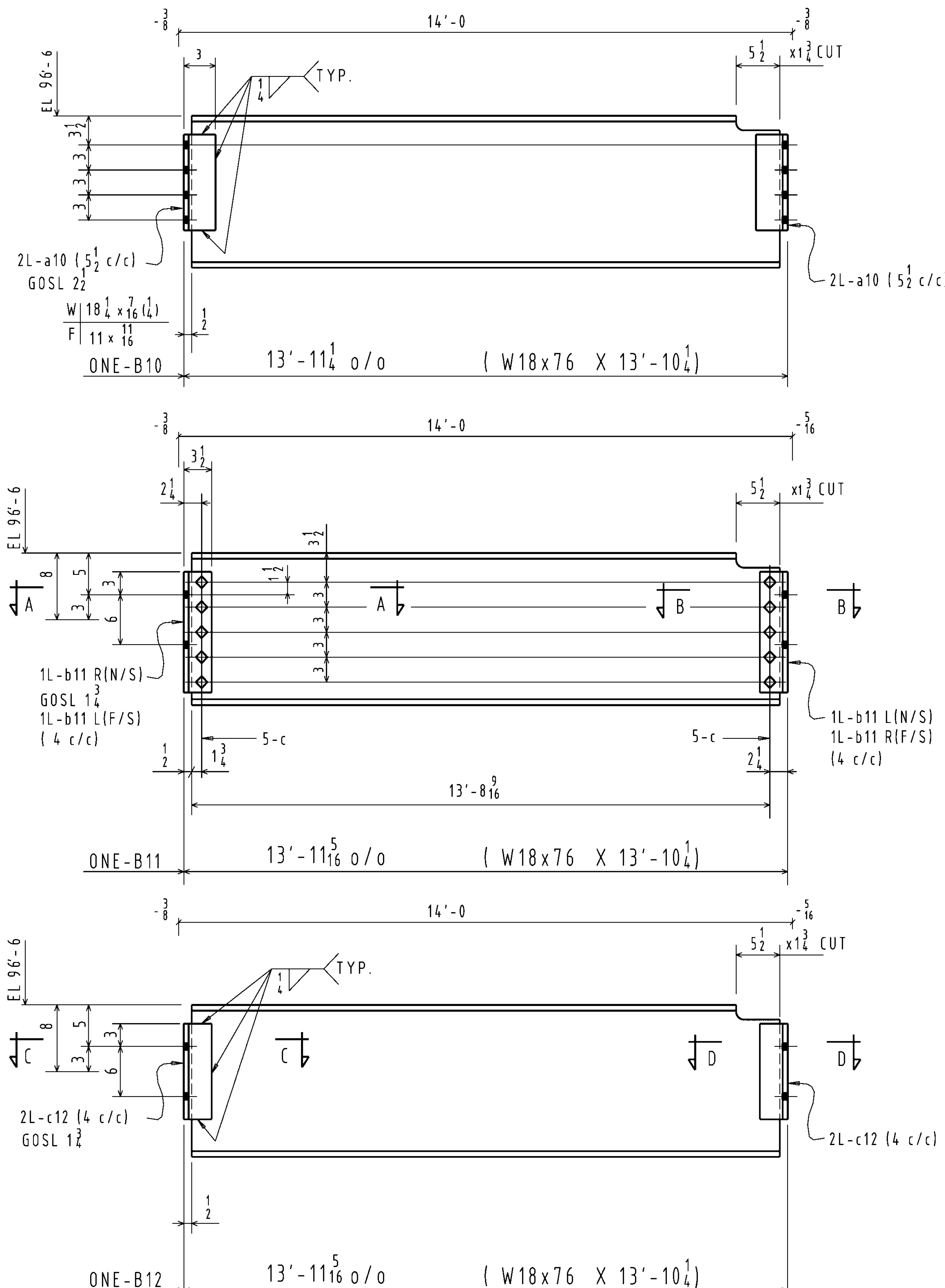
FIELD BOLTS	28- 3/4 Ø A325 x 2
	4- 3/4 Ø A325 x 2 1/2
PAINT	PER SPEC.
MATERIAL:	W Shapes: A992 Other: A36
HOLES	13/16 U/NOTED
ELECTRODES	
TOTAL WEIGHT	4951
BOLTS:	3/4 Ø A325N

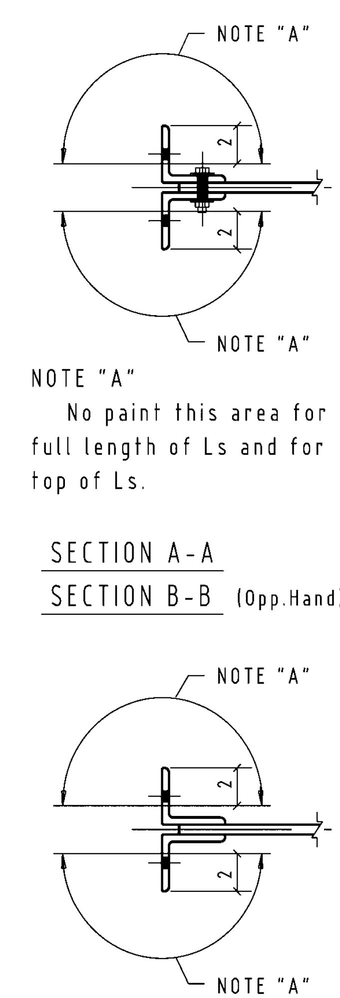
NO	DATE	DESCRIPTION	NO	DATE	DESCRIPTION

REVISIONS

Figure A7-34

DRAFTER MISD DATE 08-2000	CUSTOMER	ECKS WYE ZEE FABRICATING CO.	CONTRACT NO 3990
CHECKED AISC DATE 08-2000	STRUCTURE	MERCY HOSPITAL	DWG NO 3





NOTE "A"
No paint this area for full length of Ls and for top of Ls.

SECTION A-A
SECTION B-B (Opp.Hand)

SECTION C-C
SECTION D-D (Opp.Hand)

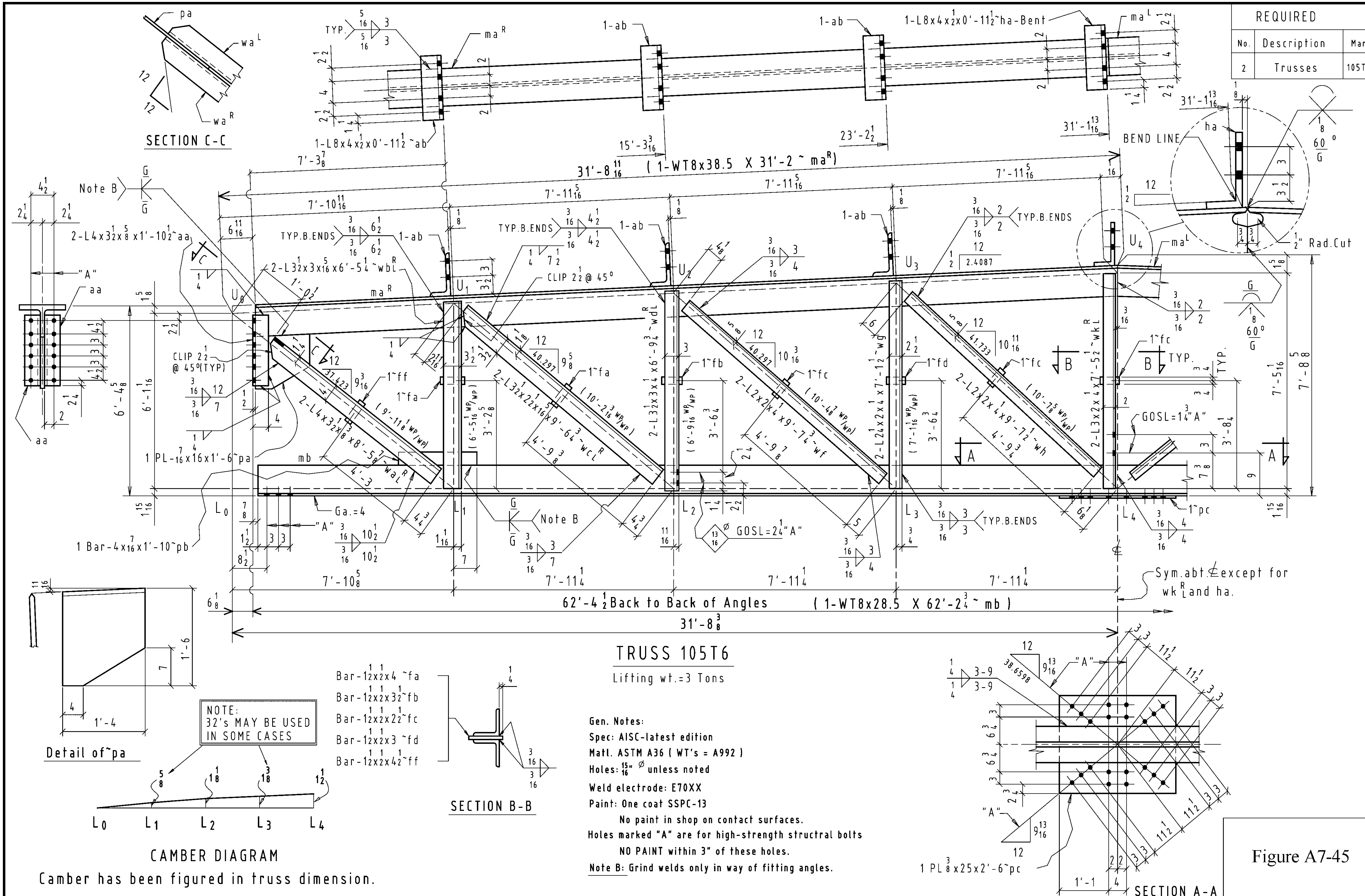
BILL OF MATERIAL

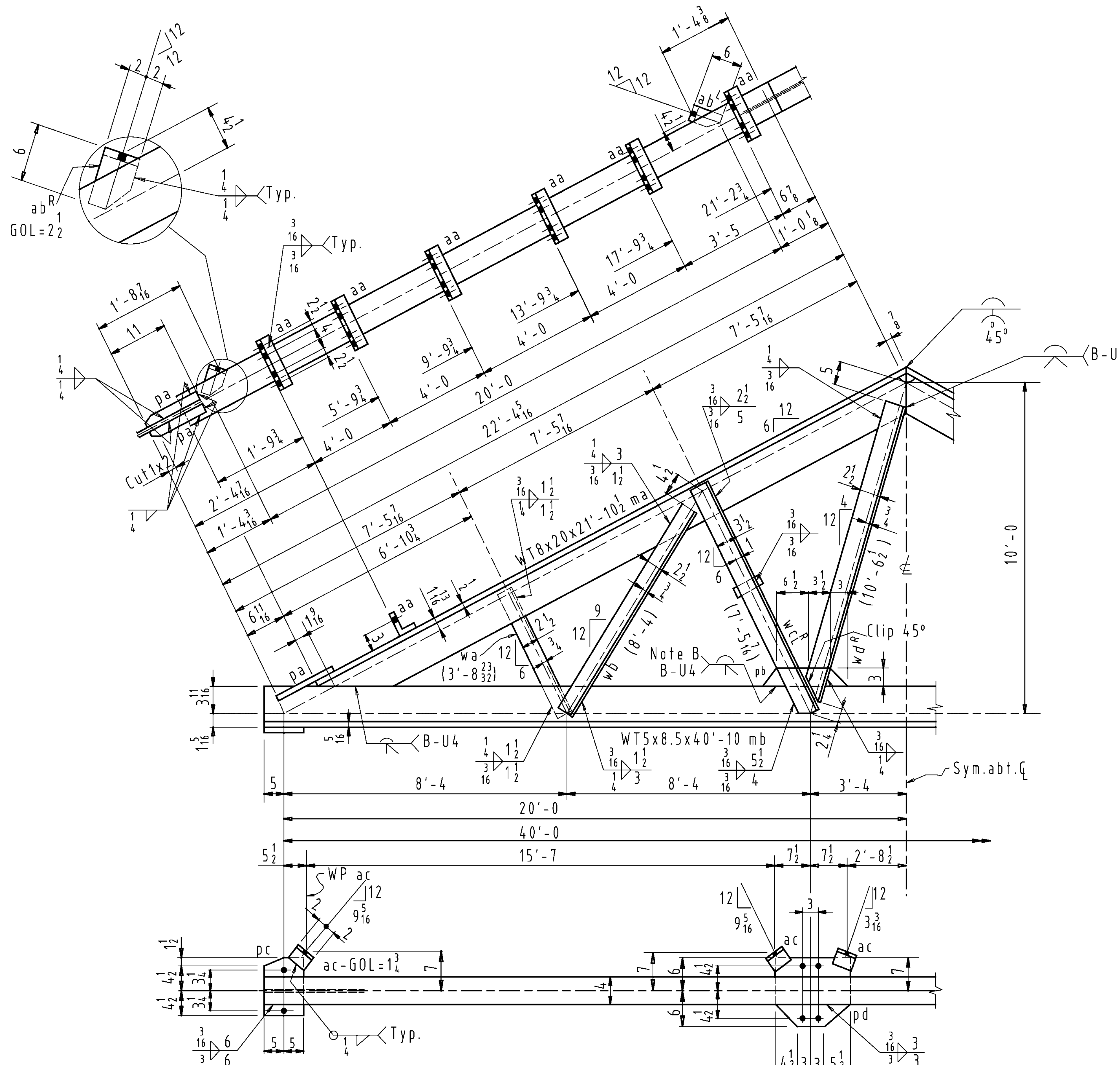
QTY	MARK	SHAPE	LENGTH FT	LENGTH IN	WGHT	GRADE	REM.
ONE	B10	W18x76	13	10¼			
4	a10	L4x3x5/16	1	0			
ONE	B11	W18x76	13	10¼			
4	b11 R/L	L3½x3x5/16	1	3			2=R;2=L
10	c	7/8 Ø A325	0	2½			
ONE	B12	W18x76	13	10¼			
4	c12	L3x3x5/16	1	0			

FIELD BOLTS

Gen. Notes:
Spec: AISC-latest edition
Matl: W Shape ASTM A992
All other mat'l ASTM A36
Holes: 15/16" Ø unless noted
Shop Fasteners: 7/8" Ø unless noted
Weld electrode: E70XX
Paint: One coat SSPC 13 except where noted "NO PAINT".

Figure A7-40


REQUIRED
No. Description Mark
2 Trusses 105T6
SECTION C-C
SECTION B-B
SECTION A-A
Detail of~pa
NOTE:
32's MAY BE USED
IN SOME CASES
CAMBER DIAGRAM
Camber has been figured in truss dimension.
TRUSS 105T6
Lifting wt.=3 Tons
Gen. Notes:
Spec: AISC-latest edition
Matl. ASTM A36 (WT's = A992)
Weld electrode: E70XX
Paint: One coat SSPC-13
No paint in shop on contact surfaces.
Holes marked "A" are for high-strength structral bolts
NO PAINT within 3" of these holes.
Note B: Grind welds only in way of fitting angles.
62'-4 1/2 Back to Back of Angles
(1-WT8x28.5 X 62'-2 3/4 ~ mb)
(1-WT8x38.5 X 31'-2 ~ maR)
Sym.abt.℄except for wkR L and ha.
BEND LINE
1/2" Rad.Cut
Figure A7-45



TRUSS - 110T3

BILL OF MATERIAL

QUAN	MARK	DESCRIPTION	LENGTH FT	LENGTH IN	WEIGHT	MILL ORDER No.	REMARKS
ONE	110T3						
2	ma	WT8x20	21	$10\frac{1}{2}$			A992
ONE	mb	WT5x8.5	40	10			A992
2	wa	$L2\frac{1}{2}x2x\frac{1}{4}$	3	$9\frac{1}{2}$			
2	wb	$L2\frac{1}{2}x2x\frac{1}{4}$	7	$11\frac{1}{2}$			
4	$wc^R{}_L$	$L3\frac{1}{2}x3\frac{1}{2}x\frac{5}{16}$	7	$6\frac{1}{4}$			2=R2=L
2	$wd^R{}_L$	$L2\frac{1}{2}x2x\frac{1}{4}$	9	$11\frac{1}{4}$			1=R1=L
4	pa	$FL2\frac{1}{2}x\frac{3}{8}$	0	11			
12	aa	$L4x3x\frac{3}{8}$	1	0			
4	$ab^R{}_L$	$L6x4x\frac{1}{2}$	0	4			2=R2=L
2	fa	$FL1\frac{1}{2}x\frac{5}{16}$	0	$4\frac{1}{2}$			
2	pb	$FL3x\frac{1}{4}$	1	4			
2	pc	$PL\frac{1}{2}x10$	0	$10\frac{1}{2}$			
2	pd	$PL\frac{3}{8}x12$	1	4			
6	ac	$L3x3x\frac{3}{8}$	0	4			

Gen. Notes:

Spec : AISC latest edition

Matl : ASTM A36

Holes : $\frac{15}{16}$ diam.

Welding : E70XX

Paint : One coat SSPC 13

No paint on shop contact surfaces.

Note A : Gouge single U groove after fitting.

Note B : Grind welds only in way of fitting angles.

No Camber.

Figure A7-46

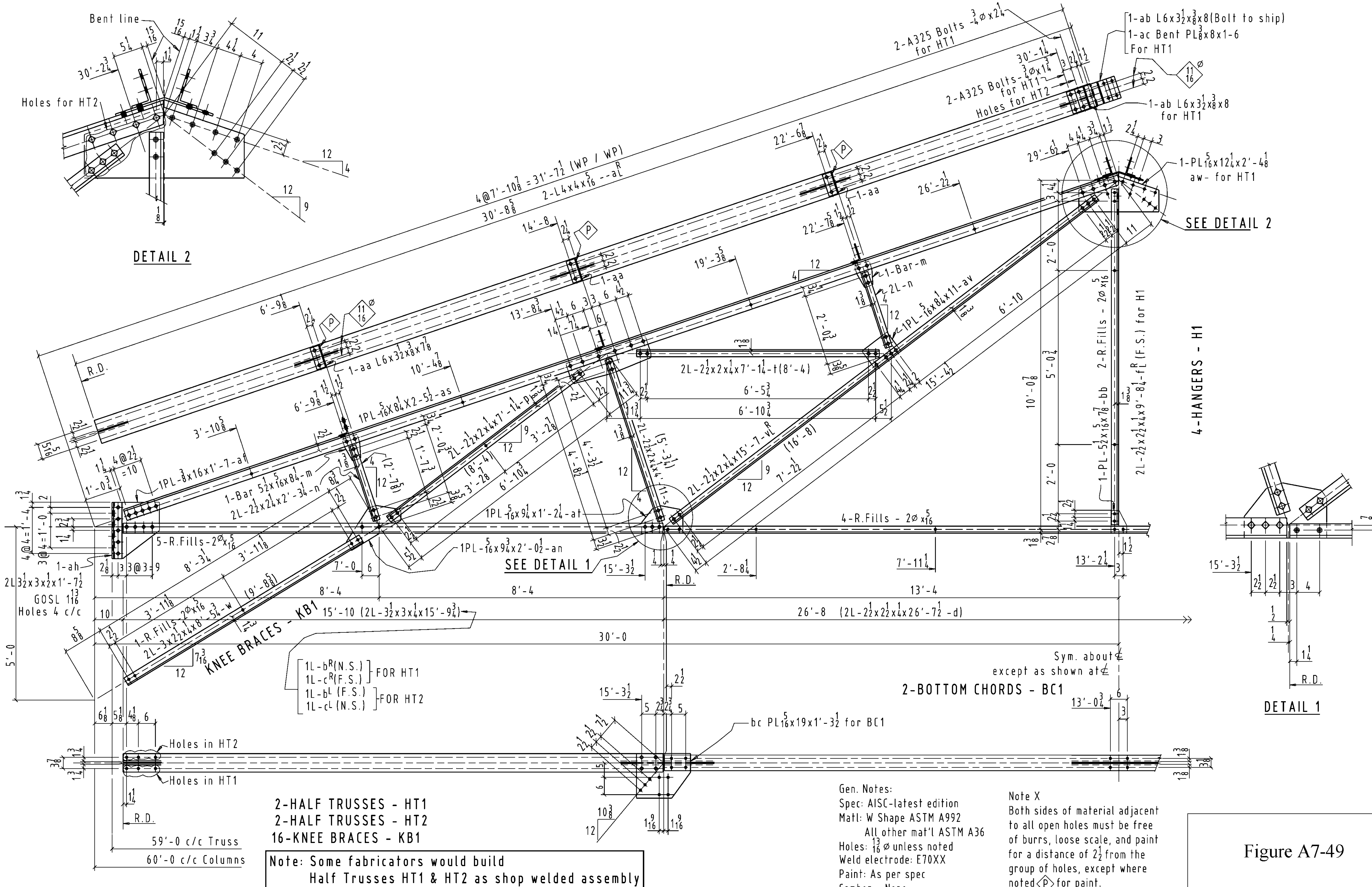
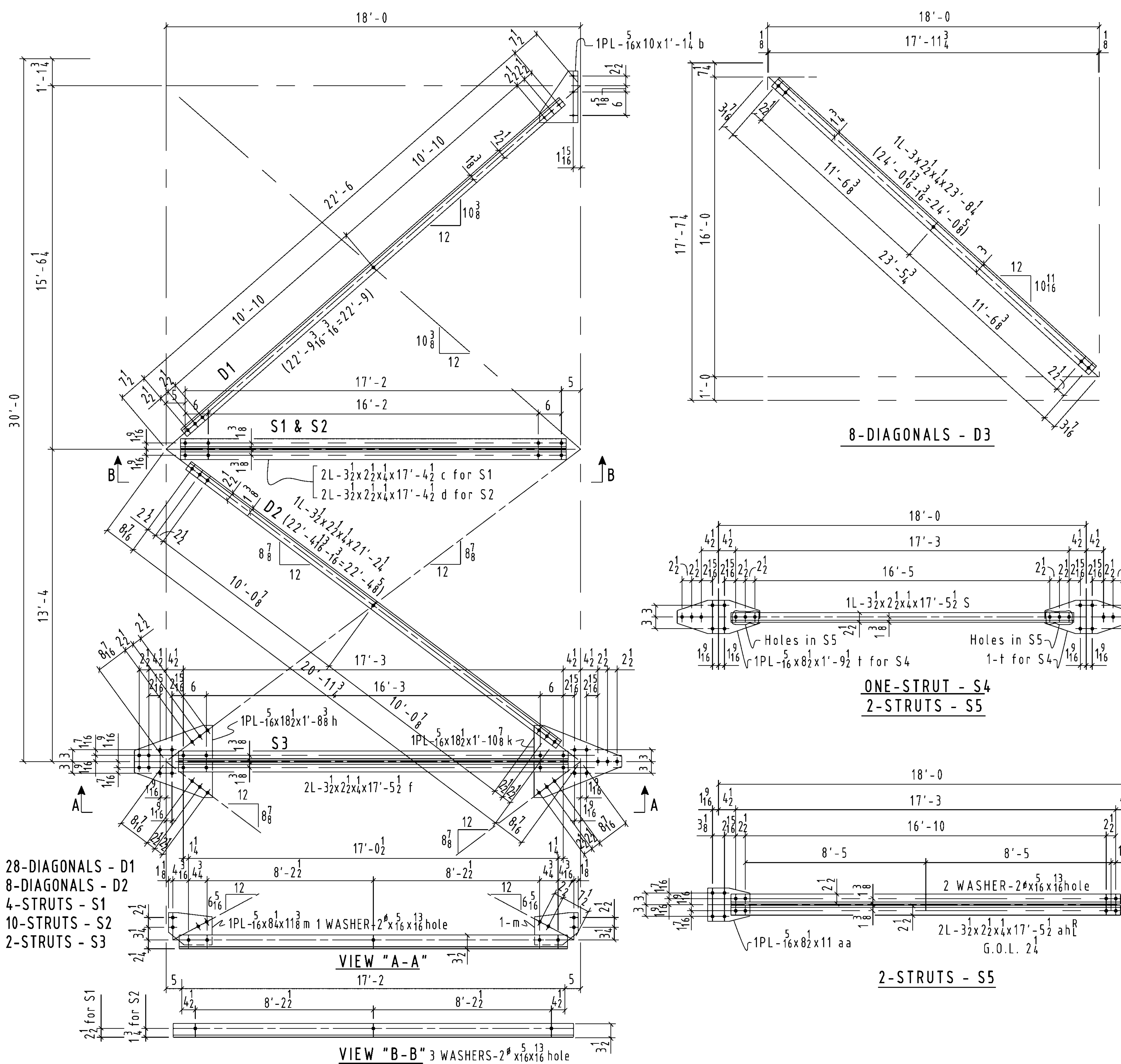
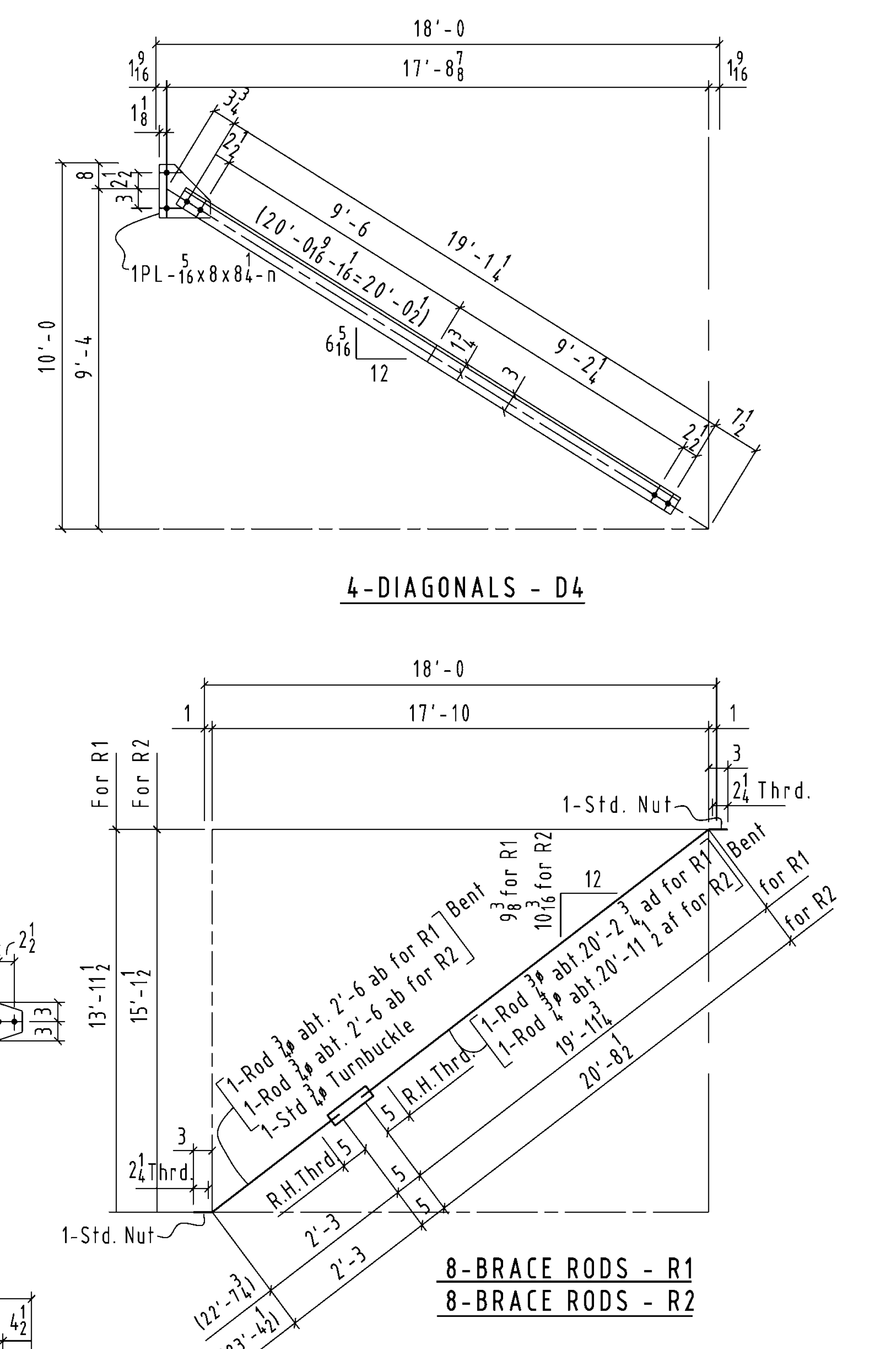

