THIRD EDITION

Handbook of Structural Steel Connection Design and Details

McGraw Hill Education

Akbar R. Tamboli

HANDBOOK OF STRUCTURAL STEEL CONNECTION DESIGN AND DETAILS

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Akbar R. Tamboli, P.E., F.ASCE, is a consultant at Thornton Tomasetti in New York. He has been a senior project engineer with CUH2A in Princeton, New Jersey, a company that specialized in architecture, engineering, and planning. He has also been a vice president and project manager at Cantor-Seinuk Group PC, Consulting Engineers, in New York, where he was the principal consulting engineer on a number of noteworthy projects, including Morgan Guaranty Bank Headquarters at 60 Wall Street and Salomon Brothers World Headquarters, Seven World Trade Center.

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PREFACE TO THE THIRD EDITION

Since the previous edition of this book was published in October 2009, there have been many developments in the various aspects of steel connection design. Improved fabrication and construction techniques have led to efficient structural connections.

The new AISC code provisions for 2016 have been incorporated in this new edition. AISC provisions have been referenced in and made part of the International Building Code.

Chapters 1 and 2 have been reworked to reflect the 2016 AISC code provisions. Chapter 3 on welding has been completely rewritten to incorporate new welding codes and the 2016 AISC code provisions. New information has been added on state-of-the-art welding procedures and special precautions needed for welded joints in seismically active regions.

Partially restrained connections, covered in Chap. 4, have been evolving and have been made part of the AISC code. This chapter has been rewritten with several examples.

Seismic connection and structural design, addressed in Chap. 5, have been improving. This chapter has been revised to reflect the improvements with actual examples.

Chapter 6, on structural details, can be found at www.mhprofessional.com/tamboli.

New construction and fabrication methods used for recent special structures are described in Chap. 7. Chapter 8, on quality control and inspection, has been completely rewritten. In many cases, the projects featured in this chapter are international; therefore, both metric and English unit tolerances are given.

Chapter 9 on steel decks has been completely updated to meet Steel Deck Institute (SDI) requirements.

Chapter 10 on composite construction connections can be found at www.mhprofessional.com/tamboli.

The editor wishes to thank the contributors for their efforts in preparing excellent manuscripts.

The editor and the contributors are grateful to several sources for providing the information presented. Space considerations preclude listing all of them, but credit is given wherever feasible, especially in references throughout the book.

Users of this handbook are welcome to communicate with the editor regarding any aspect of the book, particularly suggestions for improvement.

Akbar R. Tamboli

PREFACE TO THE FIRST EDITION

The need for the *Handbook of Structural Steel Connection Design and Details* with an LRFD approach was recognized at the time the *Steel Design Handbook: LRFD Method* was published.

This handbook was developed to serve as a comprehensive reference source for the design of steel connections using the LRFD method. Each topic is written by leading experts in the field. Emphasis is given to provide examples from actual practice. Examples are focused to give a cost-effective approach. The theory and criteria are explained and cross-references to equations to AISC are given where applicable.

The book starts with a discussion of fasteners for structural connections. It then goes into the design of connections for axial, moment, and shear forces. Detailed connection design aspects are covered in this chapter.

Welded joint design and production are treated as a separate topic, and state-of-the-art information on welding is given for use in daily practice. How to control weld cracking and joint distortion is explained for use in general consulting practice. Partially restrained connection design is explained with practical examples.

Recent seismic activity has created the need for the design of connections for seismically resistant structures. These types of connections are covered with detailed examples. Commonly used connection details are shown for use in daily practice by fabricator, detailer, and consulting engineer.

Sometimes fabricators and engineers need to design connections for special structures. Actual examples of how to approach these needs are given from real projects which are built.

To ensure quality of connection, construction inspection and quality control are vital. Therefore, detailed information on these aspects is given to achieve desired goals.

Most steel structures have steel decking. To ensure good quality and interaction, steel deck details are explained thoroughly.

The latest trend in composite construction has created the need for the design of composite construction connections. Steel-to-concrete shear wall and composite column connections are explained in detail to achieve proper interaction and strength.

The editor gratefully acknowledges the efforts of contributors in preparing excellent manuscripts. Thanks are due to the management and staff at CUH2A, Inc.

The editor and authors are indebted to several sources for the information presented. Space considerations preclude listing all, but credit is given wherever feasible, especially in references throughout the book.

Users of this handbook are urged to communicate with the editor regarding all aspects of this book, particularly any error or suggestion for improvement.

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- Lynn Beedle, Lehigh University

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The editor would also like to acknowledge the help and assistance provided by Lauren Poplawski, sponsoring editor of this handbook, who put forth invaluable support during the process of preparing the manuscript. Also, thanks go to the many other individuals at McGraw-Hill and at Cenveo who were responsible for bringing the book to press, including Lynn Messina and Stephen Smith, and Srishti Malasi, project manager.

The editor wishes to extend his thanks and appreciation to his wife, Rounkbi, and his children, Tahira, Ajim, and Alamgir, for their patience and understanding during the preparation of this handbook.

CHAPTER 1

FASTENERS AND WELDS FOR STRUCTURAL CONNECTIONS

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(*Courtesy of The Steel Institute of New York.*)

1.1 INTRODUCTION

There are two common ways to connect structural steel members—using bolts or welds. Rivets, while still available, are not currently used for new structures and will not be considered here. This chapter will present the basic properties and requirements for bolts and welds.