DETAIL 2
DETAIL 1
SEE DETAIL 1
SEE DETAIL 2
KNEE BRACES - KB1
4-HANGERS - H1
2-BOTTOM CHORDS - BC1
Sym. about ℄
except as shown at ℄
2-HALF TRUSSES - HT1
2-HALF TRUSSES - HT2
16-KNEE BRACES - KB1
Note: Some fabricators would build
Half Trusses HT1 & HT2 as shop welded assembly
59'-0 c/c Truss
60'-0 c/c Columns
Gen. Notes:
Spec: AISC-latest edition
Matl: W Shape ASTM A992
All other mat'l ASTM A36
Holes: 13/16 ⌀ unless noted
Weld electrode: E70XX
Paint: As per spec
Camber - None
Note X
Both sides of material adjacent
to all open holes must be free
of burrs, loose scale, and paint
for a distance of 2 1/2 from the
group of holes, except where
noted P for paint.

Figure A7-49





Gen. Notes:
Spec: AISC - Latest edition
Mat'l: ASTM A36

Shop bolts: ¾ø ASTM A325
Paint: As per specification
Open holes are for high strength bolts. No paint within 5" of these holes.

Figure A7-51

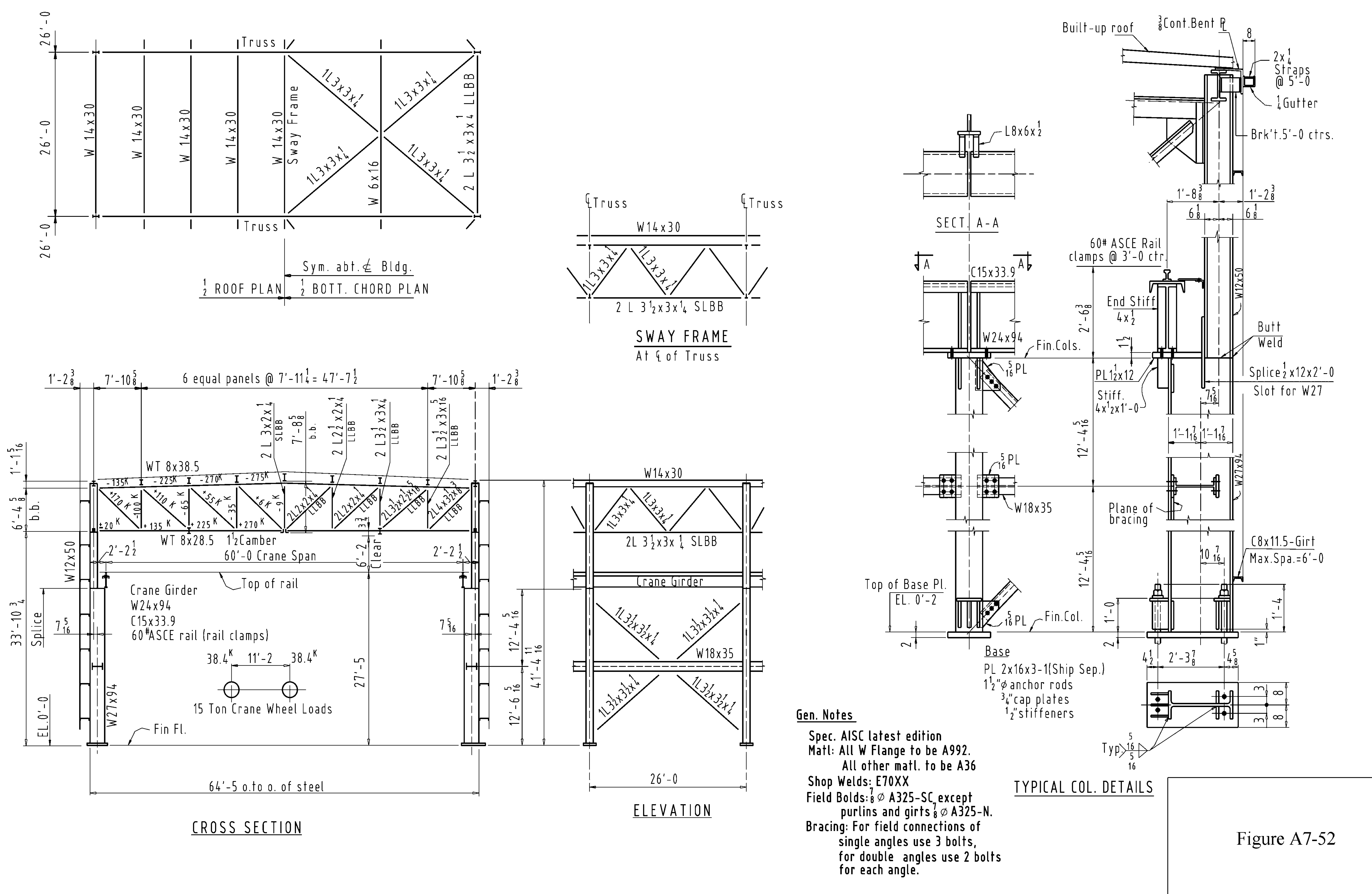
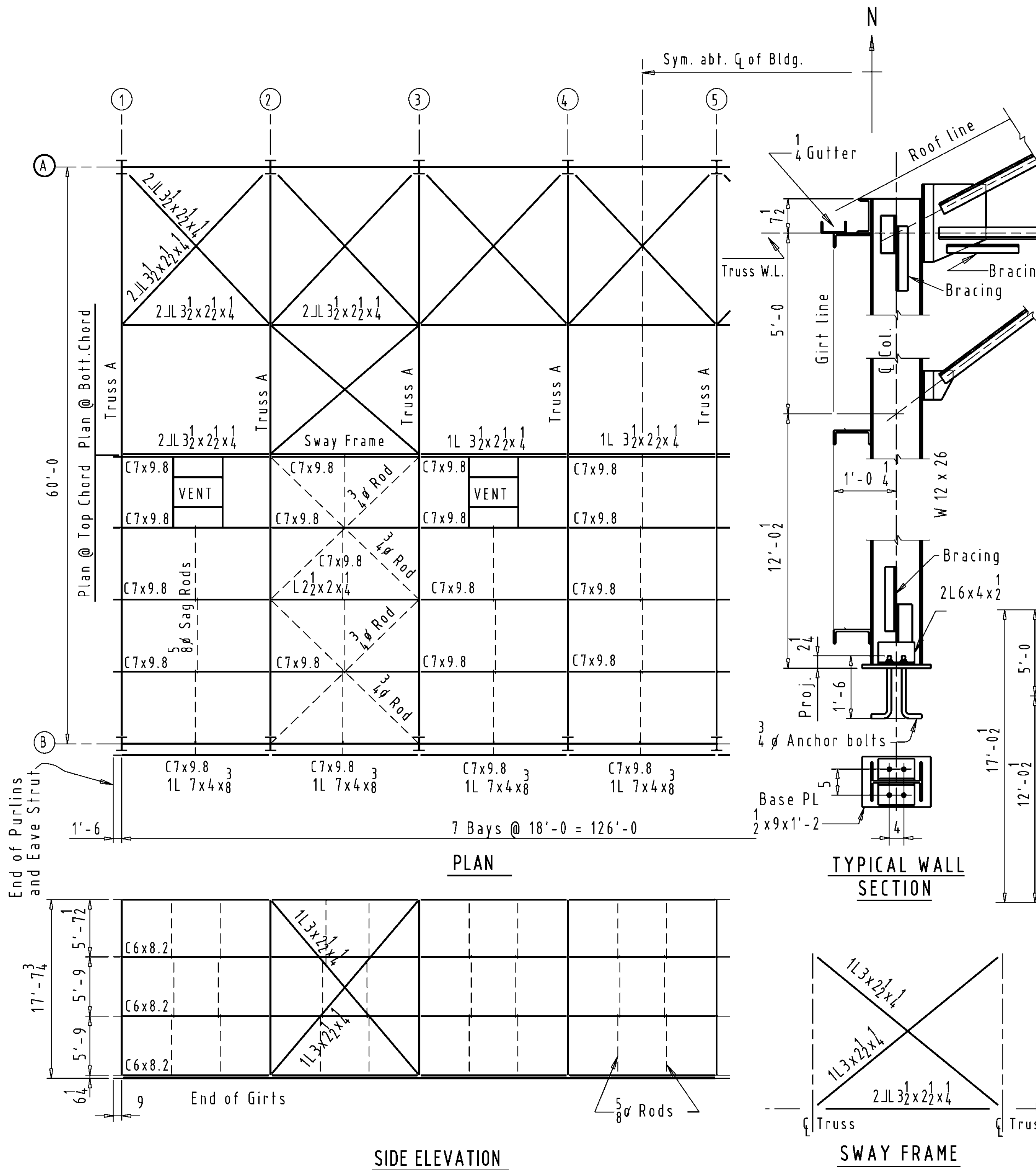
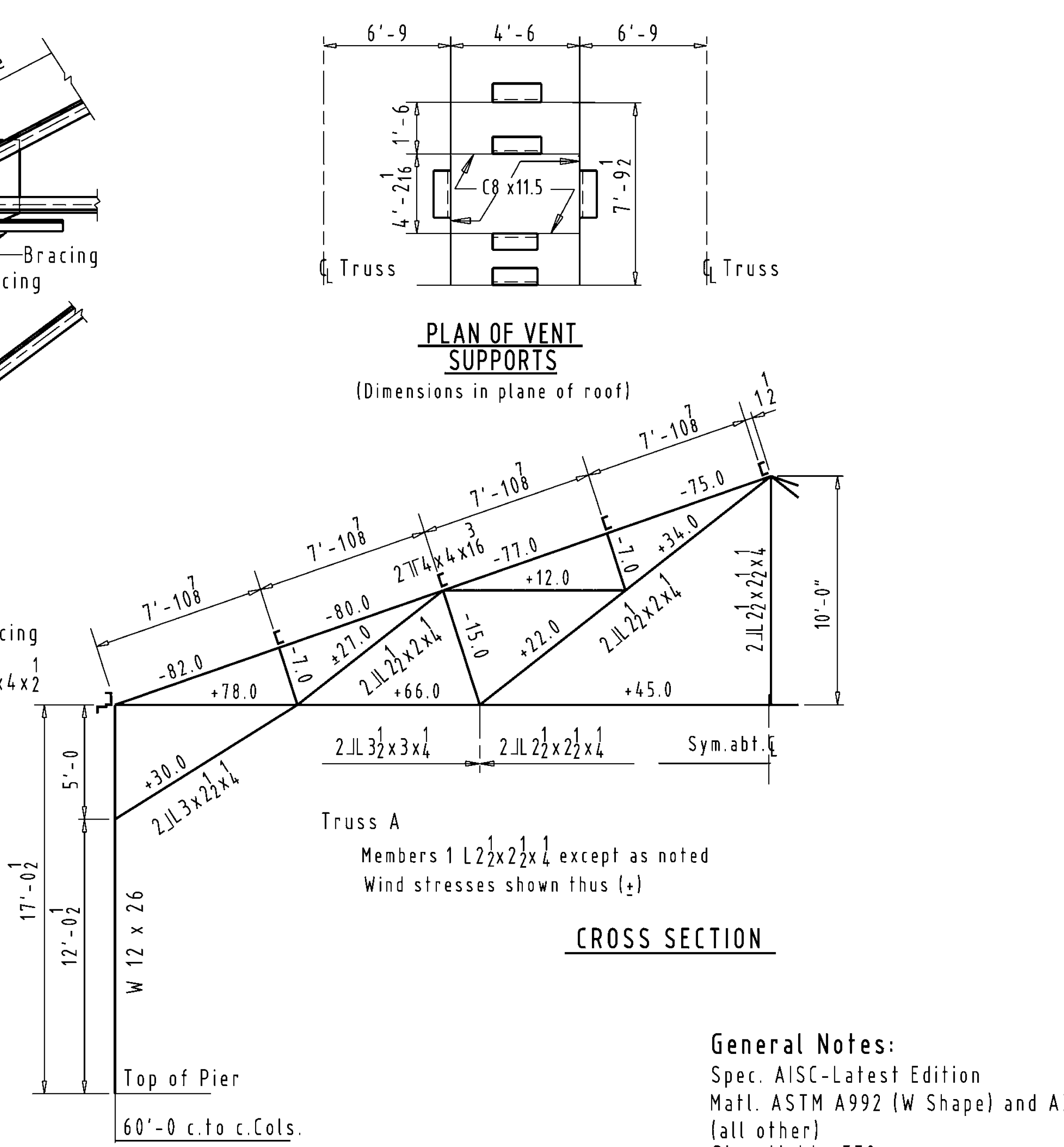

1/2 ROOF PLAN
1/2 BOTT. CHORD PLAN
Sym. abt. ℄ Bldg.
Sway Frame
SWAY FRAME
At ℄ of Truss
CROSS SECTION
ELEVATION
SECT. A-A
TYPICAL COL. DETAILS
15 Ton Crane Wheel Loads
Crane Girder
W24x94
C15x33.9
60# ASCE rail (rail clamps)
60'-0 Crane Span
64'-5 o.to o. of steel
6 equal panels @ 7'-11 1/4 = 47'-7 1/2
Gen. Notes
Spec. AISC latest edition
Matl: All W Flange to be A992.
All other matl. to be A36
Shop Welds: E70XX
Field Bolds: 7/8 ⌀ A325-SC except
purlins and girts 7/8 ⌀ A325-N.
Bracing: For field connections of
single angles use 3 bolts,
for double angles use 2 bolts
for each angle.

Figure A7-52





General Notes:
Spec. AISC-Latest Edition
Matl. ASTM A992 (W Shape) and A36 (all other)
Shop Weld : E70xx
Shop and Field bolts : ASTM A325 3/4" Ø bolts 5/8 Ø
A307 bolts may be used to connect purlins and girts not used as struts.

Figure A7-66

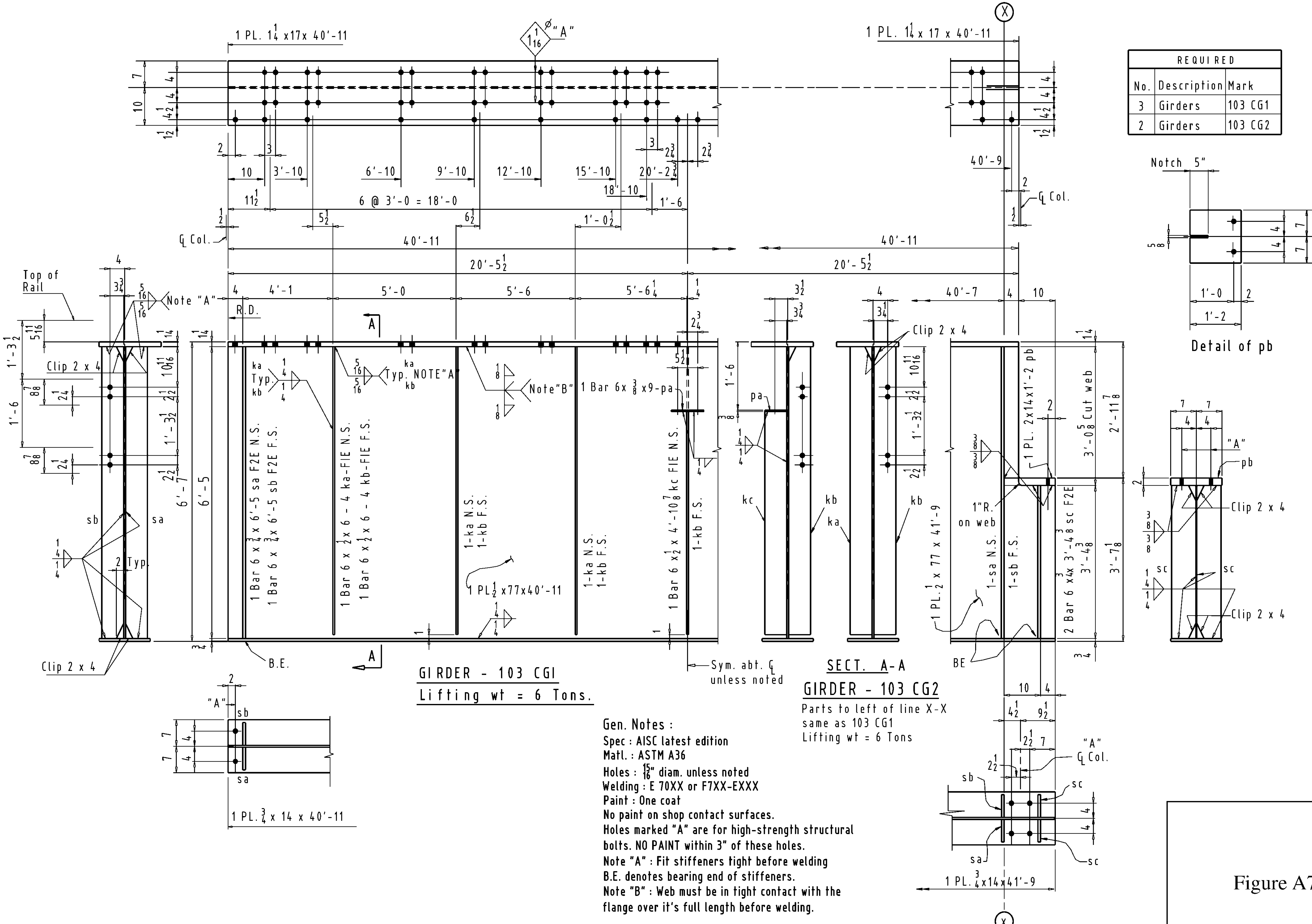

GIRDER - 103 CGI
Lifting wt = 6 Tons.
SECT. A-A
GIRDER - 103 CG2
Parts to left of line X-X
same as 103 CG1
Lifting wt = 6 Tons
Sym. abt. ℄ unless noted
Detail of pb
Notch 5"
REQUIRED
No. Description Mark
3 Girders 103 CG1
2 Girders 103 CG2
Gen. Notes :
Spec : AISC latest edition
Matl. : ASTM A36
Holes : 15/16" diam. unless noted
Welding : E 70XX or F7XX-EXXX
Paint : One coat
No paint on shop contact surfaces.
Holes marked "A" are for high-strength structural
bolts. NO PAINT within 3" of these holes.
Note "A" : Fit stiffeners tight before welding
B.E. denotes bearing end of stiffeners.
Note "B" : Web must be in tight contact with the
flange over it's full length before welding.