Connections are an intimate part of a steel structure and their proper treatment is essential for a safe and economic structure. An intuitive knowledge of how a system will transmit loads (the art of load paths), and an understanding of structural mechanics (the science of equilibrium and limit states), are necessary to achieve connections which are both safe and economic. Chapter 2 will develop this material. This chapter is based on the bolting and welding requirement specifications of the American Institute of Steel Construction (AISC), "Specification for Structural Steel Buildings," 2016, and the American Welding Society Structural Welding Code, D1.1 (2010).

1.2 BOLTED CONNECTIONS

1.2.1 Types of Bolts

There are two kinds of bolts used in steel construction. These are high-strength structural bolts (Fig. 1.1) and common bolts manufactured under ASTM A307 (Fig. 1.2). High-strength bolts are included in three separate American Society for Testing and Materials (ASTM) Specifications: F3125, F3043, and F3111. F3125 is an umbrella specification that includes four grades: A325, A490, F1852, and F2280. The AISC Specification divides high-strength bolts into three groups based on minimum tensile strength. Group A bolts have a minimum tensile strength of 120 ksi and include ASTM F3125 Grades A325, A325M, and F1852, as well as ASTM A354 Grade BC. Group B bolts have a minimum tensile strength of 150 ksi and include ASTM F3125 Grades A490, A490M, and F2280, as well as A354 Grade BD. Group C bolts have a minimum tensile strength of 200 ksi and include ASTM F3043 and ASTM A3111. The various grades of F3125 are intended for general structural use, with the use of A354 and A449 fasteners intended only for conditions where the length or diameter limits of F3125 must be exceeded. F3034 and F3111 are probably best suited to heavily loaded connections. A449 bolts are also permitted to be used where the length and diameter limitations for A325 are exceeded. They are not included in Group A due to the multiple decreases in tensile strength based on diameter. A307 bolts, which were referred to previously as *common bolts*, are also variously called *machine bolts*, *ordinary bolts*, and *unfinished bolts*. The use of these bolts is limited primarily to shear connections in nonfatigue applications.

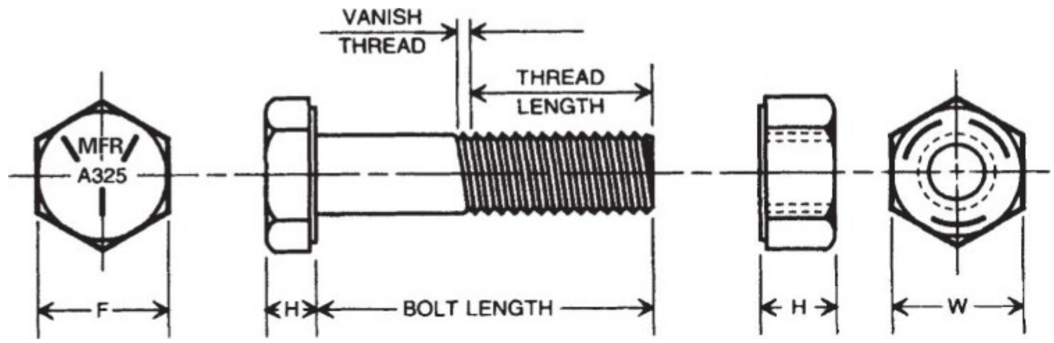


FIGURE 1.1 High-strength structural-steel bolt and nut.

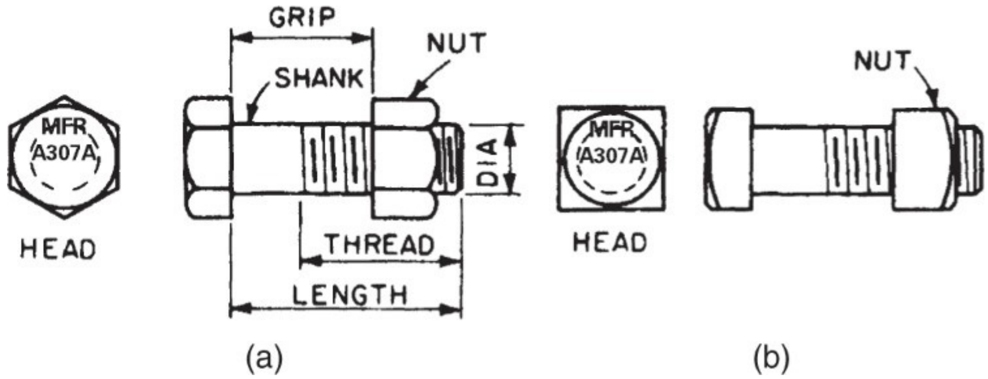


FIGURE 1.2 Unfinished (machine) or common bolts.

Structural bolts can be installed pretensioned or snug tight. Pretensioned means that the bolt is tightened until a tension force approximately equal to 70 percent of its minimum tensile strength is produced in the bolt. Snug tight is the condition that exists when all plies are in contact. It can be attained by a few impacts of an impact wrench or the full effort of a man using an ordinary spud wrench. Common bolts (A307) can be installed only to the snug-tight condition. There is no recognized procedure for tightening these bolts beyond this point.

Pretensioned structural bolts must be used in certain locations. Section J3.1 of the AISC specification requires that they be used for the following joints:

1. *Joints* that are subject to significant load reversal
2. *Joints* that are subject to fatigue load with no reversal of the loading direction
3. *Joints* with ASTM A325 or F1852 bolts that are subject to tensile fatigue
4. *Joints* with ASTM A490 or F2280 bolts that are subject to tension or combined shear and tension, with or without fatigue
5. Connections subjected to vibratory loads where bolt loosening is a consideration
6. End connections of built-up members composed of two shapes either interconnected by bolts or with at least one open side interconnected by perforated cover plates or lacing with tie plates, as required in Section E6.1 of the AISC Specification

In all other cases, A307 bolts and snug-tight A325 and A490 bolts can be used.

The use of ASTM F3125 structural bolts shall conform to the requirements of the Research Council on Structural Connections (RCSC) “Specification for Structural Joints Using Bolts,” 2004. This document contains all of the information on design, installation, inspection, washer use, compatible nuts, etc. for these bolts. Information on the installation, inspection, washer use, compatible nuts, etc. for F3043 and F3111 bolts is contained in the ASTM Specifications. There is no comparable document for A307 bolts. The RCSC “bolt spec.” was developed in the 1950s to allow the replacement of rivets with bolts.

Many sizes of high-strength bolts are available, as shown in Table 1.1. In general, a connection with a few large-diameter fasteners costs less than one of the same capacity with many small-diameter fasteners. The fewer the fasteners, the fewer the number of holes to be formed and the less installation work. Larger-diameter fasteners are generally favorable in connections, because the load capacity of a fastener varies with the square of the fastener diameter. For practical reasons, however, ¾- and ⅞-in-diameter fasteners are usually preferred. Shop and erection equipment is generally set up for these sizes, and workers are familiar with them. It is also advisable to limit the diameter of bolts that must be pretensioned to 1⅛ in since this is the largest diameter tension control (TC) bolt available.

TABLE 1.1 Thread Lengths for ASTM F3125 High-Strength Bolts

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

1.2.2 Washer Requirements

Washers are generally not required in snug-tightened joints. However, a beveled ASTM F436 washer should be used where the outer face of the bolted parts has a greater slope than 1:20 with respect to a plane normal to the bolt axis. Additionally, an ASTM F436 washer must be provided to cover the hole when a slotted or oversized hole occurs in an outer ply. Alternatively a 5⁄16 in common plate washer can be used to cover the hole.

Washers conforming to ASTM F436 are required in pretensioned and slip-critical joints as indicated in Table 1.2.

TABLE 1.2 Washer Requirements for High Strength Bolts

<table>
<tr><th colspan="8">Washer Requirements for Pretensioned or Slip-Critical Joints*</th></tr>
<tr><th rowspan="2">Bolt type</th><th rowspan="2">Bolt dia. (in)</th><th rowspan="2">Fy< 40</th><th colspan="3">Installation method</th><th colspan="2">Hole in outer ply</th></tr>
<tr><th>Calibrated wrench</th><th>Twist-off tension control</th><th>Direct tension indicator</th><th>OVS or SSL</th><th>LSL</th></tr>
<tr><td>A325 or F1852</td><td>≤1½</td><td>Not REQ'D</td><td rowspan="3">REQ'D Under turned element</td><td rowspan="3">REQ'D Under nut</td><td rowspan="3">REQ'D See RCSC spec. for location</td><td>REQ'D</td><td>5⁄16" Plt. washer or Cont. bar</td></tr>
<tr><td rowspan="2">A490 or F2280</td><td>≤1</td><td rowspan="2">REQ'D†</td><td>REQ'D</td><td rowspan="2">REQ'D w/ 3⁄8" Plt. washer or Cont. bar</td></tr>
<tr><td>>1</td><td>REQ'D 5⁄16" thick‡</td></tr>
</table>

*REQ'D indicates a washer conforming to ASTM F436 is required.
†Not required for F2280 with a circular head.
‡A 3⁄8 in plate washer and an ordinary thickness F436 washer may be used. The plate washer need not be hardened.

1.2.3 Pretensioned and Snug-Tight Bolts

As pointed out in a previous section, pretensioned bolts must be used for certain connections. For other locations, snug-tight bolts should be used because they are cheaper with no reduction in strength. The vast majority of shear connections in buildings can be snug tight, and shear connections are the predominate connection in every building.

1.2.4 Bearing-Type versus Slip-Critical Joints

Connections made with high-strength bolts may be slip-critical (material joined being clamped together by the tension induced in the bolts by tightening them and resisting shear through friction) or bearing-type (material joined being restricted from moving primarily by the bolt shank). In bearing-type connections, bolt threads may be included in or excluded from the shear plane. Different design strengths are used for each condition. Also, bearing-type connections may be either pretensioned or snug-tight, subject to the limitations already discussed. Snug-tight bolts are much more economical to install and should be used where permitted. The slip-critical connection is the most expensive, because it requires that the faying surfaces be free of paint, grease, and oil, or that a special paint be used. Hence this type of connection should be used only where required by the governing design specification, for example, where it is undesirable to have the bolts slip into bearing or where stress reversal could cause slippage. The 2016 AISC specification requires the use of slip-critical connections when

(a) Bolts are installed in oversized holes

(b) Bolts are installed in slotted holes with the direction of the load parallel to the slot

(c) Bolts joining the extended portion of bolted, partial-length cover plates, as required in Section F13.3

The RCSC specification further requires slip-critical connections for

(d) Joints that are subject to fatigue load with reversal of the loading direction

(e) Joints in which slip at the faying surfaces would be detrimental to the performance of the structure.

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

Threads Excluded from Shear Planes. The bearing-type connection with threads excluded from shear planes is the most economical high-strength bolted connection, because fewer bolts generally are needed for a given required strength. There can be difficulties involved in excluding the threads from the shear planes when either one or both of the outer plies of the joint is thin. The location of the thread runout or vanish depends on which side of the connection the bolt is entered and whether a washer is placed under the head or the nut. This location is difficult to control in the shop but even more so in the field. However, since for a given diameter of bolt the thread length is constant, threads can often be excluded in heavy joints with no additional effort.