Figure A7-69

APPENDIX B

ENGINEERING FUNDAMETALS

As noted earlier in this text, the detailer works closely with the Structural Engineer of Record (or other Owner's Designated Representative for Design) to translate design data into information that the fabricator and erector need to build the structure. Clear communication between the engineer and the steel detailer is extremely important to making this translation successful. Drawings are an important part of this communication, as is a vocabulary that each party understands. This appendix will explain engineering vocabulary and will show how engineering fundamentals are reflected in many of the common details used in the fabrication of steel structures.

DEFINITIONS

Structure

The principal components in a steel building frame are structural members and the connections that hold the members together. The role of the structural engineer is to determine the arrangement and size of these components to ensure the most safe, efficient and economic use of materials in creating the structure.

MEMBERS

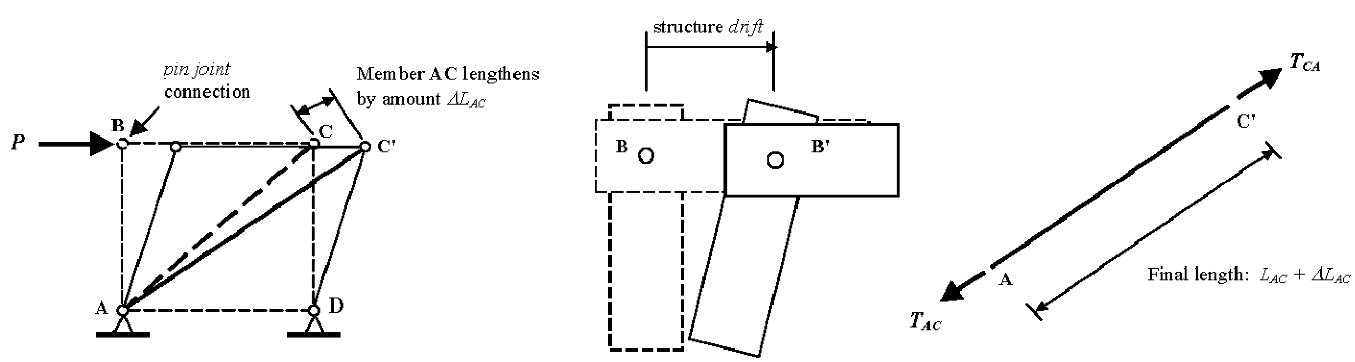
Structural members can be classified by how they are loaded or how they transfer force from one point on the structure to another. There are three main types of structural members: tension members, compression members and bending members.

Tension Members

Tension members are members that are pulled (i.e., loaded in tension) by two forces that act in a line of action through the length of the member. The line of action must coincide with the centroid, or center of gravity, of the member's cross section, otherwise the force causing tension will also cause bending.

Tension members are often used for bracing and hangers.

Figure B-1a shows a five-member truss structure loaded with a single horizontal force, *P*, applied at corner B. A truss is a structure in which all members are held together by pin-joint connections that are free to rotate as the structure deflects from an applied load (Figure B-1b). The members are arranged in triangular patterns to make the truss stable. Every member in a truss is subjected to either a tension force or a compression force. The force direction (tension or compression) can often be determined from the deflected shape of the truss. For example, member AC, whose length is L_{AC}, must be stretched by amount ΔL_{AC} in order to remain connected to joints A and C (the symbol Δ is used to denote a change, so that ΔL_{AC} means "the change in length of member AC"). The structure drift shown in Figure B-1b is the lateral motion of the entire structure at level BC. The internal forces produced in member AC by the external force *P* are T_{AC} and T_{CA}, as shown in Figure B-1c. In this case, there are two forces (forces T_{AC} and T_{CA}) acting on member AC. The two subscripts identify the member; the first subscript denotes the end of the member where the force acts.



a) Structure ABCD loaded with force *P* at joint B

b) Structure articulation at *pin joint* connection at joint B (similar for joints A , C and D)

c) Member AC

Figure B-1. Tension member—diagonal bracing.

The forces T_{AC} and T_{CA} must be equal in magnitude and opposite in direction (as shown) in order for the member to remain in equilibrium. The concept of equilibrium will be discussed later in this appendix.

Compression Members

Compression members are members that are pushed, or loaded in compression, by two forces that act in a line of action along the length of the member. Columns, posts and struts are examples of compression members. Figure B-2a shows the same five-member truss structure shown in Figure B-1, except that the diagonal member is reversed. Force P applied at corner B causes the structure to deflect to the right, requiring that member BD shortens. This shortening is produced by the compressive forces C_{BD} and C_{DB} shown on member BD in Figure B-2b. As before, equilibrium requires that C_{BD} and C_{DB} be equal.

Bending Members

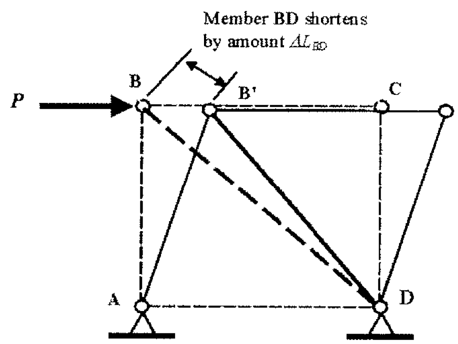
A beam is a generic name for a bending member. Girders, joists, purlins, girts and spandrels are other common names of bending members. These names describe how the member is used in the structure, not how it is loaded. Floor beams and roof beams are horizontal members that support vertical gravity floor and roof loads. Joists are another name for both floor and roof beams. Purlins are roof beams. Girts are beams used in the plane of a building's wall to transfer horizontal wind forces into the frame of the structure. Girders are usually large beams that support smaller beams. Spandrels are beams around the perimeter of a building that support the building veneer.

Beams are also classified by geometry and support conditions. Examples are shown in Figures B-3 through B-5. Each figure shows a pictorial view of the beam and a schematic view that would be used by an engineer to represent the beam and its support conditions. The dotted line on each schematic view shows the deflected shape that would be caused by the general force shown.

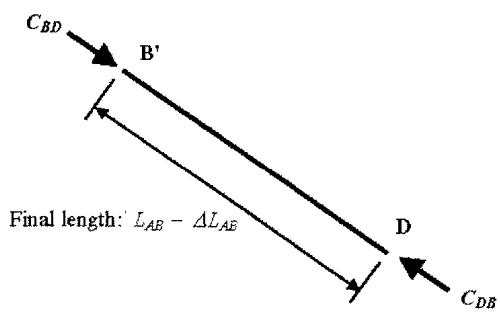
A simple beam is "simply supported" at each end. In Figure B-3a, the support at one end is a pin, and the support at the other end is a roller. Each allows the end of the beam to rotate. The pin keeps the end of the beam from moving horizontally and vertically. The roller prevents motion vertically. These provide the minimum restraints required to hold the beam in place and make it stable (e.g., a roller at each end would be unstable because if a horizontal force were to act on the beam, the whole structure would move freely in the horizontal direction like a roller-skate). A typical schematic view of a simple beam is shown in Figure B-3b. A double-angle framed beam connection (Figure B-3c) is an example of a simple support because the connection angles are flexible enough to distort, thus allowing the beam ends to rotate as the beam deflects. Shear tabs are simple supports because distortion of the bolt holes in the tab plate lets the ends of the supported beam rotate.

A fixed beam support restrains the beam end so that it cannot rotate as the beam deflects. Figure B-4a shows a beam that is fixed at both ends. The schematic view of this beam is shown in Figure B-4b. A cantilever beam is one with at least one end "free." Figure B-5a shows three possible forms of cantilevers.

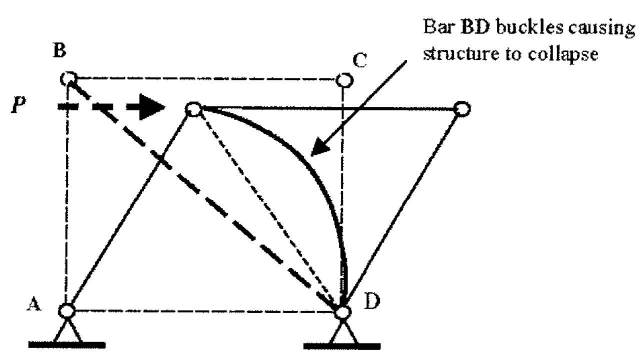
Bending members are not limited to beams that are directly loaded, such as the floor or roof beams. Members that



a) Structure ABCD loaded with force P at joint B.

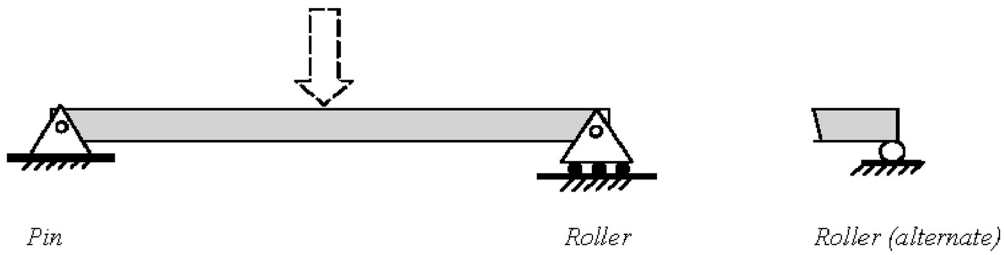


b) Member BD.

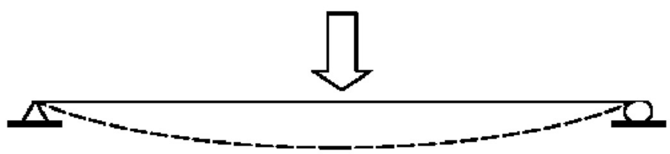


c) Compression member buckling failure

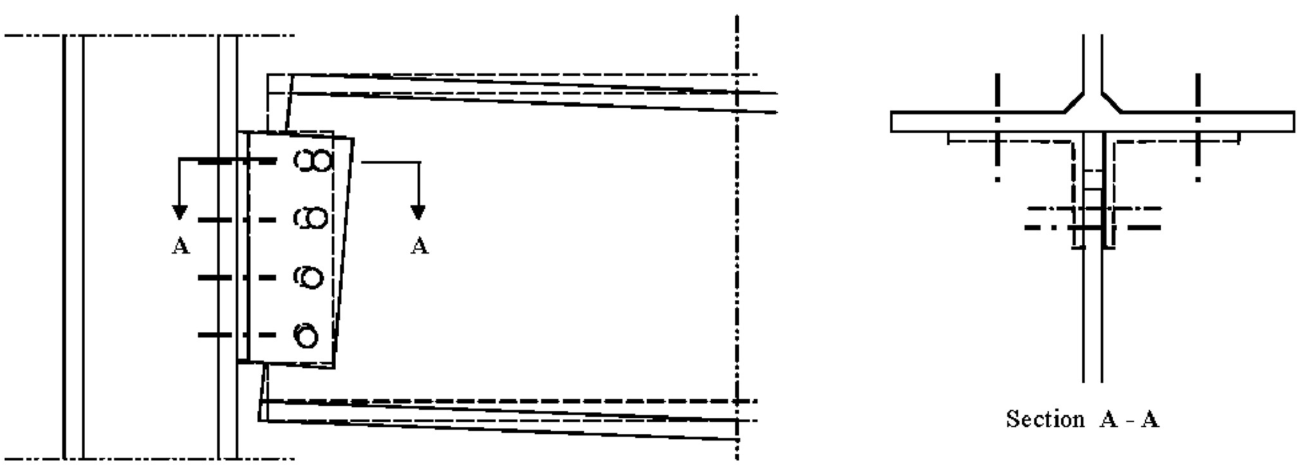
Figure B-2. Compression member—diagonal bracing.



a) Idealized beam and support details

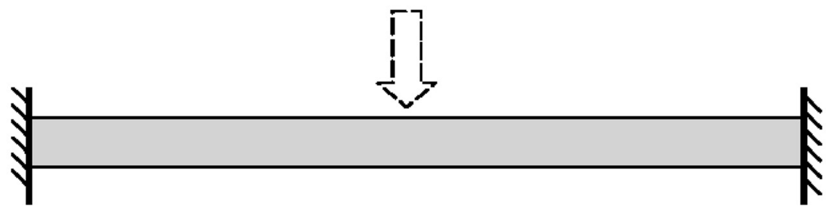


b) Schematic diagram (solid line) and deflected shape (dashed line)

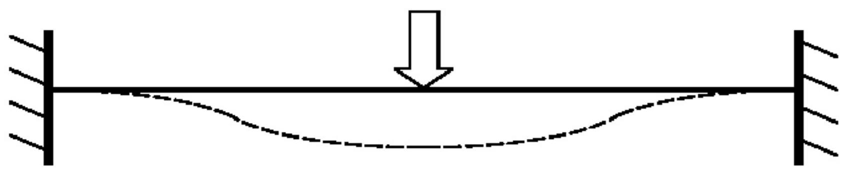


c) Framed-beam connection behavior as “simple connection”

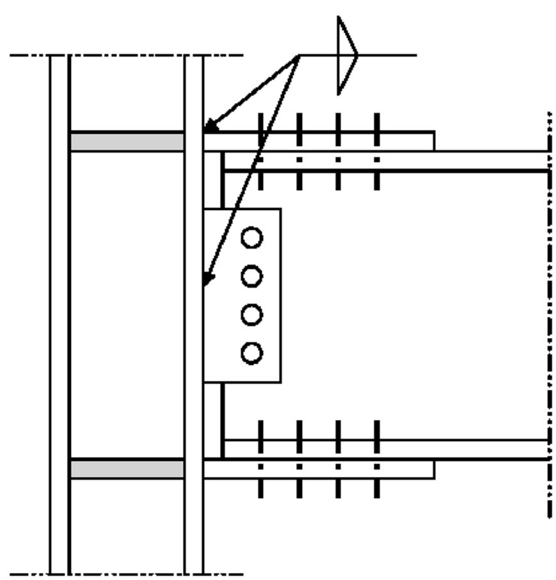
Figure B-3. Simply supported beam.



a) Idealized beam and support details



b) Schematic diagram (solid line) and deflected shape (dashed line)



c) Fully restrained (FR) flange plate moment-resisting connection

Figure B-4. Fixed beam.

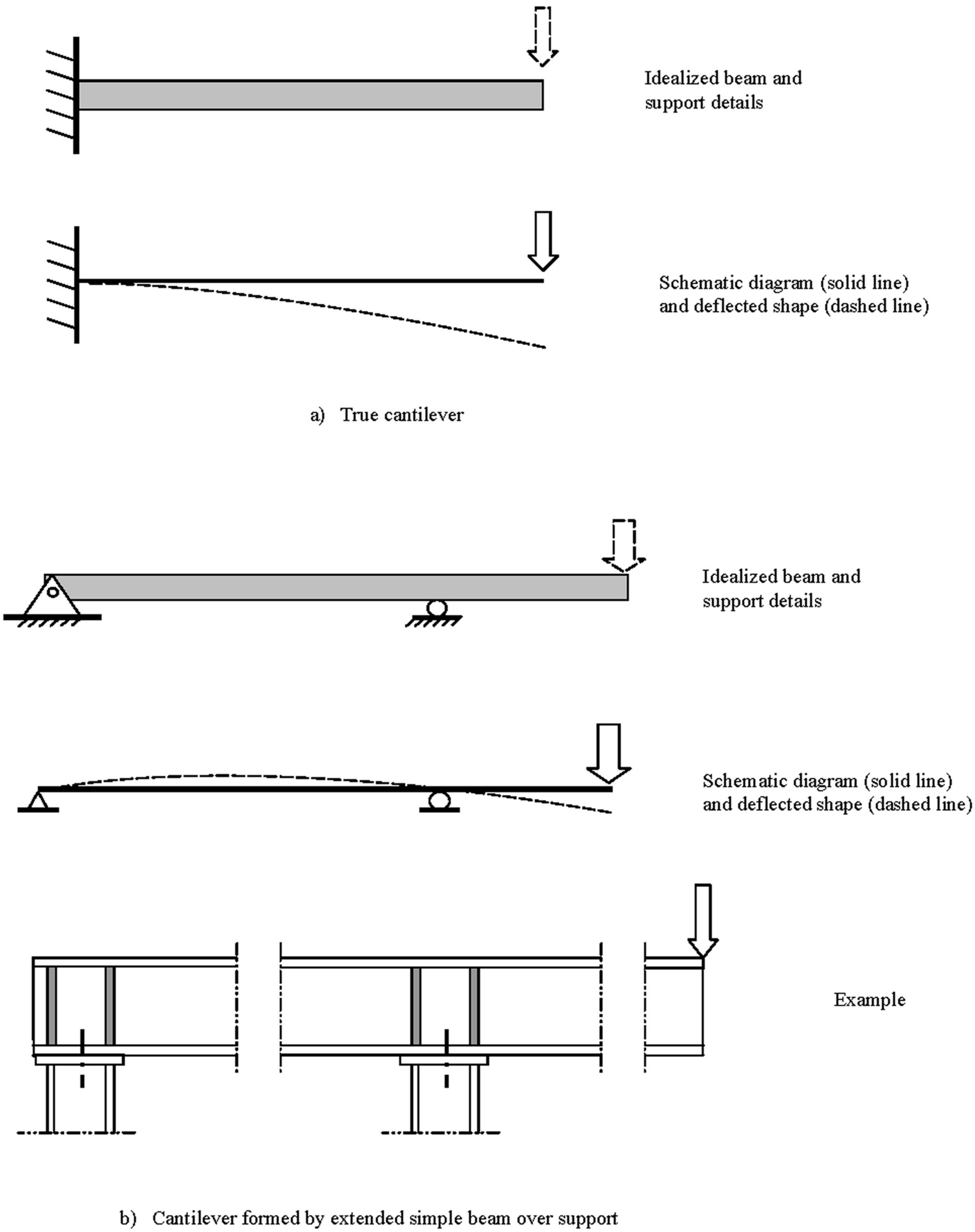


Figure B-5. Cantilevered beam.

are part of a frame flex (bend) because externally applied loads cause the frame to distort. Stability of the frame is provided by the rigid connection between the members. Two types of simple frames are shown in Figure B-6. The frames in Figures B-6a and B-6b are portal frames, in which all members are connected at right angles to each other. Gable frames (Figure B-6c) contain inclined members that do not all meet at right angles. The portal frame in Figure B-6b provides better resistance to the external load than that in Figure B-6a because the base of each column is fixed. However, fixed column bases must be carefully designed and detailed to assure the connections are actually as rigid as assumed.

LOADS (CLASSIFIED BY ORIGIN)

The main purpose of a steel structure in a building or bridge is to support loads. Loads are the forces that the structure must be designed to support. The engineer's first step in a structural design is to define the loads that the structure must support. There are three main types of loads on buildings; dead loads, live loads and loads produced by forces of nature. Loads are defined by building standards such as ASCE 7-05 (ASCE, 2006).

Dead Load

Dead load is the actual weight of the structure. Dead loads include things like the architectural features, the HVAC system and electrical systems, and the structure itself. The weights of steel members are found in the AISC *Steel Construction Manual*. The weight of the structure is usually not known at the outset of a design. In this case a weight is initially assumed and then refined as the design progresses.

Live Load

Live loads are loads created by the occupancy of the building. These generally include human occupancy and the structure's moveable contents. Sometimes these loads are very accurately defined (e.g., the volume of an elevated water tank determines the weight of the water in the tank). However, some live loads (especially human occupancy loads) are not easily defined.

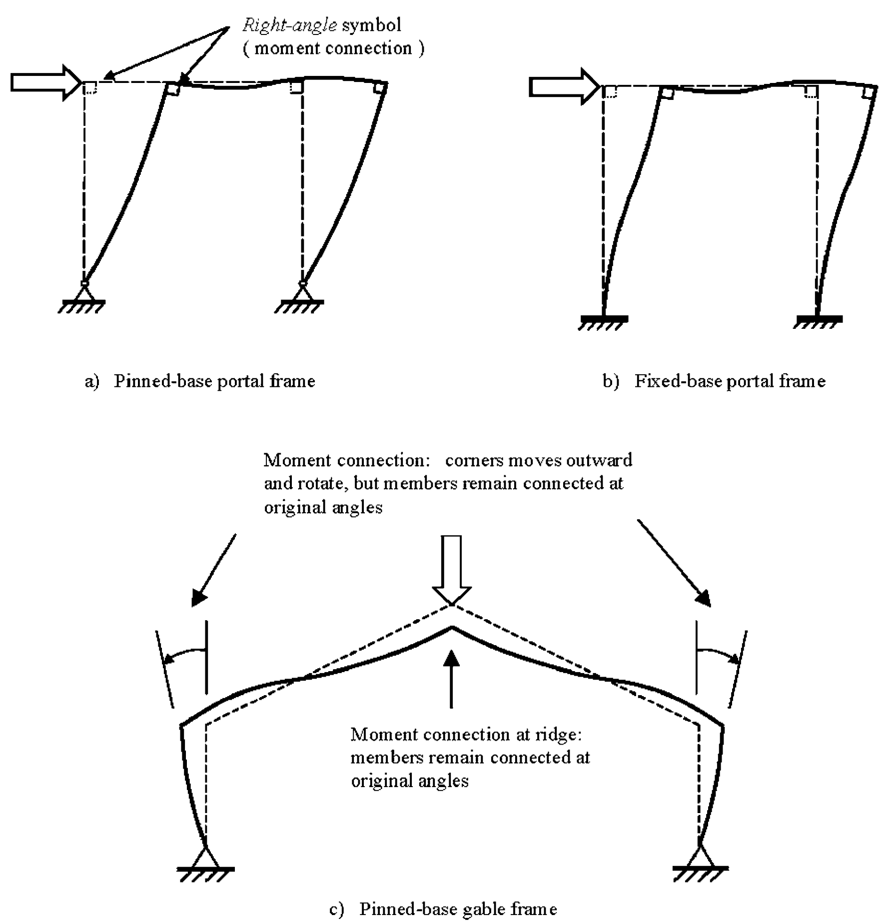


Figure B-6. Rigid frames.