Total nominal thread lengths and vanish thread lengths for high-strength bolts are given in Table 1.1. It is common practice to allow the last ⅛ in of vanish thread to extend across a single shear plane.

In order to determine the required bolt length, the value shown in Table 1.3 should be added to the grip (i.e., the total thickness of all connected material, exclusive of washers). For each hardened flat washer that is used, add 5⁄32 in and for each beveled washer, add 5⁄16 in. The tabulated values provide appropriate allowances for manufacturing tolerances and also provide for full thread engagement with an installed heavy hex nut. The length determined by the use of Table 1.3 should be adjusted to the next longer ¼ in length.

TABLE 1.3 Lengths to Be Added to Grip

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

1.2.5 Bolts in Combination with Welds

Due to differences in the rigidity and ductility of bolts as compared to welds, sharing of loads between bolts and welds should generally be avoided. However, the specification does not completely prohibit it.

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

1.2.6 Standard, Oversized, Short-Slotted, and Long-Slotted Holes

The AISC Specification requires that standard holes for bolts be 1⁄16 in larger than the nominal fastener diameter up to 1 in diameter and ⅛ in larger than the nominal diameter for larger bolts. The increased clearance for larger bolts, introduced in 2016 AISC Specification, may make the use of standard holes and snug-tight connections more practical in heavy construction. In computing net area or a tension member, the diameter of the hole should be taken $\frac{1}{16}$ in larger than the hole diameter.

Holes can be punched, drilled, or thermally cut. Punching usually is the most economical method. To prevent excessive damage to material around the hole, however, the maximum thickness of material in which holes are punched full size is often limited as summarized in Table 1.4.

TABLE 1.4 Maximum Material Thickness (in) for Punching Fastener Holes*

Type of steel	AISC
A36 steel	$d + \frac{1}{8}^{\dagger}$
High-strength steels	$d + \frac{1}{8}^{\dagger}$
Quenched and tempered steels	$\frac{1}{2}^{\ddagger}$

*Unless subpunching or subdrilling and reaming are used.
†d × fastener diameter, in.
‡A514 steel.

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

Oversized holes can be used in slip-critical connections, and the oversized hole can be in some or all the plies connected. The oversized holes are $\frac{3}{16}$ in larger than the bolt diameter for bolts ⅝ to ⅞ in in diameter. For bolts 1 in in diameter, the oversized hole is ¼ in larger and for bolts 1⅛ in in diameter and greater, the oversized of hole will be $\frac{5}{16}$ in larger.

Short-slotted holes can be used in any or all the connected plies. The load has to be applied 80 to 100° normal to the axis of the slot in bearing-type connections. Short slots can be used without regard to the direction of the applied load when slip-critical connections are used. The short slots for ⅝- to ⅞-in-diameter bolts are larger in length than the bolt diameter. For bolts 1 in in diameter, the length $\frac{5}{16}$ in larger and for bolts 1⅛ in diameter and larger, the slot will be ⅜ in longer in length.

Long slots have the same requirement as the short-slotted holes, except that the long slot has to be in only one of the connected parts at the faying surface of the connection. The width of all long slots for bolts matches the clearance for standard holes, and the length of the long slots for ⅝-in-diameter bolts is $\frac{5}{16}$ in greater, for ¾-in-diameter bolts 1⅛ in greater, for ⅞-in-diameter bolts $1\frac{5}{16}$ in greater, for 1-in-diameter bolts 1½ in greater, and for 1⅛-in-diameter and larger bolts, 2½ times diameter of bolt.

When finger shims are fully inserted between the faying surfaces of load transmitting parts of the connections, this is not considered as a long-slot connection.

1.2.7 Edge Distances and Spacing of Bolts

Minimum distances from centers of fasteners to any edges are given in Table 1.5.

TABLE 1.5 Minimum Edge Distances* (in) for Fastener Holes in Steel for Buildings

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

*Lesser distances are permitted if bolt edge tear-out is checked (J3.10).

†All edge distances in this column may be reduced ⅛ in when the hole is at point where stress does not exceed 25 percent of the maximum allowed stress in the element.

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

§d = fastener diameter in.

Source: From AISC "Specification for Structural Steel Buildings."

The AISC Specification has provisions for minimum edge distance: The distance from the center of a standard hole to an edge of a connected part should not be less than the applicable value from Table 1.5.

Maximum edge distances are set for sealing and stitch purposes. The AISC Specification limits the distance from center of fastener to nearest edge of parts in contact to 12 times the thickness of the connected part, with a maximum of 6 in. For unpainted weathering steel, the maximum is 7 in or 14 times the thickness of the thinner plate. For painted or unpainted members not subject to corrosion, the maximum spacing is 12 in or 24 times the thickness of the thinner plate.

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

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

The minimum distance between centers of fasteners should usually be at least 3 times the fastener diameter. However, the AISC Specification permits a minimum spacing of 2⅔ times the fastener diameter.

Limitations also are set on maximum spacing of fasteners, for several reasons. In built-up members, stitch fasteners, with restricted spacings, are used between components to ensure uniform action. Also, in compression members such fasteners are required to prevent local buckling.

Designs should provide ample clearance for tightening high-strength bolts. Detailers who prepare shop drawings for fabricators generally are aware of the necessity for this and can, with careful detailing, secure the necessary space. In tight situations, the solution may be staggering of holes (Fig. 1.3), variations from standard gages (Fig. 1.4), use of knife-type connections, or use of a combination of shop welds and field bolts.