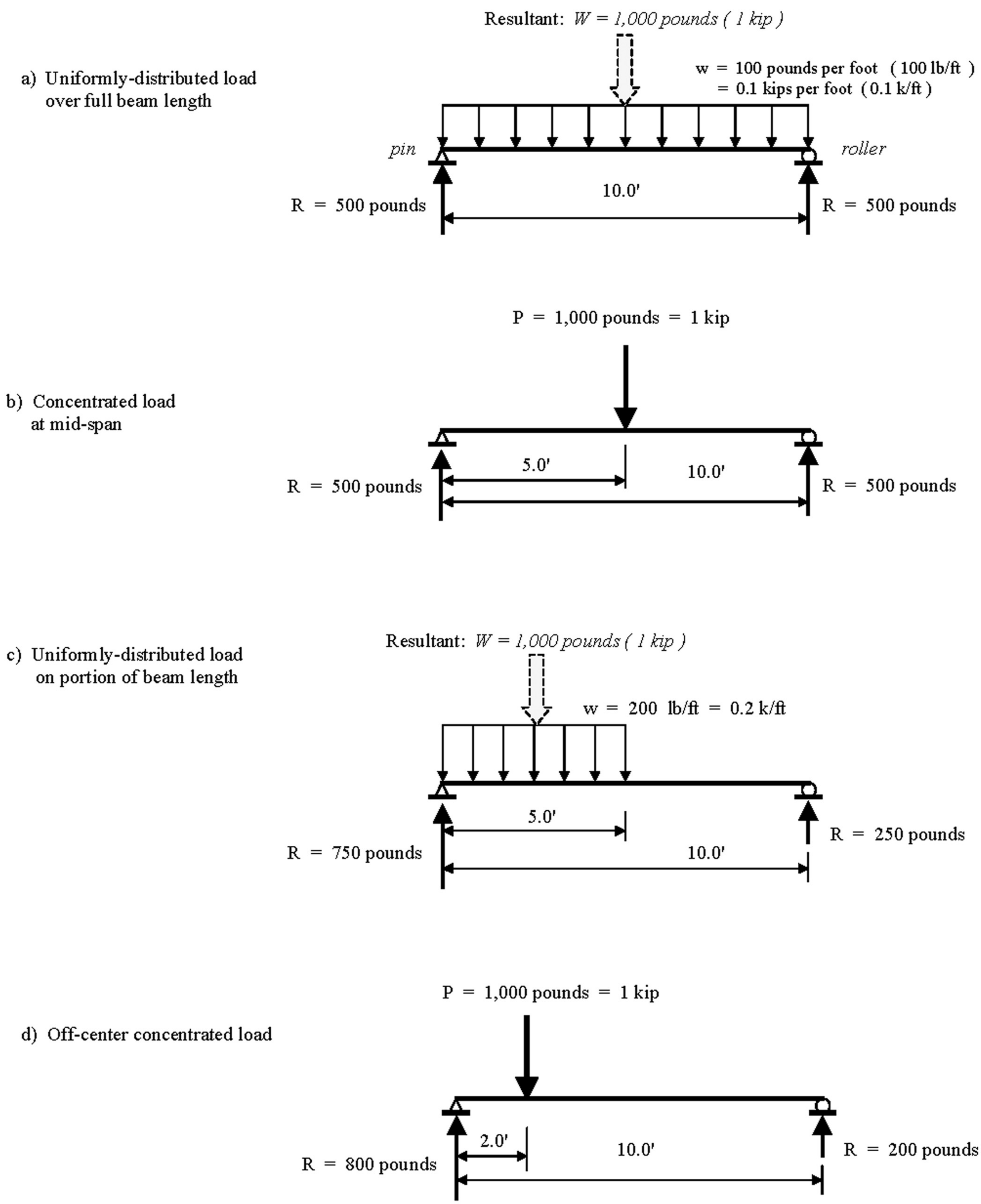


Figure B-7. The effect of load placement on beam reactions.

Other Loads

The third major class of loads is those produced by forces of nature. These include wind, earthquake, ice, snow and rainwater loads. Extensive work has been done in the areas of wind and earthquake loads to develop load models that reflect the randomness and variability of these loads.

LOADS (CLASSIFIED BY TYPE)

Loads can be classified by type, based on how the loads act on a structure. Figure B-7a shows a schematic diagram of a load that is uniformly distributed over the full length of the beam. This load could be caused from the weight of floor decking or distributed live loads. Since each 1-ft length of beam carries a 100-lb share of the load, the load is labeled as either 100 lb per foot (100 lb/ft) or 0.1 kip per foot (0.1 k/ft), where 1 kip equals 1,000 lb. The symbol w is used to denote a uniformly distributed load across the full length of the beam. The resultant (W) of the distributed load is the load intensity (load per foot) multiplied by the loaded length (feet): $W = wL = 100$ lb/ft $\times$ 10 ft $=$ 1,000 lb. The resultant is placed at the center of the loaded length (here, the midspan). The resultant is used to compute the beam reactions. Figure B-7b shows a schematic diagram of a 1000-lb (1-kip) concentrated load placed at the midspan of the beam. Although the magnitude of the concentrated load in Figure B-7b is the same as the resultant of the distributed load in Figure B-7a, and the reactions are the same in both cases, the bending stress in the beam caused by the concentrated load is twice that caused by the distributed load. Thus, a roof beam designed to support bar joists and roofing spread over the full length of the beam might not be able to support the bundled weights of these materials if they were placed at the middle of the beam during construction.

Figure B-7c shows another way that a uniformly distributed load can be applied to the beam. Here, the load intensity is the weight per foot (200 lb/ft), and the loaded length is $L/2$. The load resultant (W) is the load intensity (load per foot) multiplied by the loaded length (feet): $W = w(L/2) =$ 200 lb/ft $\times$ 5 ft $=$ 1,000 lb. It is located at the middle of the loaded length, which is at position $^1/_4$ of the distance from the left end ($L/4$).

Figure B-7d shows a 1-kip concentrated load applied at $L/4$ from the left end. Figure B-7d could represent a short beam supporting a column or cross-beam connected at the quarter-point.

In the four cases shown in Figure B-7, total load applied to the beam is 1,000 lb. In the first two cases, the beams are said to be symmetrically loaded because applied loads are symmetric with the beam mid-span. For this reason the reactions at the beam ends are equal. The beam capacity given in the tables in Part 3 of the AISC *Steel Construction Manual* assumes that each beam is loaded with a uniformly distributed load over its full length. The reaction at each end of the beam is $wL/2$. In Figures B-7c and B-7d, the beams are not loaded symmetrically so that the reactions are not equal. The larger reaction is the one nearest the applied load or resultant. It can be much greater than $wL/2$, but may not exceed the value of $\phi_v V_n$ or V_n/Ω_v given in Tables 3-6, 3-7, 3-8 and 3-9. The beam reactions must be computed, and the connections designed for the computed reactions.

EQUILIBRIUM

All structures and members must be in equilibrium and stable. Figure B-8a shows a beam that supports a single concentrated load that acts along an inclined line of action. The pin support at A and the roller support at B connect the rigid body to the rest of the world.

Reactions are forces that are provided by the structure's supports. In order for the beam in Figure B-8a not to move when the external force is applied, there must be enough reactions acting on it to keep it in balance—the beam must be in equilibrium. The beam is in equilibrium if, along any given axis, all forces add up to zero and all moments about any point in the beam add up to zero. A moment is the product of a force and a distance. A moment can be either clockwise or counterclockwise about any point on a member.

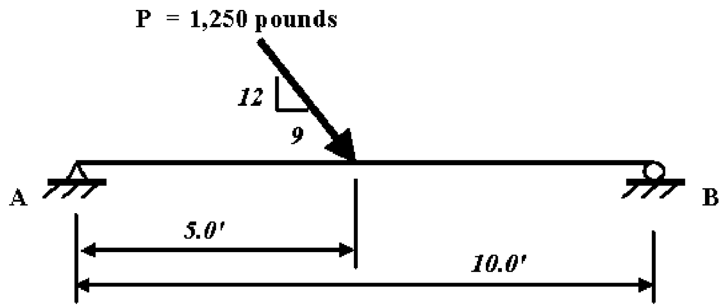
The requirement that any rigid body be in equilibrium is called the Law of Equilibrium. It is written in shorthand as equilibrium equations $\Sigma F_H = 0$, $\Sigma F_V = 0$ and $\Sigma M = 0$ (Σ is a math symbol meaning sum). Each equilibrium equation sum is an algebraic sum. For example, if forces shown acting to the right in the horizontal direction are assumed positive, then any force shown acting to the left is negative in the summation. Thus:

$F_H\,(+\rightarrow) = 0$ is a statement of equilibrium that says that "the sum of all horizontal forces (assuming forces acting to the right are positive) must be zero"

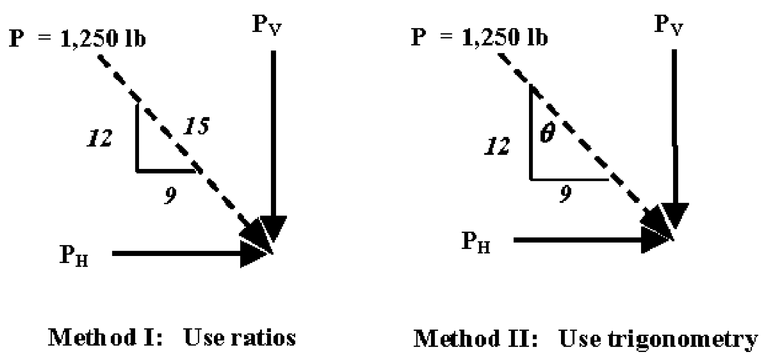
$F_V\,(+\rightarrow) = 0$ is a statement of equilibrium that says that "the sum of all vertical forces (assuming forces acting upward are positive) must be zero"

SMA ($\circlearrowright +$) $= 0$ is a statement of equilibrium that says that "the sum of all moments (assuming clockwise moments are positive) about point A must be zero"

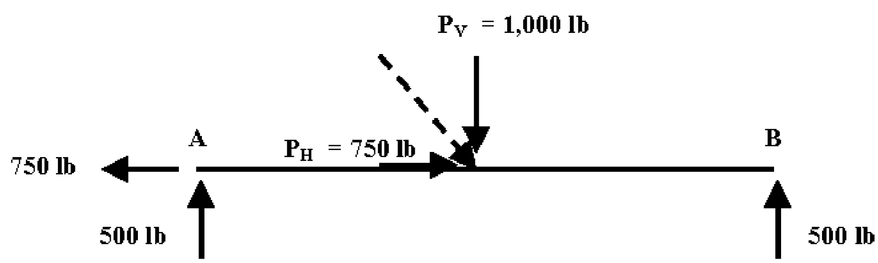
The single concentrated load applied to the beam in Figure B-8a acts on a "9 on 12" slope. It is convenient to break up the inclined load into vertical and horizontal components (P_V and P_H, respectively). This can be done two ways (Figure B-8b).



a) Schematic diagram



b) Resolve *P* into horizontal and vertical components
(two methods)



c) Reactions

Figure B-8. Simply supported beam reactions, on inclined concentrated load at midspan.

Method I: Use ratios based on "similar triangles"

$$\sqrt{12^2+9^2}=15$$

$$\frac{P_V}{p}=\frac{12}{15}\Rightarrow P_V=\left(\frac{12}{15}\right)P=\left(\frac{12}{15}\right)1250=1{,}000\text{ lb}$$

$$\frac{P_H}{P}=\frac{9}{15}\Rightarrow P_H=\left(\frac{9}{15}\right)P=\left(\frac{9}{15}\right)1250=750\text{ lb}$$

Method II: Trigonometry

$$\tan^{-1}\theta=\frac{9}{12}=0.75\Rightarrow\theta=36.9^{\circ}$$

$P_v = P\cos\theta = 1{,}250\cos 36.9° = 1{,}000$ lb

$P_H = P\sin\theta = 1{,}250\sin 36.9° = 750$ lb

The reactions at A and B shown in Figure B-8c hold the beam in equilibrium because:

$\Sigma F_H\,(+\rightarrow) = -750$ lb $+ 750$ lb $= 0$ lb

$\Sigma F_V\,(+\uparrow) = 500$ lb $- 1{,}000$ lb $+ 500$ lb $= 0$ lb

ΣM_A (?+) = (5 ft × 1,000 lb) − (10 ft × 500 lb) = 0 lb-ft

A plane truss is a collection of long, slender members assembled in triangular patterns to produce a two-dimensional structure (see Figures B-1 and B-2). A truss joint is any point where two or more truss members meet. External forces applied to a truss, and the reactions that hold the truss in equilibrium, are assumed to act on the truss at the joints. A truss distorts when loaded due to the combined effects of lengthening and shortening of the individual members. However, this distortion is small compared to the truss dimensions. Thus, it can be assumed that a truss generally retains its shape so that all distances used to compute forces and moments are those based on the truss dimensions.

Figure B-9a is a schematic diagram of a truss showing members as straight lines and joints as open circles. The symbol U denotes an upper joint and L a lower joint. The joints are numbered from left-to-right, starting with "0." In this case, joint L_0 is the "origin" (reference point) for joint numbering. The numbered vertical lines through the truss are sometimes referred to as panel points, and each horizontal segment between panel point lines is a panel. Truss equilibrium can be verified by applying the equations of equilibrium based on the applied loads:

$\Sigma F_H\,(+\rightarrow) = -5{,}000$ lb $+ 5{,}000$ lb $= 0$ lb

$\Sigma F_V\,(+\uparrow) = 10{,}500$ lb $- 6{,}000$ lb $- 9{,}000$ lb $+ 4{,}500$ lb $= 0$ lb

ΣM_{Lo} (↻+) = (18 ft × 5,000 lb) + (30 ft × 6,000 lb) + (60 ft × 9,000 lb) – (180 ft × 4,500 lb) = 0 lb-ft

INTERNAL FORCES

The external forces (the applied forces and the reaction forces) that hold the structure in equilibrium cause internal forces in the members of the structure. The internal force in each truss member is either a tension or a compression axial force, and is the same everywhere along the length of the member. For beams, the internal loads are moment and shear, and they vary along the length of the member. In this case, it is necessary to find the location of the maximum internal load.

Trusses

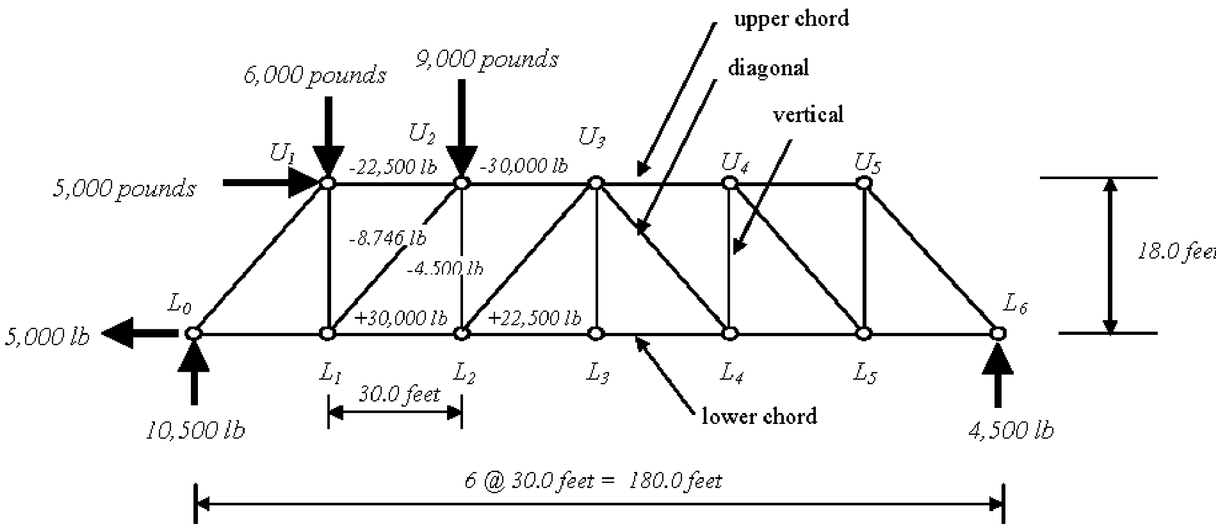
Figure B-9a shows the internal forces in selected members of a plane truss. The engineer computes these forces by performing a structural analysis of the truss. A positive member force means that the member is in tension (Figure B-1)—the member "pulls" on the joints to which it is connected, and the joints "pull back" on the member. A negative member force means that the member is in compression (Figure B-2)—the member "pushes" on the joints to which it is connected, and the joints "push back" back on the member. Figure B-9b shows the pin at joint U_2. The four members meeting at joint U_2 are all in compression, and hence push on the pin. The externally applied force pushes downward on the pin. It can be shown that horizontal components of the forces on the pin add up to zero ($\Sigma F_H = 0$), as do the vertical force components ($\Sigma F_V = 0$). Thus, the pin at joint U_2 is in equilibrium, as are the pins at every other joint in the truss.

Each set of external forces applied to a truss produces one unique set of member forces, and the member forces shown on a truss diagram will hold each joint in equilibrium. However, trusses are often designed for more than one set of loads. In this case, the design drawings may show the maximum force in each member produced by the multiple load sets. If the maximum forces in members meeting at a joint were caused by different load sets, then the joint will not be in equilibrium for the forces shown on the design drawing.

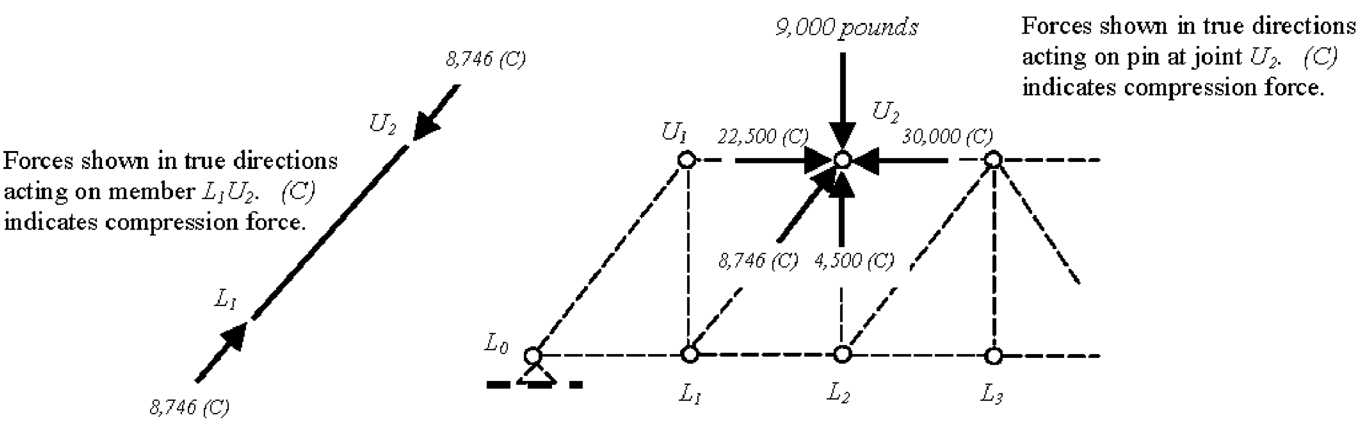
Trusses that support moving loads (such as bridge trusses) usually contain members that can be in either tension or compression depending on the position of the load on the truss. Connections for these members must be designed accordingly.

Beams

There are two internal loads, called moment and shear, produced in a beam by the external loads that bend (flex) the



a) Reactions and member forces



b) Forces acting on pin at joint U_2, and on member L_1U_2

Figure B-9. Tension and compression forces in truss members.

beam. These loads can be demonstrated by looking at how a beam deflects when the external loads are applied.

Figure B-10a shows a simply supported beam that is loaded with a generalized downward load that is represented by the load arrow at midspan (this load includes the weight of the beam). The beam is first divided into a number of imaginary equal-length segments (elements). In the figure, the elements are labeled A through M simply for reference.

When a downward load is applied, the beam deflects downward and forms a shape that is curved concave upward (Figure B-10b). In order for the beam to curve, each element must distort. The distortion consists of two parts: bending distortion and shear distortion.

Bending distortion causes the top of each element to shorten and the bottom to lengthen (shown in the left half of the beam in Figure B-10b). If the sides of the element are assumed to remain plane (so that they appear as straight lines when seen from the side of the beam), they must rotate inward so that the distorted element becomes a trapezoid. Piecing together the trapezoids produces the curved beam.

Each element can be distorted to form a trapezoid by "squeezing" the top of the element and "stretching" the bottom of the element by a pair of compression forces (C) and tension forces (T), as shown in Figure B-10c. In order for each element to be in horizontal equilibrium, C and T on each face of the element must be equal to each other. The elements along the length of the beam do not distort the same amount, and C and T are different on each element. For the beam loaded as shown, the elements at the center of the beam distort more (and are subjected to greater C and T forces) than the elements at the ends.

For a typical element (such as element G in Figure B-10c), the distortion caused by the C and T forces could also be caused by the pair of bending force effects, M, shown in Figure B-10d. The bending axes are the horizontal arms at the level of the center of gravity of the element cross section. Each bending force is called a moment. A moment is statically equivalent to the C-T force pair that it represents. Hence, instead of showing the C-T forces acting on the side face of the element, the moment can be shown instead. The magnitudes of the internal moments vary along the length of the beam, and can be plotted to form a moment (M) diagram.

The downward load on the beam also causes each element to distort as shown in the right half of the beam in Figure B-10b. Vertical forces on the sides of each element cause it to distort to form a parallelogram. These forces are called shear forces, and the distortion is called shear distortion. Shear force is represented by the symbol "V." The magnitudes of the internal shear loads vary along the length of the beam, and can be plotted to form a shear (V) diagram.

The shear forces shown in Figure B-10 act on the vertical faces of the beam elements, and hence are referred to as vertical shear. Figure B-11 shows that shear can also be produced in the horizontal direction when a beam bends. Horizontal shear is the internal resistance to "sliding" the upper portion of the beam relative to the bottom portion, much the same as the pages in a book slide when the book is bent. The sliding effect is greater at the ends of the beam than at the middle. In Figure B-11a, each pair of points A-A', B-B' and C-C', which were adjacent before bending, are horizontally offset after bending. The offset is greatest at the end of the beam, and smallest (or even zero) at the middle. The beam is much more efficient if the top and bottom of the beam can be forced to "work together," as shown in Figure B-11b. The internal load that is developed between the top and the bottom as they work together is the horizontal shear (Figure B-11c).

Horizontal shear is very important in the design of steel-concrete composite beams (Figure B-12a). The steel W-shape and the concrete slab shown in the figure work together to form a single cross-section. The internal compression load caused by bending pushes on the concrete slab and the internal tension load pulls on the steel W-shape, thus trying to cause them to slip along the intersection between the two (the shaded area in Figure B-12a). Shear studs welded to the top flange of the beam and extending into the concrete slab force the steel and concrete to work together by preventing this slip. The studs are spaced more closely at the ends of the beam than at the middle because the tendency for slip is greater at the ends of the beam than at the middle.

Shear is also important in the design of built-up beams and plate girders (Figure B-12b). Horizontal shear and vertical shear can act together in the webs of these sections to cause web buckling, which is a "crumpling" of the web. Vertical stiffeners are used to reinforce the web to prevent web buckling. The stiffeners are usually spaced more closely at the ends of built-up sections than near mid-span because shear is greatest at the ends.

Both moment and shear can be present at every point along the length of the beam. Usually, shear is the dominant internal load at the ends of a beam and moment is the dominant internal load in the middle portions of a beam. Most steel wide-flange shapes are selected based on the maximum moment they will experience, and then checked for shear.