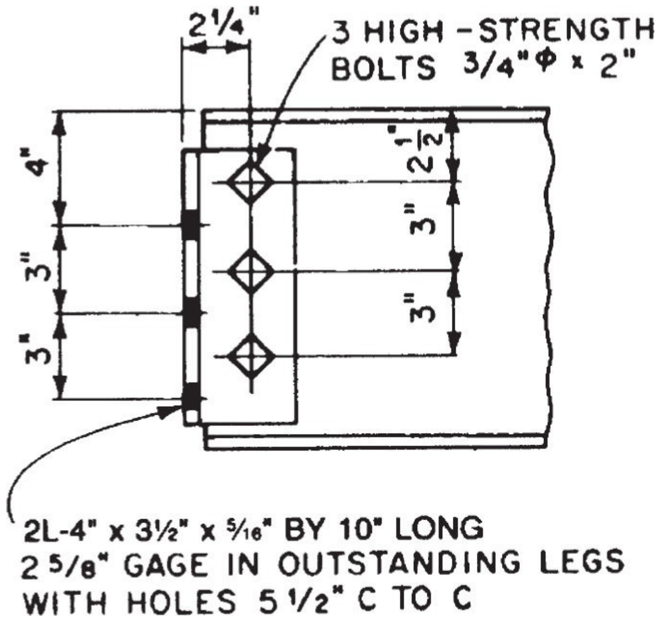


FIGURE 1.3 Staggered holes provide clearance for high-strength bolts.

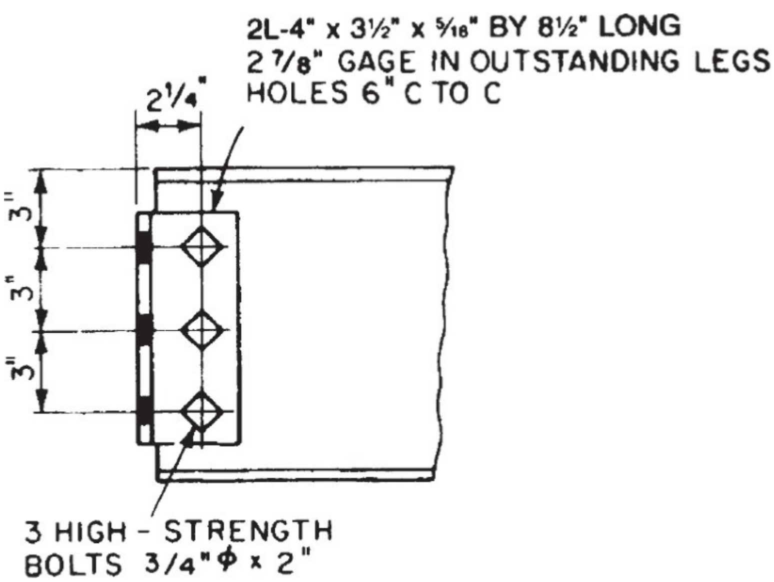


FIGURE 1.4 Increasing the gage in framing angles.

Minimum clearances for tightening high-strength bolts are indicated in Fig. 1.5 and Table 1.6.

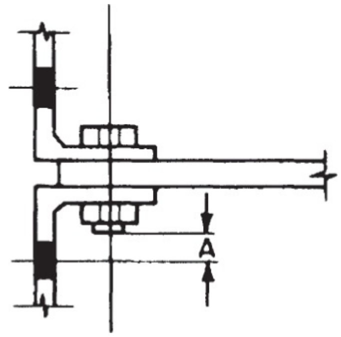


FIGURE 1.5 The usual minimum clearances.

TABLE 1.6 Clearances for High-Strength Bolts

Bolt diameter, in	Nut height, in	Usual minimum clearance, *A*, in	Minimum clearance for twist-off bolts, *A*, in	
			Small tool	Large tool
⅝	39⁄64	1	1⅝	—
¾	47⁄64	1¼	1⅝	1⅞
⅞	55⁄64	1⅜	1⅝	1⅞
1	63⁄64	1 7⁄16		1⅞
1⅛	1 7⁄64	1 9⁄16		—
1¼	1 7⁄32	1 11⁄16		—

1.2.8 Installation

All parts of a connection should be held tightly together during installation of fasteners. Drifting done during assembling to align holes should not distort the metal or enlarge the holes. Holes that must be enlarged to admit fasteners should be reamed. Poor matching of holes is cause for rejection though per the AISC Code of Standard Practice moderate amounts of reaming and the drawing of elements into line with drift pins is considered to be normal erection operations.

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

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

Tensioning should be done by one of the following methods, as given in the RCSC Specifications (2004).

Calibrated-Wrench Method. When a calibrated wrench is used, it must be set to cut off tightening when the required tension has been exceeded by 5 percent. The wrench should be tested periodically (at least daily on a minimum of three bolts of each diameter being used).

For this purpose, a calibrating device that gives the bolt tension directly should be used. In particular, the wrench should be calibrated when bolt size or length of air hose is changed. When bolts are tightened, bolts previously tensioned may become loose because of compression of the connected parts. The calibrated wrench should be reapplied to bolts previously tightened to ensure that all bolts are tensioned to the prescribed values.