Figure B-13a shows a simply supported beam with a single concentrated load at midspan, and the reactions computed for this load case. Figure B-13b shows a segment of the beam formed by making a section cut at point A that is midway between the left end and the load, P, at the center of the beam. The external load, R_L, and internal loads, V_A and M_A, at the location of the cut are shown on this segment. The internal shear load, V_A, must be downward (and equal to reaction R_L) for the segment to be in vertical equilibrium. The internal moment, M_A, must be counterclockwise in order for the segment to be in rotational equilibrium ($\Sigma M = 0$). Summing moments

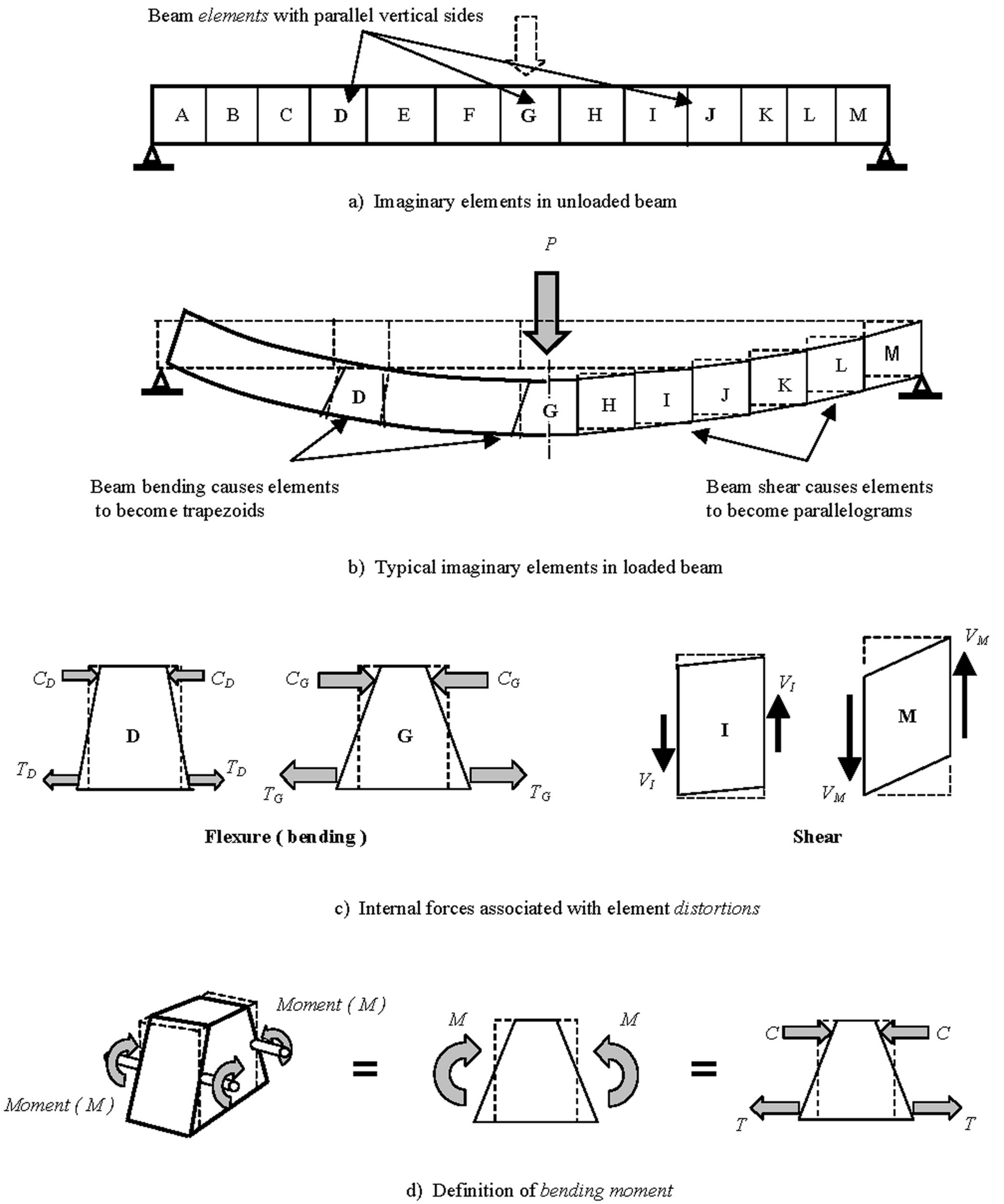
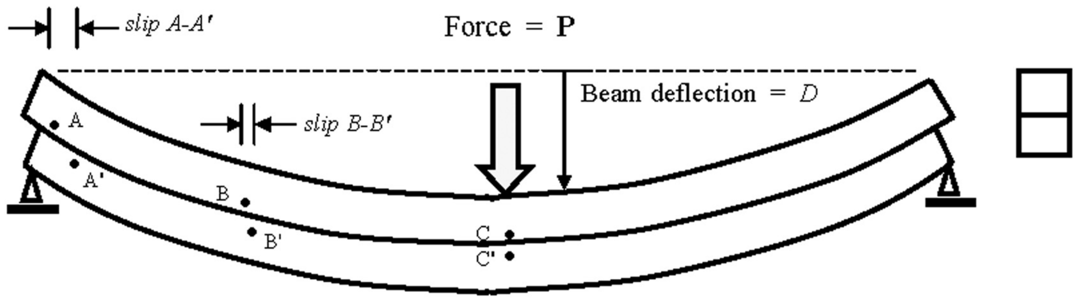
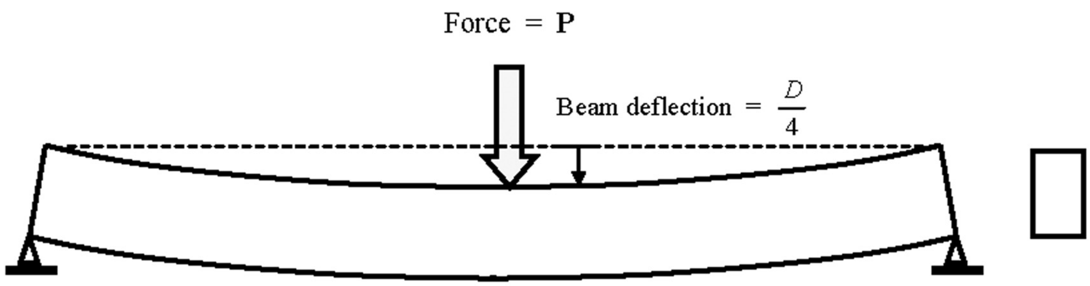


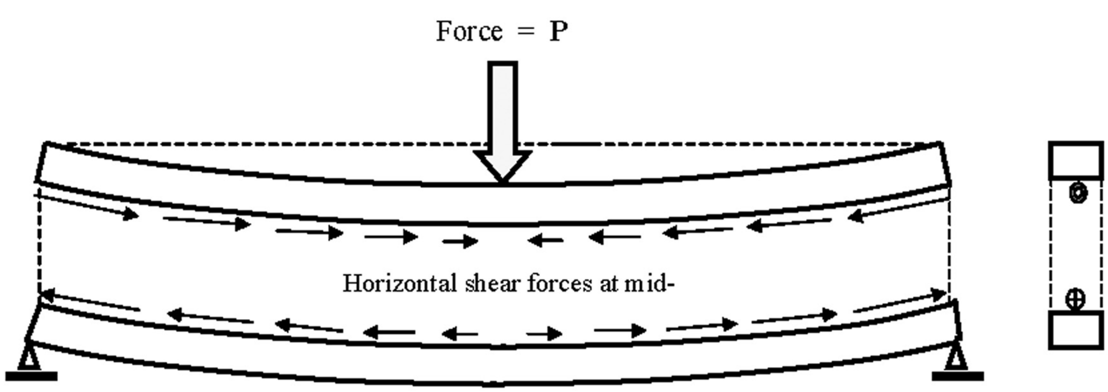
Figure B-10. Moment and vertical shear in beams.



a) Effect of horizontal slip at mid-depth

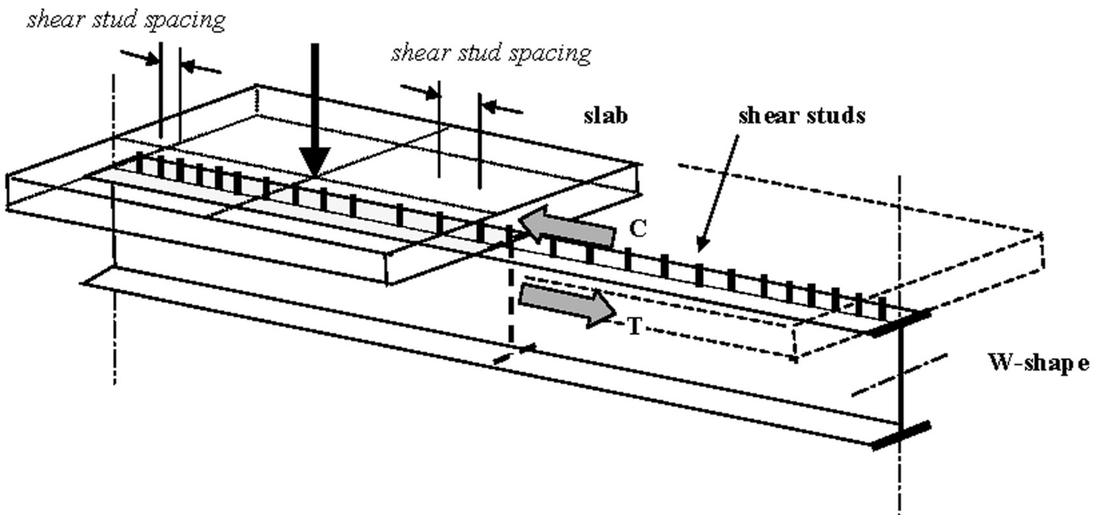


b) Eliminating horizontal slip increases beam efficiency

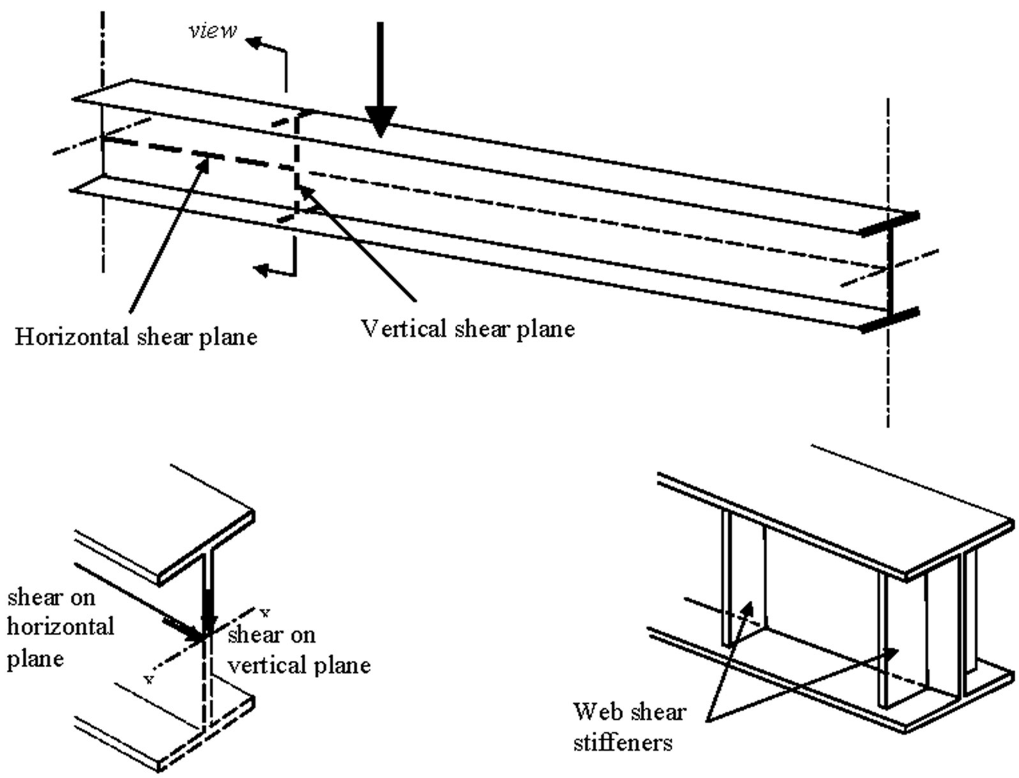


c) Horizontal shear forces (maximum at mid-depth)

Figure B-11. Horizontal shear in beams.



a) Shear studs in steel-concrete composite beams



b) Vertical web shear stiffeners in built-up beams and plate girders

Figure B-12. Shear details in beams.

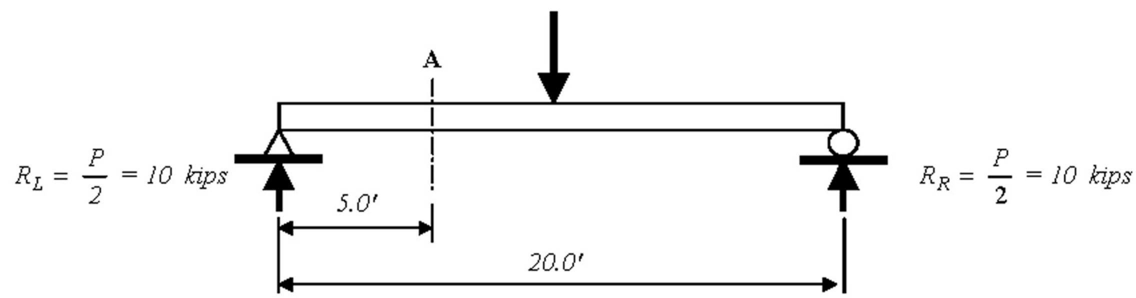
about point A ($\Sigma M_A = 0$) involves two loads that produce moments about point A. These are the reaction, R_L, and the moment, M_A, which is one of the internal loads "exposed" by the section cut. The other load, shear V_A, produces no moment about point A because it passes through that point. It is very important to note that the symbol "M_A" is used two ways. First, M_A is a specific internal load applied at point A. Secondly, $\Sigma M_A = 0$ is an equilibrium statement that says "the sum of the moments about point A caused by all loads, including the internal moment, M_A, acting at point A, must add up to zero."

Table 3-23, Shears, Moments and Deflections on pages 3-211 through 3-226 of the *Steel Construction Manual*, 13th Edition, gives shear and moment diagrams for many typical beam configurations and load cases. Also included in Table 3-23 are expressions for the magnitude of the maximum moment for many cases.

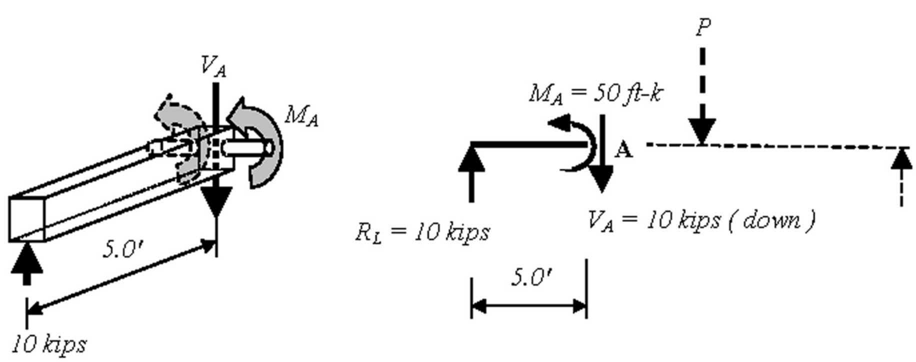
STRESSES

The tension and compression loads in truss members, and the shears and moments in beams, are the internal loads in these members caused by the external loads acting on the structure. The stresses that these loads produce are a way of measuring the intensity of these internal loads. In each case, the internal load in each member is the resultant (sum) of the stresses acting on the member cross-section surface.

$$f_a = \frac{P\,(\text{kips})}{A\,(\text{in.}^2)}$$



a) Idealized beam and reactions



b) Segment formed by section cut midway between left reaction and load P

Figure B-13. Internal shear and moment forces in a beam.

Figure B-14 shows a member loaded with axial tension load, T. The axial stress that this load produces is computed by dividing the load by the cross-sectional area of the section. If the load is in kips and the area is square inches, then the resulting stress has units of kips/in.^{2} or ksi (kips per square inch).

The symbol f is used to denote applied stress (i.e., stress caused by the external loads on the structure). The subscript a indicates that the stress is axial stress, one that is caused by either tension or compression. The axial stress, f_a, is uniform (has the same intensity) over the entire cross section. The resultant of the tension stress at any cross-section is $T = A\,f_a$, which is the internal load in the member. This resultant, and the axial stress it represents, is the same for a section cut taken at any point on the member.

Figure B-15 shows a W-shape bending member for whom the internal shear load at a section along the length of the beam is V. The average shear stress in the web of the W-shape is found by dividing the shear load, V, by a shear area equal to the overall section depth, d, multiplied by the web thickness, t_w:

$$f_b = \frac{V}{dt_w}\left[\text{units: kips/in.}^2\right]$$

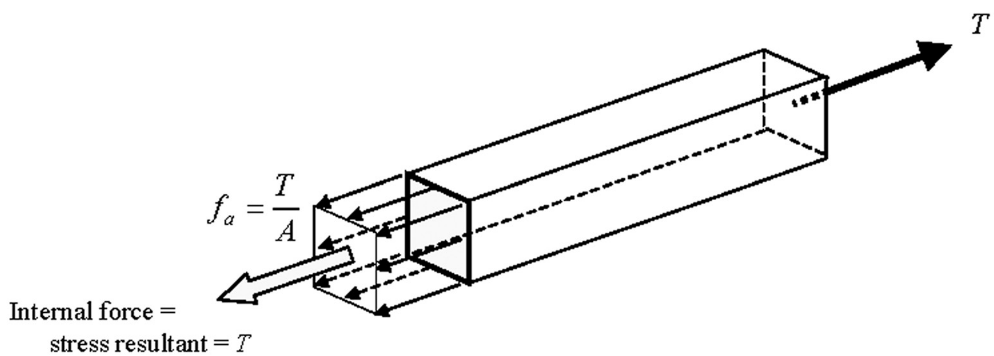


Figure B-14. Stress distribution in a tension member.

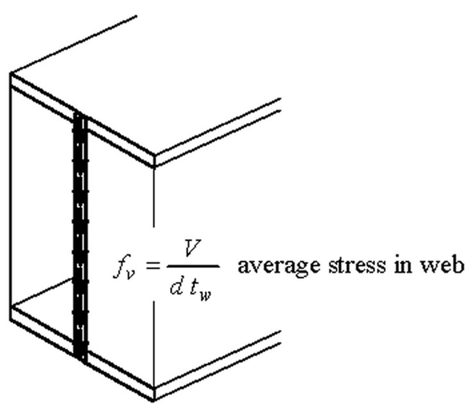


Figure B-15. Shear stress distribution in a W-shape steel beam.

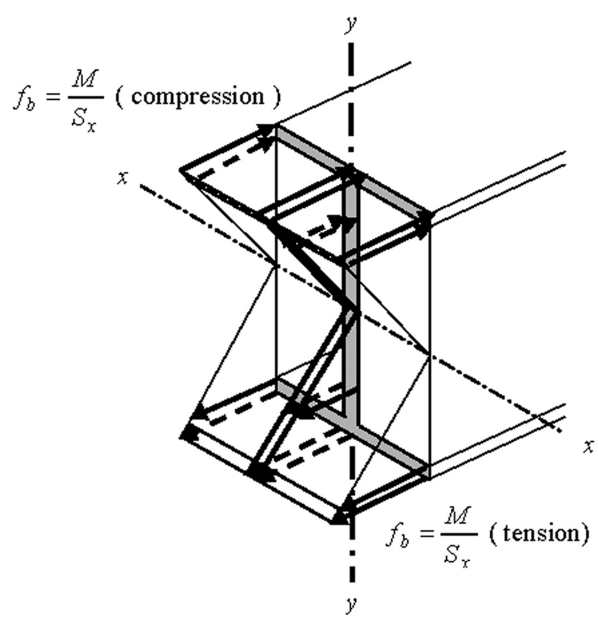


Figure B-16. Bending (flexural) stress distribution in a W-shape steel beam.

Figure B-16 shows a bending member for which the internal moment at a section along the length of the beam is M. As was shown in Figure B-10, the internal moment on each face of each beam element is equivalent to a pair of forces C and T that cause the element to deform to a trapezoid. The internal flexural (bending) stresses caused by internal moment, M, act perpendicular to the face of the cross-section area, and are distributed over this area in the pattern shown in the figure. The maximum flexural stress occurs at the upper and lower edges of the cross-section, and the flexural stress at the center of gravity of the cross-section (which is the mid-depth of the beams shown in Figure B-16) is zero. The bending axis (shown as horizontal arms in Figures B-10) passes through the center of gravity and lies on the line of zero stress. This is often called the neutral axis.

The maximum flexural stress is denoted as f_b. It is computed by dividing the bending moment by the section modulus, S, of the cross section:

$$f_b = \frac{M\left(\text{in.- kips}\right)}{S\left(\text{in.}^3\right)}\left[\text{units: kips /in.}^2\right]$$

The steel W-shapes shown in Part 1 of the *Steel Construction Manual* are shown oriented with the flanges horizontal and an x-axis for these sections parallel to the flanges. The maximum flexural stress, f_b, for a W-shape shown in Figure B-16 is "spread" horizontally the full width of the flanges. The stress at the inner edge of each flange is slightly smaller than f_b at the outer edge of the flange. Between the flanges, the stress is "spread" over the width of the web. The stress diminishes to zero at the neutral axis. For W-shapes the neutral axis is the x-axis at the mid-depth of the beam. Placing the majority of the cross-section area in the flanges where the stress is greatest increases the efficiency of the moment resistance for W-shapes compared to rectangular shapes. This is why W-shapes are manufactured this way. Thus, whenever possible, steel beams are oriented with the webs vertical so that gravity loads produce bending about the stronger x-axis.

In summary, there are two types of stress that can act on a surface: normal stress (perpendicular to the surface) and shear stress (parallel to the surface). However, as seen in Figure B-16, stress f_b is part of a set of normal stresses that add up to produce a bending effect. This is the reason f_b is called a "bending (flexural) stress." Similarly, internal stresses in a member loaded in tension are normal stresses that act along the longitudinal axis of the member. Torsional stresses are produced on the cross-section of a member twisted by a torsional moment. The words "flexural," "axial," "torsional" used in conjunction with the word "stress" tells the type of internal load in the member to which these stresses are related.

ENGINEERING PROPERTIES OF STEEL

Structural steel has three characteristics that make it an extremely versatile building material: strength, stiffness and ductility. These characteristics can be demonstrated by observing the behavior of an idealized model of the simple tension member shown in Figure B-17.

As the tension load, P, is gradually increased, the member stretches in the loaded direction and contracts proportionally in its cross section. In Figure B-17, it is assumed that force P causes an elongation whose value is an amount ΔL (change in length). Stiffness is the resistance to elongation. If the member were made stiffer (such as by increasing the cross-section area), it would take greater force P to produce the same ΔL.

For load stages [1] through [3], stretching the tension member is like stretching a rubber band. The elongation of the member is proportional to the applied force. Another characteristic of elastic behavior is that if the member was loaded to force $2.5P$ (causing it to elongate to $2.5\Delta L$) and then unloaded, the member would return to its original length. This is called elastic behavior.