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

1. *Fit up of connection*. Enough bolts are tightened a sufficient amount to bring contact surfaces together. This can be done with fit-up bolts, but it is more economical to use some of the final high-strength bolts.
2. *Snug tightening of bolts*. All high-strength bolts are inserted and made snug-tight (tightness obtained with a few impacts of an impact wrench or the full effort of a person using an ordinary spud wrench). While the definition of snug-tight is rather indefinite, the condition can be observed or learned with a tension-testing device.
3. *Nut rotation from snug-tight position*. All bolts are tightened by the amount of nut rotation specified in Table 1.7. If required by bolt-entering and wrench-operation clearances, tightening, including by the calibrated-wrench method, may be done by turning the bolt while the nut is prevented from rotating.

TABLE 1.7 Number of Nut or Bolt Turns from Snug-Tight Condition for High-Strength Bolts*

		Slope of outer faces of bolted parts	
Bolt length (Fig. 1.1)	Both faces normal to bolt axis	One face normal to bolt axis and the other sloped†	Bolt faces sloped†
Up to 4 diameters	1/3	1/2	2/3
Over 4 diameters but not more than 8 diameters	1/2	2/3	5/6
Over 5 diameters but not more than 12 diameters‡	2/3	5/6	1

*Nut rotation is relative to the bolt regardless of whether the nut or bolt is turned. For bolts installed by 1/2 turn and less, the tolerance should be ±30°. For bolts installed by 2/3 turn and more, the tolerance should be ±45°. This table is applicable only to connections in which all material within the grip of the bolt is steel.

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

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

Direct Tension Indicator. The direct tension indicator (DTI) hardened-steel load-indicator washer has dimples on the surface of one face of the washer. When the bolt is tensioned, the dimples depress to the manufacturer’s specification requirements, and proper pretension can be verified by the use of a feeler gage. Special attention should be given to proper installation of flat hardened washers when load-indicating washers are used with bolts installed in oversize or slotted holes and when the load-indicating washers are used under the turned element.

Twist-Off-Type Tension-Control Bolts. The twist-off or TC bolt is a bolt with an extension to the actual length of the bolt. This extension will twist off when torqued to the required tension by a special torque gun. The use of TC bolts have increased for both shop and fieldwork, since they allow bolts to be tightened from one side, without restraining the element on the opposite face. A representative sample of at least three TC assemblies for each diameter and grade of fastener should be tested in a calibration device to demonstrate that the device can be torqued to 5 percent greater tension than that required.

For all pretensioning installation methods bolts should first be installed in all holes and brought to the snug-tight condition. All fasteners should then be tightened, progressing systematically from the most rigid part of the connection to the free edges in a manner that will minimize relaxation of previously tightened fasteners. In some cases, proper tensioning of the bolts may require more than a single cycle of systematic tightening.

An excellent source of information on bolt installation is the *Structural Bolting Handbook* (2016).

1.3 WELDED CONNECTIONS

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

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

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

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

Prequalified joints, welding procedures, and procedures for qualifying welders are covered by AWS D1.1, *Structural Welding Code—Steel*, American Welding Society (2006). Common types of welds with structural steels intended for welding when made in accordance with AWS specifications can be specified by note or by symbol with assurance that a good connection will be obtained.

In making a welded design, designers should specify only the amount and size of weld actually required. Generally, a $\frac{5}{16}$-in weld is considered the maximum size for a single pass. A $\frac{3}{8}$-in weld, while only $\frac{1}{16}$-in larger, requires three passes and engenders a great increase in cost.

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

1.3.1 Types of Welds

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

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

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

Fillet welds have the general shape of an isosceles right triangle (Fig. 1.6). The size of the weld is given by the length of leg. The strength is determined by the throat thickness, the shortest distance from the root (intersection of legs) to the face of the weld. If the two legs are unequal, the nominal size of the weld is given by the shorter of the legs. If welds are concave, the throat is diminished accordingly, and so is the strength.

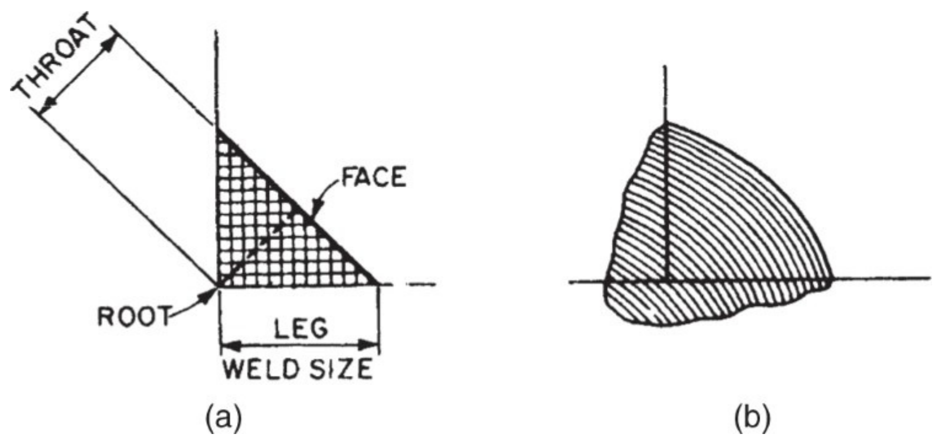


FIGURE 1.6 Fillet weld: (*a*) theoretical cross section and (*b*) actual cross section.

Fillet welds are used to join two surfaces approximately at right angles to each other. The joints may be lap (Fig. 1.7) or tee or corner (Fig. 1.8). Fillet welds also may be used with groove welds to reinforce corner joints. In a skewed tee joint, the included angle of weld deposit may vary up to 30° from the perpendicular, and one corner of the edge to be connected may be raised, up to $3/16$ in. If the separation is greater than $1/16$ in, the weld leg must be increased by the amount of the root opening. A further discussion of this is continued in Sec. 1.3.7.