For the steel tension member, there is an upper limit on the force for which it will behave elastically. In Figure B-17, this force is randomly assigned the value of $3P$. At a force just slightly greater than theoretical force $3P$, the member will stretch (and continue to stretch) without application of any additional load. A portion of the length of the member will dramatically contract (neck down). The stretching will continue as long as the force is maintained at essentially $3P$. This increase in elongation (with the associated contraction) at constant force is called yielding, and is similar to pulling taffy. If the member were to elongate to the length shown in stage [4] and the load then removed, it would not return to its original length. It would shorten by the amount that it had stretched elastically (elastic recovery), but the increase in length due to yielding would not be 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 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-sectional area) and its length. It has already been shown that dividing the force by the cross-sectional 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 inches per inch 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 inch (ε), the greater the overall elongation, ΔL, caused by the force. Figure 1-3 (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

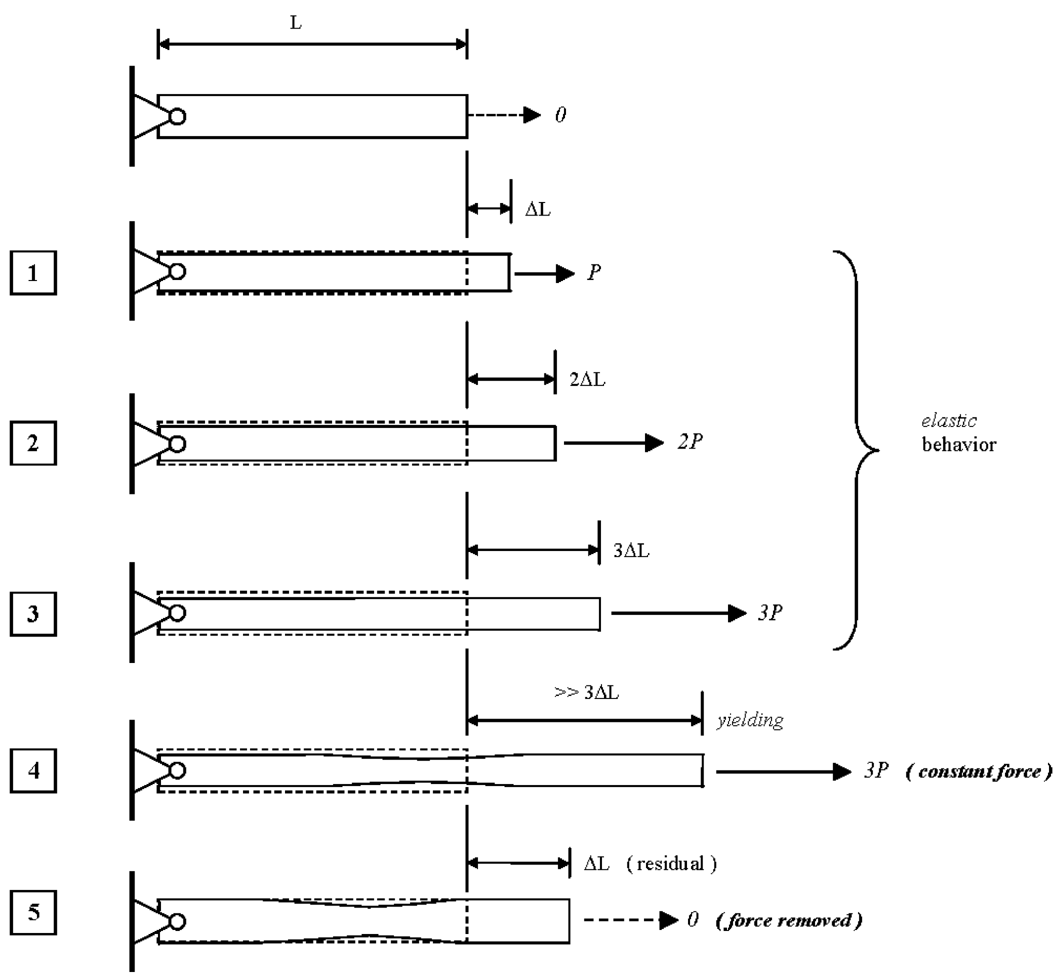


Figure B-17. Yielding failure of a steel tension member.

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 required strength would be exceeded and the beam would be said to have reached a strength limit state. The required strength is based on maximum expected day-to-day loads that are determined according to ASCE 7 load combinations.

LRFD load combinations contain load factors, which 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 a steel beam that supports 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 2005 AISC *Specification for Steel Buildings* does not include load combinations (as was done in editions of the *LRFD Specification* prior to 1999), 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 load at which failure will occur. In these models, tension members fail by yielding, columns fail by buckling and connections fail by either yielding or rupture (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 *Steel Construction Manual* includes the 2005 AISC *Specification for Structural Steel Buildings*, hereafter referred to as the AISC *Specification*, which 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. Both Allowable Stress Design and Load and Resistance Factor Design are included.

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 produced in a tension member is uniform over the entire cross-section of the member, and it is the same at any section cut made along the length of the member. If the external force, T, were increased until the stress reached F_y, the entire cross-section would yield. Yielding is a strength limit state for a tension member because once the member yields its usefulness as a load carrying member is ended. From AISC *Specification* Chapter D, the nominal tensile yield strength, T_n, for a tension member is:

$$T_n = F_y A$$

This equation is the strength model that says a tension member will fail by yielding at a force that is equal to the cross-section area multiplied by the yield stress. Any member that is in compression (such as the compression member in Figure B-2) is susceptible to buckling, as shown in Figure B-18. The average compressive stress never reaches yielding because the member buckles before the internal stress reaches the yield stress. Dividing the force at which buckling occurs by the area gives the critical stress,

$$f_{cr} = \frac{P_{buckling}}{A}$$

The critical stress depends on the length of the compression member, its cross-section properties and how the member is used in the structure. AISC *Specification* Chapter E contains the engineering models used to predict the critical stress, F_{cr}, for compression members. Once F_{cr} is found using the appropriate equations in Chapter E, the nominal strength, P_n, for compression then becomes:

$$P_n = F_{cr} A$$

Figure B-16 shows the flexural internal stress distribution in a W-shape beam (these are the stresses produced by service loads). A simplified view of this stress distribution is shown in Figure B-19a. The nominal flexural strength of a beam is the internal bending moment at which the beam will fail. Increasing the externally applied loads causes the internal moment to increase, thus increasing the maximum stress, f_b. At the yield moment, M_y, the maximum stress $f_b = F_y$ (Figure B-19b). Unlike the tension member, whose entire cross section yields (thus "fails") as soon as $T_y = F_{yA}$, the bending member does not fail when $M_y = SF_y$ because only a small portion of the cross-section has yielded. Increasing the bending moment beyond M_y causes the section to yield progressively (Figure B-19c) until full yielding occurs (Figure B-19d). The bending moment at which full-depth yielding occurs (the beam "fails by yielding") is the plastic moment, M_p:

$$M_p = F_y Z_x$$

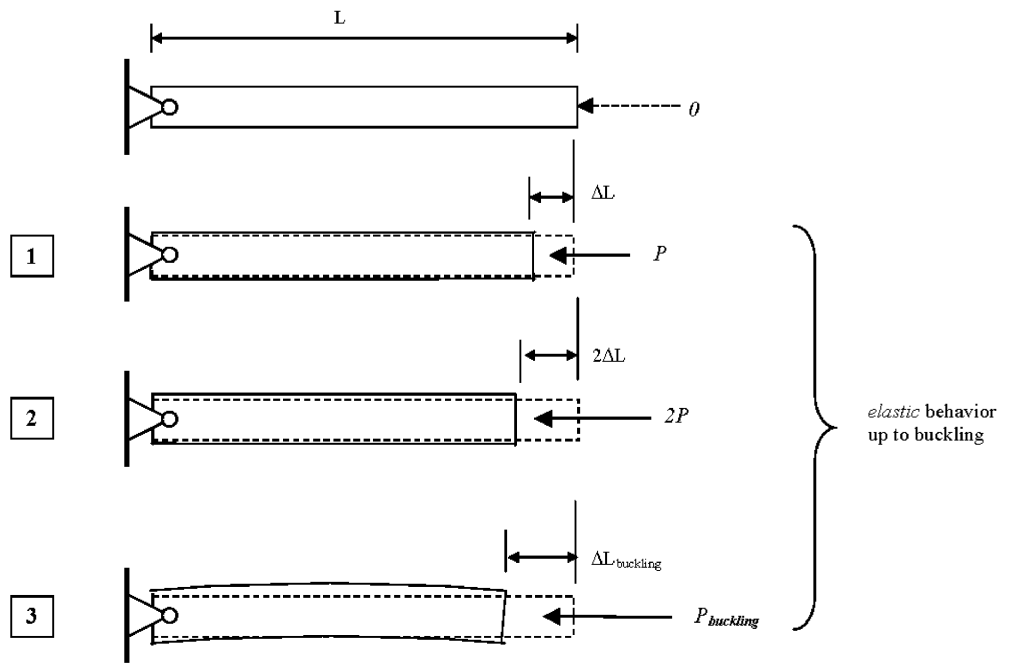


Figure B-18. Buckling failure of a steel compression member.

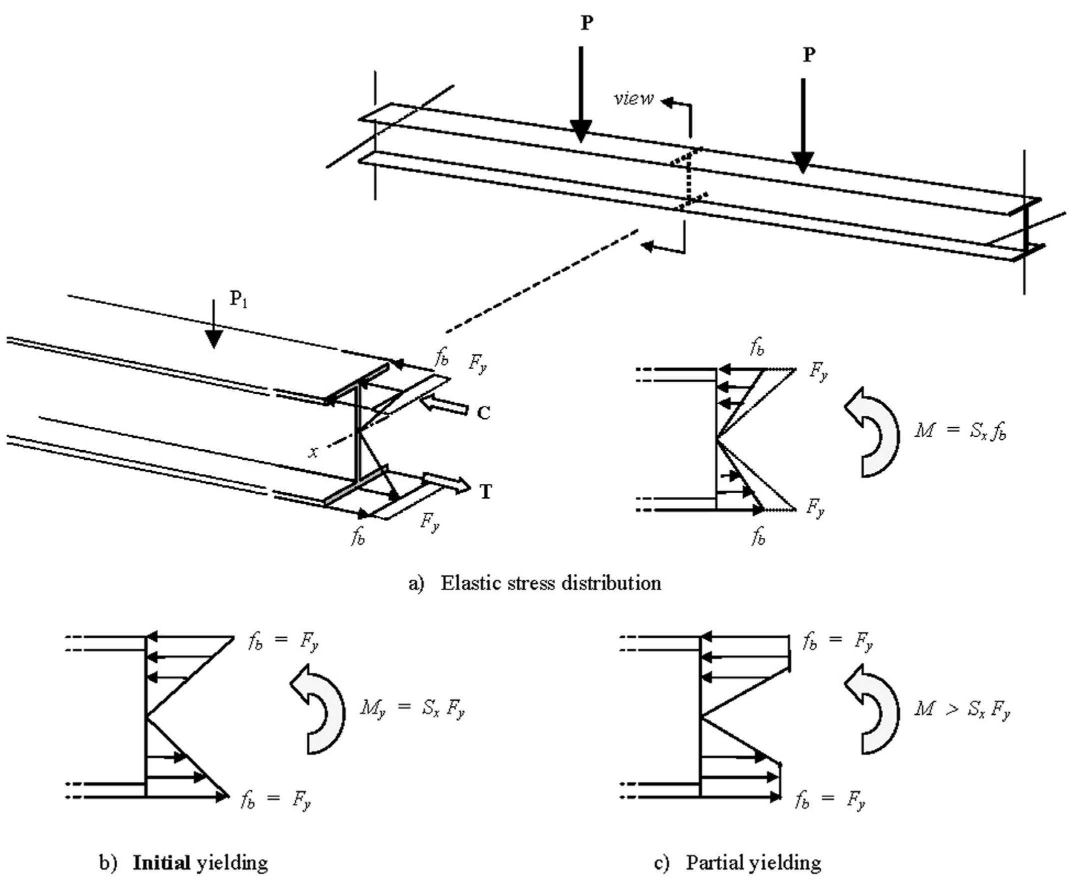


Figure B-19. Yielding failure of a steel beam.

where Z_x is the plastic modulus for bending about the x-axis. The Z_x values for steel shapes are contained in Part 1 of the LRFD Manual.

The maximum possible nominal strength for any particular beam is based on the yielding limit state just shown. However, lateral-torsional buckling can cause a beam to fail at internal moments less than M_p. Lateral-torsional buckling occurs when the compression in the beam causes it to simultaneously buckle sideways and twist. Lateral-torsional buckling can be prevented by bracing the compression flange to prevent it from moving sideways (Figure B-20). In this figure, the beam is braced so that its web (shaded area) remains in the vertical plane as load is applied and the beam deflects. The unbraced length of a beam is the distance between braced points. If the unbraced length is small enough, the internal moment can reach the plastic moment, M_p, before the beam buckles. It is important to see that lateral-torsional buckling is effected by how a beam is used in the structure. Members that frame into the beam (particularly members that connect to the side of the beam that is in compression) are often used to prevent lateral-torsional buckling. For example, bar joists welded to the upper flange of floor or roof beams are generally spaced close enough to prevent buckling. In this case, the unbraced beam length is the bar joist spacing.

It is emphasized that the strength equations in the AISC *Specification* that give the nominal strengths of steel members are only models. The accuracy of each model depends on how well it predicts the failure strength for every possible member of the type for which the model is to be used. The accuracy of the model depends on things like the possible man-

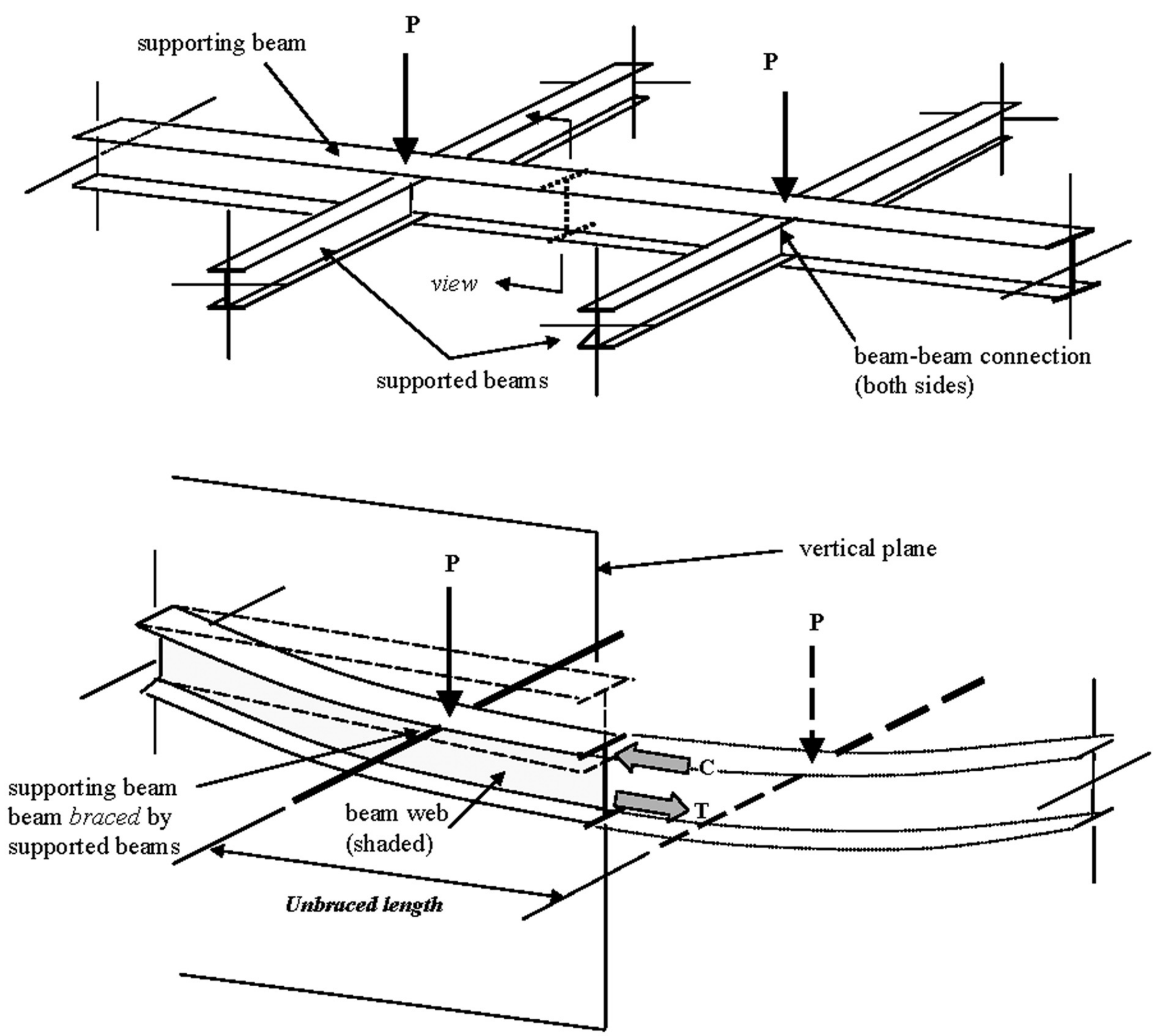


Figure B-20. Beam bracing to prevent out-of-plane buckling (lateral-torsional buckling).

ufacturing variations in the member that could affect the actual cross-sectional properties used in the model and the actual yield stress, F_y, for the particular batch of steel from which the member is made. Some models (such as those that predict beam buckling and column buckling strengths) are sensitive to the type, size and use conditions of the members. For these reasons, the LRFD procedure applies a resistance factor, ϕ, to the nominal strength predicted by the model. This is done to offset variations or inaccuracies in the nominal strength, whatever the cause. Thus, ϕT_n is the design tensile strength of a tension member based on yielding ($\phi = 0.90$; $T_n = F_y A$), ϕM_n is the design flexural strength of a beam ($\phi = 0.90$), ϕV_n is the design shear strength of a beam ($\phi = 0.90$ or 1.00, depending on the member type), and ϕP_n is the design compressive strength of a column ($\phi = 0.90$). In addition, $\phi = 0.75$ is the resistance factor applied to fracture strengths computed for connections. Numerous design aids are provided in the various sections of the *Steel Construction Manual* to assist the engineer in computing the design strength for each limit state for each member type.

Load and Resistance Factor Design states that the strength limit state of a member is satisfied if the design strength of the member equals or exceeds the required strength based on the factored load combinations. Thus, for floor beams that support dead and live loads, the strength limit states require that:

	Required Strength		Design Strength
Shear:	$1.2V_{DL} + 1.6V_{LL}$	$\leq$	ϕV_n
Moment:	$1.2M_{DL} + 1.6M_{LL}$	$\leq$	ϕM_n

where V_{DL}, V_{LL}, M_{DL} and M_{LL} are the maximum internal shear and moment caused by the applied loads, and V_n and M_n are the nominal strengths based on the critical limit states (yielding, buckling).

Deflection is a serviceability limit state. For the floor beam example, a typical deflection requirement is that the deflections produced by the live service load not exceed the beam span length divided by 360. Deflection is generally not a safety issue, but rather is a limitation imposed based on how the structure is to be used.

The information contained in this appendix is supplied to give the steel detailer only a very basic understanding of the principles used in engineering steel structures. The information is for general knowledge purposes only. Engineering design should be completed or approved by a licensed engineer.

REFERENCE

ASCE (2006), *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-05, American Society of Civil Engineers.

APPENDIX C
ELECTRONIC DATA EXCHANGE

The Use of Electronic Drawings by Detailers

The role of computers in the development, storage and transfer of information traditionally communicated on drawings has grown at an unprecedented pace. Technology continues to develop, and adoption of electronic drafting and data transfer in various segments of the steel construction industry continues to grow. The delivery of steel to a project is critical to many construction schedules, and the advancement of technology in the design and construction industry is a response to a growing demand of the construction market to complete projects faster and cheaper than ever before. In order to remain competitive in a business environment that seeks improved productivity and agility, members of the construction community must continue to develop effective practices within their organizations and must participate in industry efforts to coordinate data exchange between various segments of the design and construction community.

The technology available and commonly used in the development of electronic drafting information varies widely from libraries of standard details in general drawing programs to sophisticated programs that generate three-dimensional models, accept and calculate loads, draw the appropriate connections and ultimately communicate with machinery to fabricate the steel. The specific technology used by various participants in the construction community varies significantly, depending on their position in the industry and ability to adopt available technology into their existing methods of conducting business. The application and utilization of technology will continue to grow, and all members of the steel industry must work together to ensure that the transition is productive and ultimately successful. Little can be said in a static text that will be helpful in developing future technology. However, the successful resolution of issues involved in data transfer between companies will pave the way for a smooth transition from traditional methods to the adoption of developing technology.

DIRECT BENEFITS OF INFORMATION SHARING

The work of detailers is greatly aided by the use of information sharing. Potential benefits for detailers include:

- Time saved to enter the building frame into detailing software.
- Easy adaptation of layered electronic plans into erection drawings by "turning off" unnecessary levels of detail.
- Accurate realization of the design intent into the final product, especially in the case of highly complex or irregular designs.
- Accurate transfer of structural loadings to the fabricator for connection design.

Dominant issues involved in the efficient use of detailing technology include format, scale and quality of information.

DATA FORMAT

What is a data format? Data format refers to the way that information is delivered in a data set—fonts, numbering, degree of detail, etc. Data formats in electronic drawings are analogous to drafting standards in hand detailing. While each independent producer of drawings could work with an independent standard, information sharing would benefit from one unified format. The term "data format" is not to be confused with the term "file format" (such as DXF), which refers to the computer file structure for the reading of information by computer programs, which is outside of the control of the typical designer or detailer.

AISC has evaluated available data format standards and believes that CIS/2 is the most effective neutral data file format for design and construction information exchange available. The CIS/2 data format, developed in Europe, is a method of accurately conveying all necessary information for a project with a degree of consistency and accuracy that allows it to be used by all members of any given project team, when they have obtained written permission to do this from the Owner's Designated Representative for Design. The use of the CIS/2 data format allows information that is calculated for the development of a design but is not presented explicitly on a drawing to be stored with the drawing data file for future use on another portion of the project. All loads—individual loads, factored loads, service loads and combined loads—can be stored with the drawings, supplying the fabricator with all of the information necessary for the accurate creation of connection details.

If properly utilized, consistent CIS/2 data formatting can save weeks in a project schedule. The benefits of this system will be most easily realized in design-build project delivery systems, where structural steel is often on the critical path of a construction schedule and the underlying goal of the project is mutual benefit derived from working together toward greater overall efficiency, or in a complex 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 for 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 superseded 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 for Steel Buildings and Bridges*, 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, meaning 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.

APPENDIX D
SI UNITS FOR STRUCTURAL STEEL DESIGN

Although there are seven metric base units in the SI system, only four are currently used by AISC in structural steel design. These four units are listed in Table D1.

TABLE D1. BASE SI UNITS FOR STEEL DESIGN

Quantity	Unit	Symbol
length	meter	m
mass	kilogram	kg
time	second	s
temperature	celsius	°C

Similarly, of the numerous decimal prefixes included in the SI system, only three are used in steel design; see Table D2.