FIGURE 1.7 Welded lap joint.

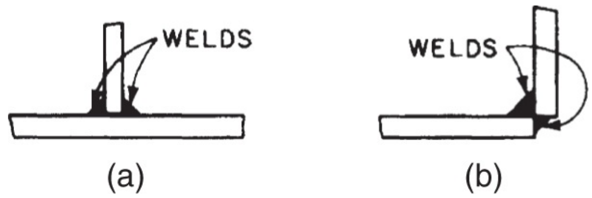


FIGURE 1.8 (*a*) Tee joint and (*b*) corner joint.

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

Standard types of groove welds are named in accordance with the shape given the edges to be welded: square, single V, double V, single bevel, double bevel, single U, double U, single J, and double J (Fig. 1.9). Edges may be shaped by flame cutting, arc-air gouging, or edge planing. Material up to ⅜ in thick, however, may be groove-welded with square-cut edges, depending on the welding process used.

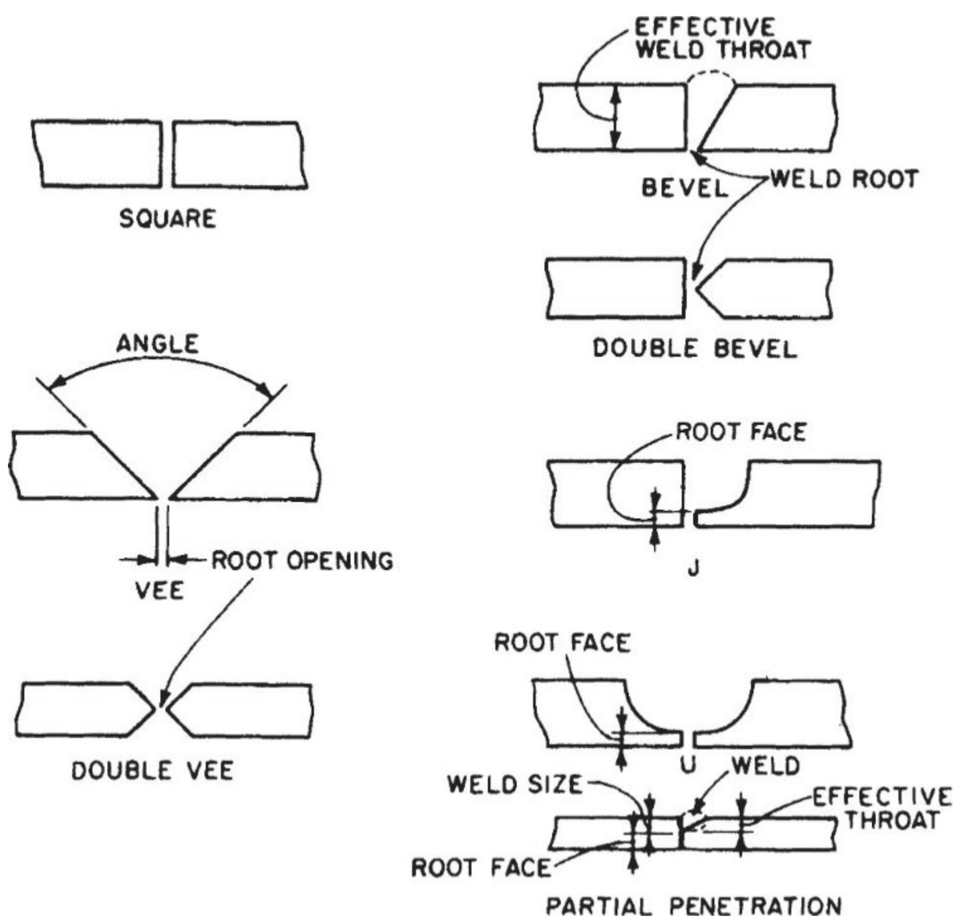


FIGURE 1.9 Groove welds.

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

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

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

Partial-joint-penetration welds should be used when forces to be transferred are less than those requiring a complete-joint-penetration weld. The edges may not be shaped over the full joint thickness, and the depth of the weld may be less than the joint thickness (Fig. 1.11). But even if the edges are fully shaped, groove welds made from one side without a backing bar or made from both sides without back gouging are considered partial-joint-penetration welds. They are often used for splices in building columns carrying axial loads only.

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

Economy in Selection. In selecting a weld, designers should consider not only the type of joint but also the labor and volume of weld metal required. While the strength of a fillet weld varies with size, the volume of metal varies with the square of the size. For example, a ½-in fillet weld contains 4 times as much metal per inch of length as a ¼-in weld but is only twice as strong. In general, a smaller but longer fillet weld costs less than a larger but shorter weld of the same capacity.

Furthermore, small welds can be deposited in a single pass. Large welds require multiple passes. They take longer, absorb more weld metal, and cost more. As a guide in selecting welds, Table 1.8 lists the number of passes required for some frequently used types of welds. This table is only approximate. The actual number of passes can vary depending on the welding process used. Figure 1.10 shows the number of passes and fillet weld strength. It can be seen that cost, which is proportional to the number of passes increases much faster than strength.