TABLE D2. SI PREFIXES FOR STEEL DESIGN

Prefix	Symbol	Order of Magnitude	Expression
mega	M	10^6	1,000,000 (one million)
kilo	k	10^3	1,000 (one thousand)
milli	m	10^{-3}	0.001 (one thousandth)

In addition, three derived units are applicable to the present conversion. They are shown in Table D3.

TABLE D3. DERIVED SI UNITS FOR STEEL DESIGN

Quantity	Name	Symbol	Expression
force	newton	N	$N = kg \times m/s^2$
stress	pascal	Pa	$Pa = N/m^2$
energy	joule	J	$J = N \times m$

Although specified in SI, the pascal is not universally accepted as the unit of stress. Because section properties are expressed in millimeters, it is more convenient to express stress in newtons per square millimeter ($1\ N/mm^2 = 1\ MPa$). This is the practice followed in recent international structural design standards. It should be noted that the joule, as the unit of energy, is used to express energy absorption requirements for impact tests. Moments are expressed in terms of $N \times m$.

A summary of the conversion factors relating traditional U.S. units of measurement to the corresponding SI units is given in Table D4.

TABLE D4. SUMMARY OF SI CONVERSION FACTORS		
Multiply	**By:**	**To obtain:**
inch (in.)	25.4	millimeters (mm)
foot (ft)	305	millimeters (mm)
pound-mass (lb)	0.454	klogram (kg)
pound-force (lbf)	4.448	newton (N)
ksi	6.895	N/mm^2
ft-lbf	1.356	joule (J)
psf	47.88	N/m^2
plf	14.59	N/m

Note that fractions resulting from metric conversion should be rounded to whole millimeters. Common factors of inches and their metric equivalent are in Table D5.

TABLE D5. SI EQUIVALENT OF FRACTIONS OF AN INCH		
Fraction, in.	**Exact conversion, mm**	**Rounded to: (mm)**
$^1/_{16}$	1.5875	2
$^1/_8$	3.175	3
$^3/_{16}$	4.7625	5
$^1/_4$	6.35	6
$^5/_{16}$	7.9375	8
$^3/_8$	9.525	10
$^7/_{16}$	11.1125	11
$^1/_2$	12.7	13
$^5/_8$	15.875	16
$^3/_4$	19.05	19
$^7/_8$	22.225	22
1	25.4	25

Bolt diameters are taken directly from the ASTM Specifications A325M and A490M rather than converting the diameters of bolts dimensioned in inches. The metric bolt designations are in Table D6.

TABLE D6. SI BOLT DESIGNATION		
Designation	**Diameter, mm**	**Diameter, in.**
M16	16	0.63
M20	20	0.79
M22	22	0.87
M24	24	0.94
M27	27	1.06
M30	30	1.18
M36	36	1.42

The yield strengths of structural steels are taken from the metric ASTM Specifications. It should be noted that the yield points are slightly different from the traditional values. See Table D7. The modulus of elasticity of steel E is taken as 200,000 N/mm^2. The shear modulus of elasticity of steel G is 77,000 N/mm^2.

TABLE D7. SI YIELD STRESSES

ASTM Designation	Yield stress, N/mm^2	Yiels stress, ksi
A36M	250	36.26
A572M Gr. 345	345	50.04
A588M		
A852M	485	70.34
A514M	690	100.07

TABLE D8. WEIGHTS AND MEASURES INTERNATIONAL SYSTEM OF UNITS (SI)[a] (METRIC PRACTICE)

Base Units

Quantity	*Unit*	*Symbol*
length	meter	m
mass	kilogram	kg
time	second	s
electric current	ampere	A
thermodynamic temperature	kelvin	K
amount of substance	mole	mol
luminous intensity	candela	cd

Supplementary Units

Quantity	*Unit*	*Symbol*
plane angle	radian	rad
solid angle	steradian	sr

DERIVED UNITS (WITH SPECIAL NAMES)

Quantity	*Unit*	*Symbol*	*Formula*
force	newton	N	$kg\text{-}m/s^2$
pressure, stress	pascal	Pa	N/m^2
energy, work, quantity of heat	joule	J	N-m
power	watt	W	J/s

DERIVED UNITS (WITH SPECIAL NAMES)

Quantity	*Unit*	*Formula*
area	square meter	m^2
volume	cubic meter	m^3
velocity	meter per second	m/s
acceleration	meter per second squared	m/s^2
specific volume	cubic meter per kilogram	m^3/kg
density	kilogram per cubic meter	kg/m^3

SI PREFIXES

Multiplication Factor	*Prefix*	*Symbol*
1 000 000 000 000 000 000 = 10^{18}	exa	E
1 000 000 000 000 000 = 10^{15}	peta	P
1 000 000 000 000 = 10^{12}	tera	T
1 000 000 000 = 10^{9}	giga	G
1 000 000 = 10^{6}	mega	M
1 000 = 10^{3}	kilo	k
100 = 10^{2}	hecto[b]	h
10 = 10^{1}	deka[b]	da
0.1 = 10^{-1}	deci[b]	d
0.01 = 10^{-2}	centi[b]	c
0.001 = 10^{-3}	milli	m
0.000 001 = 10^{-6}	micro	µ
0.000 000 001 = 10^{-9}	nano	n
0.000 000 000 001 = 10^{-12}	pico	p
0.000 000 000 000 001 = 10^{-15}	femto	f
0.000 000 000 000 000 001 = 10^{-18}	atto	a

[a] Refer to ASTM E380 for more complete information on SI.
[b] Use is not recommended.

TABLE D9. SI CONVERSION FACTORS[a]

Quantity	Multiply	by	to obtain	
Length	inch	25.400	millimeter	mm
	foot	0.305	meter	m
	yard	0.914	meter	m
	mile (U.S. Statute)	1.609	kilometed	km
	millimeter	39.370×10^{-3}	inch	in
	meter	3.281	foot	ft
	meter	1.094	yard	yd
	kilometer	0.621	mile	mi
Area	square inch	0.645×10^{3}	square millimeter	mm^2
	square foot	0.093	square meter	m^2
	square yard	0.836	square meter	m^2
	square mile (U.S. Statute)	2.590	square kilometer	km^2
	acre	4.047×10^{3}	square meter	m^2
	acre	0.405	hectare	
	square millimeter	1.550×10^{-3}	square inch	in^2
	square meter	10.764	square foot	ft^2
	square meter	1.196	square yard	yd^2
	square kilometer	0.386	square mile	mi^2
	square meter	0.247×10^{-3}	acre	
	hectare	2.471	acre	
Volume	cubic inch	16.387×10^{3}	cubic millimeter	mm^3
	cubic foot	28.317×10^{-3}	cubic meter	m^3
	cubic yard	0.765	cubic meter	m^3
	gallon (U.S. liquid)	3.785	liter	*l*
	quart (U.S. liquid)	0.946	liter	*l*
	cubic millimeter	61.024×10^{-6}	cubic inch	in^3
	cubic meter	35.315	cubic foot	ft^3
	cubic meter	1.308	cubic yard	yd^3
	liter	0.264	gallon (U.S. liquid)	gal
	liter	1.057	quart (U.S. liquid)	qt
Mass	ounce (avoirdupois)	28.35	gram	g
	pound (avoirdupois)	0.454	kilogram	kg
	short ton	0.907×10^{3}	kilogram	kg
	gram	35.274×10^{-3}	ounce (avoirdupois)	oz av
	kilogram	2.205	pound (avoirdupois)	lb av
	kilogram	1.102×10^{-3}	short ton	

[a] Refer to ASTM E380 for more complete information on SI.
The conversion factors tabulated herein have been rounded.

TABLE D9. SI CONVERSION FACTORS[a]

Quantity	Multiply	by	to obtain	
Force	ounce-force	0.278	newton	N
	pound-force	4.448	newton	N
	newton	3.597	ounce-force	
	newton	0.225	pound-force	lbf
Bending Moment	pound-force-inch	0.113	newton-meter	N-m
	pound-force-foot	1.356	newton-meter	N-m
	newton-meter	8.851	pound-force-inch	lbf-in
	newton-meter	0.738	pound-force-inch	lbf-ft
Pressure, Stress	pound-force per square inch	6.895	kilopascal	kPa
	foot of water (39.2 F)	2.989	kilopascal	kPa
	inch of mercury (32 F)	3.386	kilopascal	kPa
	kilopascal	0.145	pound-force per square inch	lbf/in^2
	kilopascal	0.335	foot of water (39.2 F)	
	kilopascal	0.295	inch of mercury (32 F)	
	foot-pound-force	1.356	joule	J
	[b]British thermal unit	1.055×10^3	joule	J
	[b]calorie	4.187	joule	J
	kilowatt hour	3.600×10^6	joule	J
	joule	0.738	foot-pound-force	ft-lbf
	joule	0.948×10^{-3}	[b]British thermal unit	Btu
	joule	0.239	[b]calorie	
	joule	0.278×10^{-6}	kilowatt hour	kW-h
Power	foot-pound-force/second	1.356	watt	W
	[b]British thermal unit per hour	0.293	watt	W
	horsepower (550 ft-lbf/s)	0.746	kilowatt	kW
	watt	0.738	foot-pound-force/ second	ft-lbf/s
	watt	3.412	[b]British thermal unit per hour	Btu/h
	kilowatt	1.341	horsepower (550 ft-lbf/s)	hp
Angle	degree	17.453×10^{-3}	radian	rad
	radian	57.296		degreeg
Temperature	degree Fahrenheit	$t°C = (t°F - 32)/1.8$		degree Celsius
	degree Celsius	$t°F = 1.8 \times t°C + 32$	degree Fahrenheit	

[a] Refer to ASTM E380 for more complete information on SI.
[b] International Table.
The conversion factors tabulated herein have been rounded.

GLOSSARY

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

Note: Terms designated with * are consistent with the glossary terms in the AISC *Specification for Structural Steel Buildings*.

Advance bill or Advance bill of material. 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 for Steel Buildings and Bridges*.

AISC *Specification*. The AISC *Specification for Structural Steel Buildings*.

AWS D1.1. *American Welding Society Structural Welding Code—Steel*.

Alloy steel. Steel containing silicon, manganese, nickel or another element or elements.

Anchor. Device for fastening steelwork to masonry or concrete.

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

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

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

Approval. 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 for Steel Buildings and Bridges.*

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

ASD (Allowable Strength Design).* Method of proportioning structural components such that the allowable strength equals or exceeds the required strength of the component under the action of the ASD load combinations.

ASTM. American Society for Testing and Materials.

AWS. American Welding Society.

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

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

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

Bar. 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.

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

Base plate. Load-distributing plate upon which a column bears.

Batten plate. 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.* Structural member that has the primary function of resisting bending moments.

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

Bearing stress. 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. Amount of pressure in bearing, either total or per unit of area.

Bending moment. 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. Planar framework of beams or trusses and the columns that support these members.

Bevel. Slope of a line with respect to another line.

Beveled washer. Washer beveled on one side.

Bill of finished parts. See Shipping Bill.

Blast furnace. Furnace in which iron ore is reduced to pig iron.

Block. See Cope.

Blueprint. Form of reproduction of a drawing. Blueprints are commonly made with a process that exposes sensitized paper to light shone through an original drawing.

Bolt. Cylindrical fastening with a head at one end and a threaded length at the other. The thread may extend the full length or a part length of the bolt shank, depending upon the type of bolt.

Bore. To enlarge a punched or drilled hole by means of a cutter that accurately pares the inner surfaces of the hole to fit a pin.

Box girder. Girder with two or more web plates that, with coverplates, form a closed box.

Box section. Member in which the component parts enclose a space that is accessible only at the ends.

Brace. Diagonal member placed between panel points in a frame of other members.

Braced frame.* An essentially vertical truss system that provides resistance to lateral forces and provides stability for the structural system.

Bracing system. A series of diagonals and struts placed between main members to resist wind or other lateral forces.

Bracket. Projecting type of a connection usually made of a tee, a combination of plates, or a plate and angles.

Buckle. Deflected position taken by a straight member under the influence of a compression load.

Building code. Compilation of the laws and ordinances that relate to building construction.

Built-up member.* Member fabricated from structural steel elements that are welded or bolted together.

Butt joint. Joint in which the ends of the parts connected are cut to bear against each other. The ends are held in place by splice plates or by welding.

Butt weld. Type of weld in which the edge or end of a piece is welded to the edge or end of another piece so that the pieces are in line.

Button head. Rounded head of a bolt as distinguished from one that is hexagonal, countersunk or square.

Camber.* Curvature fabricated into a beam or truss so as to compensate for deflection induced by loads.

Canted. Orientation of a beam or girder that is perpendicular to the face of a supporting member, but rotated so that its flanges are tilted with respect to those of the support.

Cantilevered beam or girder. Beam or girder that projects beyond one or both supports and is free to deflect at the end. Also, a cantilever beam or girder may have one end fixed at a support and the other end free or unsupported.

Cap angle or Cap plate. Angle or plate at the top of a column or portion of a column.

Casting. Anything formed by pouring molten iron, steel or other material into a mold and allowing it to harden.

Center of gravity. That point through which the resultant of the forces of gravity acting upon a body in any position must pass. If the body could be supported at this single point, it would remain in equilibrium in any position.

Center punch. Cylindrical piece of steel with a sharp point protruding from one end. It is inserted in the holes of templates and struck with a hammer to make indentations in the steel to indicate where holes are to be punched.

Centroid. See Center of gravity.

Change order. Document issued to make changes in the material already ordered.

Channel. Common structural steel shape the cross section of which is similar to that of an S-shape except that the flanges are on only one side of the web.

Check. To verify the accuracy of all information relating to sketches of structural members and their details, dimensions, notes and material on a drawing, advance bill or any other document originated within the detailing group.

Checker. Person in the detailing group who, by reason of experience and ability, has advanced successfully from a beginning steel detailer to a position with more responsibility.

Checkered plate. See Raised pattern floor plate.

Chip. To cut off projecting parts, as with a pneumatic chisel.

Chord. Main top or bottom member, or line of members, in a truss.

Clear span. Length of a span from face-to-face of supports.

Clearance. Space left between members, or parts of members, to allow for inaccuracies in cutting and to facilitate placing them in position.

Clevis. Forging used to connect a clevis rod to a plate or angle. The clevis is arranged to thread on the end of the rod and the plate is inserted between two flattened ends through which a pin is passed.

Clip. Small connection angle. Also, to remove the corner of a plate or angle at 45° to clear a beam fillet or a fillet weld.

Column.* Structural member that has the primary function of resisting axial force.

Column base. Base plate, slab or pedestal upon which a column stands, together with any connecting angles or plates.

Column schedule. Drawing upon which is summarized information regarding the composition and lengths of different sections of the columns in a tier building.

Component. One of two or more parts into which a force or stress may be resolved. The force or stress is the resultant of its components.

Composite beam. Steel beam structurally connected to a concrete slab so that the beam and slab respond to loads as a unit.

Compression member. Member in which the principal stresses tend to compress or shorten the member.

Compressive stress. Stress induced by axial forces on a member directed toward each other. They tend to compress or shorten the member.

Concentrated load. Load that is assumed to act at one point.

Concrete. Mixture of cement, water, sand and coarse aggregate, which hardens in forms to make structural members, slabs, walls, etc.

Connection.* Combination of structural elements and joints used to transmit forces between two or more members.

Connection angle or Connection plate. Angle or plate used to connect members.

Construction manager. Agent retained by the owner to represent the interests of the owner in the proper execution of the project.

Continuous beam or Continuous girder. Beam or girder that spans continuously over one or more intermediate supports.

Contraflexure. Change in the direction of bending in any member.

Cope.* Cutout made in a structural member to remove a flange and conform to the shape of an intersecting member.

Cost plus contract. Type of contract wherein payment is based on the actual cost of material and all labor plus a percentage of these costs.

Cotter pin. Cylindrical steel pin held in place by a split steel key or cotter placed through a hole in the pin.

Countersink. To ream a hole to receive the conical head of a bolt or screw so that the head will not project beyond the face of the part connected.

Cover plate.* Plate welded or bolted to the flange of a member to increase cross-sectional area, section modulus or moment of inertia.

Crane. Hoisting machine arranged to move heavy loads both vertically and horizontally. An overhead traveling crane usually is used in mill buildings, being supported by longitudinal girders on opposite sides of the building.

Crane-clearance diagram. Diagram made to show a crane manufacturer the principal column, truss and knee-brace dimensions so that the crane can perform.

Crane runway girder. Girder that supports one of the rails upon which a traveling crane runs.

Crane stop. Cast or built-up block attached to a crane runway girder at the end of a runway to stop a crane.

Cross bracing. Bracing with two intersecting diagonals forming the shape of an X.

Cross-section. Transverse section. Also, a view representing the appearance of a structure or member where cut by an imaginary section plane.

Crosshatch. To draw fine sloping lines signifying a cross-section. Optional in the current practice of structural steel detailing, generally used only when necessary to improve the clarity of presentation.

Cut. Term used by some fabricators to describe the removal of a portion of a steel piece.

Dead load. Comparatively constant static load on a structure due to its weight, as distinguished from the live or moving load, wind load or seismic load.

Deflection. Movement at right angles to either principal axis of a member. Also, the linear measurement of 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. Hoisting machine so pivoted that a load may be swung horizontally. Two types are a guyed derrick and a stiff-leg derrick.

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

Design drawing. 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 factor multiplied by the nominal strength, ϕR_n.

Designer. 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. 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 member carrying primarily axial force in a braced frame.

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

Diaphragm.* Roof, floor or other membrane or bracing system that transfers in-plane forces to the lateral force resisting system.

Die. Steel form used in forging or cutting any piece.

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

Direct tension indicators. 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 Structural Joints Using ASTM A325 and A490 Bolts* for further information.

Double shear. 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. 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.

Draw. Tension induced in diagonals of bracing systems to ensure initial tightness.

Drift. Lateral deflection of structure.

Drift pin. 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. 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. Longitudinal member between the tops of columns at the eaves of a building.

Eccentric connection. 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. Perpendicular distance from a resultant force to some other point or line.

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

Effective weld length. 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. Property of steel in which steel will return to its original shape following deformation by an externally applied force not exceeding the yield strength.

Elevation. Vertical distance from a reference surface or datum. Also, a drawing or view that represents the projection of a structure or member upon a vertical plane.

Embedment. Steel component cast into concrete and used to transmit externally applied loads to the concrete by means of bearing, shear, bond, friction or any combination thereof.

Entering and tightening clearance. Minimum distance from the center of a bolt to the nearest projecting part that might interfere with the use of the wrench used to tighten the bolt.

Equilibrium. Loads that act upon a body are said to be in equilibrium when they so balance each other that the body has no tendency to move.

Erasing shield. Thin sheet of metal containing openings of different shapes through which parts of a drawing may be erased without disturbing the adjacent parts.

Erection. Assembling and connecting of the different members of a structure in their proper positions at the job site.

Erection bolts. Bolts used during erection to hold members in position until field welds are placed or permanent field bolts are installed.

Erection clearance. Clearance between members provided to facilitate erection.

Erection drawing. Drawing consisting of line diagrams representing framing in plan, elevation, section, etc., to which are added principal dimensions, erection marks, notes and, when required, enlarged details, thus providing the erector with sufficient information to place the members in the structure. These drawings do not normally show the erection scheme, schedule, rigging devices, temporary supports, safety devices, etc.

Erection mark. Identifying mark that aids the erector in properly placing a member.

Erection seat. Seat angle bolted or welded to a supporting member to support a beam, girder or similar member temporarily during erection.

Erector. Party responsible for the erection of the structural steel. Also, a term used to identify any or all of the workers performing erection activities.

Estimate. To compile the quantities, weights and costs of a structure as a basis for bidding.

Expansion bolt. Bolt for attaching steelwork to a masonry/concrete wall or concrete slab. The bolt is surrounded by a split sleeve that expands as the bolt is tightened in a hole drilled into the masonry or concrete.

Extension dimensions. Cumulative dimensions from a given point, such as the left end of a beam or the finished bottom of a column shaft, used to locate several connections with the same position of a measuring tape.

Extras. Costs incurred for performing work beyond the scope of work originally contracted to perform.

Fabrication. Shop work required to convert raw material into complete structural members or the work done in a structural shop.

Fabricator. Party responsible for furnishing fabricated structural steel.

Face. To plane or smooth a surface. Also, the exterior plane surface of any solid.

Factored load.* Product of a load factor and the nominal load.

Falsework. Temporary system of steel and/or timber columns, beams and bracing for supporting a structure during erection or demolition.

Fastener.* Generic term for bolts, rivets, or other connecting devices.

Fatigue.* Limit state of crack initiation and growth resulting from repeated application of live loads.

Faying surface.* Contact surface of connection elements transmitting a shear force.

Field. Term applied to the work done on parts of a structure at or near the job site as opposed to work done in the shop. Also, used interchangeably with the terms Job site or Site.

Field check. Partial checking of the drawings of a structure to ensure the proper connection of the members in the field.

Field connection. Connection of different members made by the erector.

Field bolt or field weld. Bolt installed or weld placed in the field as distinguished from a shop bolt or weld.

Field bolt summary. List of bolts required to make the necessary field connections in a structure.

Field work drawing. Drawing showing sketches and details of connections to existing structures. Also, it may be used as a preliminary drawing to list field measurements, which will determine the length and connection details of new members connecting to an existing structure.

Filler.* Plates used to build up the thickness of one component.

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

Fillet weld.* Weld of generally triangular cross-section made between intersecting surfaces of elements.

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

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

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

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

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

Fixture. 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. Gas flame or torch used for cutting steel by melting a narrow slot by means of an intense heat.

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

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

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

Flat. Narrow plate with rolled edge.

Flexible connection. 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. Beam participating in the support of a floor.

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

Floor plate. See Raised pattern floor plate.

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

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

Footing. Concrete pier or foundation for a column.

Force.* Resultant of distribution of stress over a prescribed area.

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

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

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

Gage.* Transverse center-to-center spacing of fasteners.

Gage lines. 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 assembly during installation.

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

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

General contractor. 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.* See Beam.

Girt.* Horizontal structural member that supports wall panels and is primarily subjected to bending under horizontal loads, such as wind load.

Government anchor. Short rod with a V-shaped bend in the center, used to anchor the end of a wall-bearing beam.

Grillage. Tier of closely spaced beams connected to each other to support a heavily loaded column. When more than one tier is required to support a column, additional tiers are laid across and connected to the tiers below.

Grip. Combined thickness of steel connected by a bolt or a pin.

Groove weld. Weld made in a groove between adjacent ends, edges or surfaces of two parts to be joined. Grooves usually are prepared by flame cutting, arc-air gouging or edge planing.

Gross area. Full area of cross-section, in contrast to net area.

Group spacing. Number of equal spaces dimensioned together as, for example, 4 at 6” = 2’-0.

Grout. Fluid mixture of cement, water and sand, which can be poured to establish a bearing surface under a column base plate, to fill small voids in concrete or to smooth or to level a surface of a wall or footing.

Gusset plate.* Plate element connecting truss members or a strut or brace to a beam or column.

Hand hole. Hole made in a member for the insertion of a hand to install bolts that would be inaccessible otherwise.

Hanger. Steel tension member used to support a load.

Heel plate. Gusset plate at the heel, or main support, of a roof truss.

High-strength bolts. ASTM A325 carbon steel bolts and ASTM A490 quenched and tempered alloy steel bolts.

Hip. Junction of the top chord of a truss with the inclined end post. Also, the intersection of two roofs where the drainage is away from the intersection, as distinguished from a valley.

Hollow structural sections (HSS).* Square, rectangular or round hollow structural steel section produced in accordance with a pipe or tubing product specification.

Hook. Curved or bent end of an anchor rod.

Hook bolt. Rod with a hook at one end and threaded at the other, used to attach light rails to crane runway beams.

Impact. Increased effect of live loads that are applied suddenly.

Inflection point. See Contraflexure.

Information sheet. Sheet which will accompany the design drawings containing all information for basis of the contract.

Item number. Serial number assigned to each different item that is ordered from the rolling mills and steel distribution center.

Itemize. To add the item numbers and other mill information to a shop bill.

Jig. See Fixture.

Job Site. Location where a structure is to be erected.

Joint.* Area where two or more ends, surfaces or edges are attached. Categorized by type of fastener or weld used and method of force transfer.

K bracing. System of diagonals used in a braced frame in which the pattern of the diagonals resembles the letter K, either normal or on its side.

Kerf. Width of material removed during the process of cutting a steel member by torch or saw.

Kip. Term relating to force. One kip equals 1,000 pounds.

Knock down. Term referring to the shipping in smaller, manageable sections of structural steel for members that are too large or heavy to ship in one piece.

Lag screw. Large screw for wood with a square head to be turned with a wrench.

Lamellar tearing. Separation in highly restrained base metal caused by through-thickness strains induced by shrinkage of adjacent weld metal.

Laminated. In layers, as a steel member built of several thinner pieces bolted together.

Lateral. Sidewise or at right angles to the weak principal axis.

Lateral bracing member. Member utilized individually or as a component of a lateral bracing system to prevent buckling of members or elements and/or to resist lateral loads.

Laying out. Marking of steel from templates or otherwise, indicating where holes are to be punched or drilled and where cuts are to be made.

Layout. Preliminary drawing or sketch by means of which distances may be determined by scaling.

Left. Member is so marked when made exactly opposite to a corresponding member marked "right," the latter being represented on the drawing. Also, means left of sketch when applied to a shipping mark.

Leg. One of the two steel flanges or parts of an angle. Also, the distance from the root to the toe of a fillet weld.

Limit state(s).* Condition in which a structure or component becomes unfit for service and is judged to be no longer useful for its intended function (serviceability limit state) or to have reached its ultimate load-carrying capacity (strength limit state).

Linear. Pertaining to line or to length. A linear dimension usually is one measured parallel to the length of the member.

Live loads. Loads from occupants and contents of the building, elevators, certain types of machinery or equipment stored in the structure.

Load.* Force or other action that results from the weight of building materials, occupants and their possessions, environmental effects, differential movement, or restrained dimensional changes.

Load factor.* Factor that accounts for deviations of the nominal load from the actual load, for uncertainties in the analysis that transforms the load into a load effect, and for the probability that more than one extreme load will occur simultaneously.

LRFD (Load and Resistance Factor Design).* Method of proportioning structural components such that the design strength equals or exceeds the required strength of the component under the action of the LRFD load combinations.

Lug. Small projecting connection, as a connection angle or plate.

Lump sum. Type of contract wherein payment is based upon satisfactory completion of a specific amount of work explicitly shown on the contract documents.

Masonry plate. Bearing plate placed on masonry.

Matchmark. System whereby structural steel members too large or heavy to ship are fabricated in smaller sections that are assembled in the shop for fitting or reaming connections. After each field splice is given an individual identifying mark to enable the erector to reassemble the sections in the same manner as they were in the shop, the pieces are disassembled and shipped to the site.

Material list or Material order bill. See Advance bill.

Member. Part of a structure that is assembled completely in the shop and shipped to the site where it is combined with other members.

Mill. Plant in which plates and shapes are produced. Also, to plane the end of a member by means of a rotary planer or milling machine.

Mill building. Steel-framed building with a roof of comparatively large pitch and span, but usually without partitions, intermediate floors or interior bracing, except knee braces.

Mill test reports. Documents furnished by steel producers to show the results of physical and chemical tests for steel produced.

Mill tolerance. Term used to describe permissible deviations from the published dimensions of shapes and plates listed in mill catalogs and the AISC *Steel Construction Manual*.

Mill variation. Rolling and cutting tolerances in size or length as practiced in the rolling mills.

Milled surface.* Surface that has been machined flat by a mechanically guided tool to a flat, smooth condition.

Milling machine. Machine for milling or planing the end of a member.

Mitered joint. Joint in which the angle between the connected parts is bisected by the plane of contact.

Modulus of elasticity. Ratio of stress to strain in the elastic range, designated by the letter *E*.

Moment. Term referring to the bending or rotating effect of an eccentric force upon a member or joint. The rotation may be clockwise or counterclockwise. Moment is the product of a force expressed in units of weight (kips or pounds) times a distance expressed in units of length (feet or inches), such as kip-feet, pound-inches, etc.

Moment diagram. Graphical depiction of the magnitude of the bending effects at any point along the length of a beam, girder or column.

Moment of inertia. Value used in determining the resistance of material to flexure and torsion.

Monitor. Raised portion of a roof of a mill building or similar structure, arranged to provide additional ventilation or light through the louvers or windows in the sides.

Multiple punch. Machine arranged to punch two or more holes simultaneously.

Nailing strip. 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).* Gross area reduced to account for removed material.

Nominal load.* Magnitude of the load specified by the applicable building code.

Nominal strength.* Strength of a structure or component (without the resistance factor or safety factor applied) to resist load effects, as determined in accordance with the AISC *Specification*.

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

Orthographic projection. 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. Small angle or similar piece fastened to an end purlin of a building to support the roof, which overhangs the gable end.

Overrun. 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-sectional dimensions from those published in ASTM A6, A500 or A53 (as summarized in the AISC *Steel Construction Manual*).

Owner's designated representative for construction. Owner or the entity that 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. Owner or the entity that 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.

Panel point. Intersection of the working lines of different members of a truss.

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

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

Piece mark. See Assembly mark.

Pier. Concrete column footing.

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

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

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

Pitch. 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. Drawing that represents the horizontal projection of a structure or part of a structure. Often less accurately used to refer to 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.

Plate girder.* Built-up beam.

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. Comparatively short vertical, or near vertical, compression member. Also, a compression member less than 300 lb in weight per OSHA.

Pound price. Method of contract payment wherein payment is based upon the calculated weight of structural steel in pounds (or tons) multiplied by the contractual price per pound (or ton). This method of payment often is used when a design is incomplete or when additions and changes are expected.

Preliminary bill. See Advance bill.

Prequalified joint. American Welding Society joint that is welded and prepared conforming to all AWS D1.1 Code and specification provisions for design, material and workmanship.

Projection line. Line drawn at right angles to a dimension line to indicate the extent of the dimension.

Prying action.* Amplification of the tension force in a bolt caused by leverage between the point of applied load, the bolt and the reaction of the connected elements.

Punch. To make a full-size hole with a single stroke, as distinguished from drilling or boring. Also, a punching machine.

Purchaser. Person employed by the fabricator to buy the raw material.

Purlin.* Horizontal structural member that supports roof deck and is primarily subjected to bending under vertical loads such as snow, wind or dead loads.

Qualified joints. Welded joints (apart from prequalified joints) used by a fabricator. The joints meet the requirements of AWS D1.1.

Radius of gyration. Value, derived from the shape of a member, used in determining the resistance of the member or part of the member to buckling under compression or flexure.

Rafter. Inclined member parallel to the roof slope used to support the purlins in place of a truss.

Rail clamp. Device for fastening a crane rail to the flange of a crane runway beam or girder.

Raised pattern floor plate. Steel plate with raised ribs or a raised pattern used for a walking surface.

Raw material. Structural steel obtained by a fabricator from steel producers and steel distribution centers for fabrication.

Reaction. Load on a beam, girder or truss imparted by the support to balance the loads.

Ream. Enlarge a hole by means of a rotating fluted cutter.

Reinforced concrete. Combination of concrete and steel reinforcing bars.

Reinforcing plate. Plate used to reinforce a member.

Required strength.* Forces, stresses and deformations acting on the structural component, determined by either structural analysis, for the LRFD or ASD load combinations, as appropriate, or as specified by the AISC *Specification* or Standard.

Residual stress. Stresses that remain in an unloaded member after it has been formed into a finished product. Examples of such stresses include, but are not limited to, those induced by cold bending, cooling after rolling or welding.

Resistance. The capacity of a structure or component to resist the effects of loads. It is determined by computations using specified material strengths, dimensions and formulas derived from accepted principles of structural mechanics, or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions. Resistance is a generic term that includes both strength and serviceability limit states.

Resistance factor, ϕ.* Factor that accounts for unavoidable deviations of the nominal strength from the actual strength and for the manner and consequences of failure.

Resisting moment. Moment of the internal forces or stresses which resist the bending moment of a beam, girder or column.

Restrained beam. Structurally speaking, a beam that is restrained or "fixed" at a support. Also, in fire resistance applications, a beam that has thrust restraint against thermal expansion.

Resultant or Resultant force. Simplest single force that can replace a system of forces and have an equivalent effect.

Reversal of stress. Change of stress from compression to tension, or vice versa.

Ridge strut. Longitudinal structural member along the ridge or peak of a roof.

Right. Member is so marked when another member marked "left" is to be made exactly opposite from the same drawing.

Right section. Section at right angles to the principal axis.

Rigid frame. Structure in which connections maintain the angular relationship between beam/girder and column members under load.

Rivet. Short cylindrical rod of steel with upset heads used to fasten together component parts of a steel structure. One head is formed before the rivet is placed in position, the other head later by a riveting machine. Currently, replaced by the use of high-strength bolts.

Rod. Rolled bar of steel with round or square cross section.

Rolling mill. See Mill.

Rotary planer. Machine for planing or milling the end of a member for uniform bearing.

Round. Round rod.

S beam. Rolled structural steel shape with a cross-section that resembles the letter I and is characterized by narrow, tapered flanges and relatively thick webs.

Sag rod. Vertical or inclined tie rod used to reduce the sagging of a girt or purlin.

Sawtooth roof. Roof arrangement providing for sky light without direct sunlight. The vertical or steeper slopes are glazed while the smaller slopes toward the sun are covered with roofing material.

Scale. Flat or triangular measuring stick used in plotting a drawing in proportion to the thing represented. Also, indicates proportion.

Seat angle. Angle bolted or welded to one member such that its outstanding leg is, or nearly is, horizontal to support the end of another member.

Section. Cut across a member or structure made by an imaginary section plane.

Section lines. Fine sloping lines used to shade an area cut by a section plane. In current practice, these lines are omitted in sectional views on structural steel shop drawings.

Sectional view. Projection of one segment of a member or structure upon a section plane.

Separator. Plate or piece of pipe placed between the webs of beams to keep them a fixed distance apart.

SER. Structural Engineer of Record.

Serviceability limit state.* Limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability, or the comfort of its occupants or function of machinery, under normal usage.

Setback. In detailing beams and girders, the distance from the extreme end of the member to a major reference line, such as the centerline of the supporting beam, girder or column.

Shank. Cylindrical part of a bolt, as distinguished from the head.

Shape. General term for structural rolled steel of any cross-section other than a plate.

Shear. To cut by shearing. The machine used for shearing steel plates or angles. Also, in material an expression for the algebraic sum of all the external forces acting on one side of a section plane through the material.

Shear diagram. Graphical depiction of the magnitude of vertical shear force at any point along a beam or girder.

Shear lag. Occurrence where less than the total cross-sectional area of a member is connected, resulting in a decrease in efficiency of the section in the region of the connection.

Shear lugs. Plates and other steel shapes that are embedded in the concrete and located transverse to the direction of the shear force. They transmit shear loads introduced into the concrete by local bearing at the shear lug-concrete interface.

Shear wall. Wall that, in its own plane, resists shear forces resulting from applied wind, earthquake or other transverse loads or provides frame stability.

Sheared plate. Plate that is passed between horizontal rolls at the mill and then trimmed (sheared or gas cut) to the desired size on all edges, as distinguished from a Universal mill plate.

Shearing stress. Stress caused by vertical loads applied to a beam or girder and tending to slide one section along an adjacent section.

Shim.* Thin layer of material used to fill a space between faying or bearing surfaces.

Shipping bill. List of members to be shipped or transported from the shop to the site.

Shipping mark. See Erection mark.

Shipping memorandum. See Shipping bill.

Shipping weld. See Tack weld.

Shipping yard. Area at a fabricating shop in which fabricated members are sorted and stored for shipment to the job site as required.

Shop. Place where the component parts of a structure are fabricated into members.

Shop bill. Summary of material required for fabricating the members shown on a particular drawing.

Shop bolt. Bolt that is installed in the shop, as distinguished from a field bolt.

Shop drawing. Working drawing prepared for use in the shop.

Shop paint. Coat of paint applied, if required, at the fabricating shop.

Sidesway. Lateral movement of a structure under the action of lateral loads, unsymmetrical vertical loads or unsymmetrical properties of the structure.

Simple beam. Unrestrained beam that is supported at both ends only.

Simply supported. Loaded beam or girder in which the ends are free to rotate at the points of support.

Single punch. To punch one hole at a time.

Single shear. Tendency to shear, or the resistance to shear, a single element or group of elements on one plane.

Site. See Job Site.

Skeleton. Term used to describe a building frame. The frame may be of steel, concrete, wood or any combination of these materials.

Sketch. Illustration of a structural steel member, connection or detail made free-hand. Also, the individual illustrations of members and details presented on a shop drawing.

Skewed. A beam or girder is described as skewed when its flanges are parallel to the flanges of the supporting beam/girder, but the webs are inclined to each other.

Skids. Parallel supports of timber or steel used to elevate members a convenient distance above the floor of a shop to make them more accessible.

Skin. Term used to describe the exterior wall (covering) of a building. The skin may be brick, concrete, glass, metal panels, stone or a combination of any of these materials.

Slab. Thick steel plate used as a column base or similarly. Also, a reinforced-concrete floor.

Sleeve nut. Long tubular nut having a right-handed thread in one half and a left-handed thread in the other, used for joining two threaded rods and pulling them together to tighten them.

Slenderness ratio. Ratio of the effective length of a column to the radius of gyration of the column, both with respect to the same axis of bending.

Slip-critical connection.* Bolted connection designed to resist movement by friction on the faying surface of the connection under the clamping forces of the bolts.

Slope. Bevel or inclination of one line with reference to another. It is measured by the tangent of the angle of inclination expressed in inches and fractions to a base of one foot.

Sloped. Beam or girder is described as sloped if its web is perpendicular to the web of the supporting member, but its flanges are not perpendicular to this face.

Slot weld. Weld made in and filling a slot in one of the parts connected.

Slotted hole. Elongated hole with semicircular ends and parallel sides. May be either short slots or long slots as defined by the AISC *Steel Construction Manual*.

Snipe. See Clip.

Snug-tightened joint.* Joint with the connected plies in firm contact as specified in Chapter J in the AISC *Specification*.

Span. Distance between the supports of a beam, girder, truss, etc.

SSPC. Society for Protective Coatings or the Steel Structures Painting Council.

Spiking piece. Wooden strip bolted to a steel beam or similar member to which strip planking or sheathing may be nailed or spiked.

Spiral. Small round or square rod bent in the form of a helix, used in reinforced-concrete columns to prevent the longitudinal reinforcing bars from spreading.

Splice. Connection between two structural elements joined at their ends to form a single, longer element.

Split beam. Beam that is split to form a structural tee or bracket. Also, a beam split part way with one flange bent out to permit welding a plate in the web for strengthening the joint for rigid-frame construction.

Spotting. Placement of a spot of paint on the faying surface of a single angle, single plate or tee. It is one of several methods used to alert the erector as to which side to connect a supported member.

Staggered bolts. Bolts which alternate on two parallel bolt lines.

Statical moment. Product of an area times the distance from an axis to the center of gravity of the area.

Steel. Modified form of iron used in construction.

Steel detailer. Individual who uses contract documents to make detailed working drawings for each piece of steel to be furnished by a fabricator and erected by an erector.

Steel mill. See Mill.

Stiffener.* Structural element, usually an angle or plate, attached to a member to distribute load, transfer shear or prevent buckling.

Stitch bolts. Bolts used to hold the angles of a multi-shape member together at intervals. If the elements are separated by connection (gusset) plates, a stitch filler is placed between them at each stitch bolt to maintain a constant space.

Strain. Deformation in a member caused by external forces. Strain is measured in linear units.

Strength limit state.* Limiting condition affecting the safety of the structure, in which the ultimate load-carrying capacity is reached.

Stress.* Force per unit area caused by axial force, moment, shear or torsion.

Stress concentration.* Localized stress considerably higher than average (even in uniformly loaded cross sections of uniform thickness) due to abrupt changes in geometry or localized loading.

Strip shim. Shim consisting of a narrow piece of structural steel punched with round holes.

Striping. Placement of a stripe of paint on the faying surface of a single-angle, single-plate or tee. It is one of several methods used to alert the erector as to which side to connect a supported member.

Strong axis.* Major principal centroidal axis of a cross section.

Structural design documents. Documents prepared by the designer (plans, design details and job specifications). These documents define the responsibilities of the parties involved in bidding, purchasing, supplying and erecting structural steel.

Structural drafting. The preparation of the working drawings for the members in a structure.

Structural shop. Shop where rolled steel shapes and plates are punched, cut, bolted, welded and otherwise prepared for erection in a steel structure.

Structural tee. T-shaped member made by cutting a W-, M- or S-shape in two along the length of its web.

Strut. Comparatively light compression member, usually with no intermediate connections.

Subpunch. To punch a hole to a smaller diameter than required, that will be reamed after parts are assembled.

Substructure. Concrete piers or foundation for a structure.

Superstructure. Main part of the structure above the concrete foundation or substructure.

Sway bracing. Bracing in a vertical plane as between the roof trusses of a mill building.

Sweep. Comparatively flat horizontal curve in a beam or girder induced through cold bending or by the application of heat.

Swedge bolt. Type of anchor rod threaded at one end for a nut and having depressions in the remaining length of shank to furnish bond when embedded in masonry or concrete.

Tack weld. Small, temporary weld made to hold component parts of a member together until they can be welded permanently.

Tee. See Structural tee.

Template. Full-size pattern or guide (made of wood, cardboard or steel) used to locate punched or drilled holes and cuts or bends to be made in the steel.

Tension. Condition of a material loaded in such a way that the load tends to stretch the material in the direction of its length.

Tension-control (twist-off) bolt. Alternative design type of fastener with a splined end extending beyond the threaded portion of the bolt. The bolt is installed using a special wrench to remove the splined end when the proper pretension is induced in the bolt.

Tension member. Member in which the principal stresses tend to lengthen the member.

Tensile strength.* Maximum tension force that a member is capable of sustaining.

Throat. In a fillet weld, the distance from the root to the face of the weld.

Tie. Light tension member, such as the diagonal in a bracing system.

Tie plate.* 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.

Tier. Term relating to a length of column supporting one, two or three floors of steel.

Toe of fillet.* Junction of a fillet weld face and base metal. Tangent point of a rolled section fillet.

Top angle. Connection angle used at the top of a beam in conjunction with a seat angle on a column or in a girder web.

Torsion. Action of applied load(s) that tend to twist a member.

Tracing paper. Paper specially treated to make it transparent so that copies can be made of drawings on it.

Truck crane. Crane mounted on a mobile truck.

Truss. Framed structure that acts like a beam. The principal members usually form a series of triangles and each member primarily is subjected to axial stress/loads only.

Turnbuckle. Similar to a sleeve nut except that a transverse opening is provided at the center for the insertion of a crowbar for turning the turnbuckle.

Turn-of-nut method.* Procedure whereby the specified pretension in high-strength bolts is controlled by rotation of the nut a predetermined amount after the snug-tightened condition has been achieved.

U-bolt. Rod bent in the shape of the letter U with each end threaded for nuts.

Undercut. Groove melted into the base metal adjacent to the weld toe or weld root and left unfilled by weld metal.

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

Uniform load. Load that is distributed uniformly over a portion or full length of the beam or girder.

Unit price. See Pound-price.

Universal mill (UM) plate. Plate rolled between horizontal and vertical rolls and trimmed (sheared or gas cut) on ends only, as distinguished from a sheared plate.

Upset. To enlarge the end of a rod by hammering or pressing into a die while hot.

Valley. Intersection of two roofs where drainage is toward the intersection as distinguished from a hip.

Vertical bracing system. System of shear walls, braced frames or both, extending through one or more floors of a building.

View. In orthographic projection, a view is the projection of an object upon a plane by means of parallel lines.

Wall anchor. Rod or a pair of angles used to anchor a wall bearing beam to a masonry wall.

Wall plate. Bearing plate on a wall used to distribute the load from a steel beam or girder. Also, a plate along the top of a beam to furnish bearing for a superimposed wall.

Washer. Usually a flat disk with a central hole placed under the head or the nut of a bolt. Also, may be a square bar with a central or eccentric hole.

Weak axis.* Minor principal centroidal axis of a cross section.

Web. Web plate of a built-up girder or column. Also, the corresponding thin portion between the flanges of a rolled W-, M-, S-, HP-, C- or MC-shape.

Web buckling. Buckling of a web or web plate.

Web connection angles. Angles at the end of a beam or girder used to connect to another member.

Web crippling. Local failure of a web plate in the immediate vicinity of a concentrated load or reaction.

Web member. Intermediate member of a truss between the chords.

Web splice. Splice in the web of a rolled shape or in a web plate.

Weld. To connect metal parts by fusion of the parts, with or without additional metal, with the necessary heat being supplied by electric arc or otherwise.

Weld face. Exposed surface of a weld on the side from which welding was done.

Weld pass. Single progression of welding along a joint, the result of which is a weld bead or layer.

Weld root. Point at which the root surface intersects the base metal surfaces. In a fillet weld the root is the point at which the legs intersect.

Weld spatter. Metal particles expelled during fusion welding that do not form a part of the weld. These particles stick to the surrounding metal.

Weld throat. Shortest distance between the root and face of a weld.

Welding clearance. Necessary working space around a welded joint so that the electrode can be used to the best advantage.

Wide-flange beam. Common, rolled structural steel shape used principally as a beam or column and referred to as a W-shape.

Wind bracing. System of bracing which resists loads induced by the wind.

Wind bracket. Bracket used to stiffen a joint to resist loads caused by the wind.

Working drawing. Detailed drawing prepared for the workers who fabricate the members or parts represented.

Working line. Reference line to which the dimensions of a member are referred. A working line of one member can be used in conjunction with the working lines of other members to form a system of working lines for a truss or a bracing system. Also, working lines are used at the site as a reference to locate, for example, foundation walls, piers, shear walls, embedments, edges of floor slabs, and equipment.

Working load. Also called service load. The actual load assumed to be acting on the structure.

Working point. Intersection of two or more working lines.

Wrapped connection. Connection in which the end of the web of a beam or girder is connected to the inside face of a single angle.

Yield point.* First stress in a material at which an increase in strain occurs without an increase in stress as defined by ASTM.

Yield strength.* Stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain as defined by ASTM.

Yield stress.* Generic term to denote either yield point or yield strength, as appropriate for the material.

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