TABLE 1.8 Number of Passes for Welds

Weld size,* in	Fillet welds	Single-bevel groove welds (backup weld not included) 30° bevel	45° bevel	Single-bevel groove welds (backup weld not included) 30° open	60° open	90° open
3/16	1					
1/4	1	1	1	2	3	3
5/16	1					
3/8	3	2	2	3	4	6
7/16	4					
1/2	4	2	2	4	5	7
5/8	6	3	3	4	6	8
3/4	8	4	5	4	7	9
7/8		5	8	5	10	10
1		5	11	5	13	22
1 1/8		7	11	9	15	27
1 1/4		8	11	12	16	32
1 3/8		9	15	13	21	36
1 1/2		9	18	13	25	40
1 3/4		11	21			

*Plate thickness for groove welds.

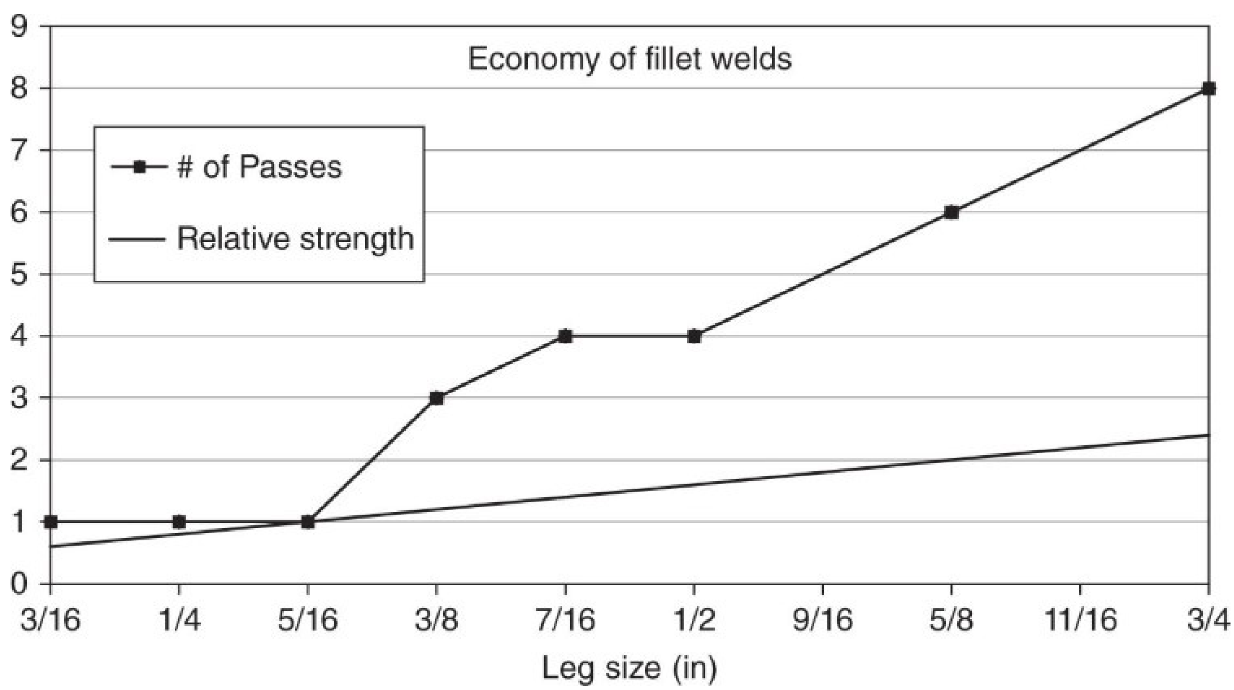


FIGURE 1.10 Relationship of number of passes to strength.

Double-V and double-bevel groove welds contain about half as much weld metal as single-V and single-bevel groove welds, respectively (deducting effects of root spacing). Cost of edge preparation and added labor of gouging for the back pass, however, should be considered. Also, for thin material, for which a single weld pass may be sufficient, it is uneconomical to use smaller electrodes to weld from two sides. Furthermore, poor accessibility or less favorable welding position (Sec. 1.3.4) may make an unsymmetrical groove weld more economical, because it can be welded from only one side.

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

For a given size of fillet weld, the cooling rate is faster and the restraint is greater with thick plates than with thin plates. To prevent cracking due to resulting internal stresses, the AISC Specification Section J2.2 sets minimum sizes for fillet welds depending on plate thickness, see Table 1.9.

TABLE 1.9 Minimum Plate Thickness for Fillet Welds

Minimum size of fillet welds,* in	Thickness of thinner part joined, in†	Minimum plate thickness for fillet welds on each side of the plate, in	
		36-ksi steel	50-ksi steel
⅛‡	To ¼ inclusive	0.213	0.190
$^3/_{16}$	Over ¼ to ½	0.320	0.286
¼	Over ½ to ¾	0.427	0.381
$^5/_{16}$	Over ¾	0.534	0.476

*Single pass fillets must be used.
†Plate thickness is the thickness of the thinner part joined.
‡Minimum weld size for structures subjected to dynamic loads is $^3/_{16}$ in.

To prevent overstressing of base material at a fillet weld the maximum weld size is limited by the strength of the adjacent base metal.

A limitation is also placed on the maximum size of fillet welds along edges. One reason is that edges of rolled shapes are rounded, and weld thickness consequently is less than the nominal thickness of the part. Another reason is that if weld size and plate thickness are nearly equal, the plate comer may melt into the weld, reducing the length of weld leg and the throat. Hence the AISC Specification requires in Section J2.2b the following: *Along edges of material less than ¼ in thick, maximum size of fillet weld may equal material thickness. But along edges of material ¼ in or more thick, the maximum size should be* $^1/_{16}$ *in less than the material thickness.*

Weld size may exceed this, however, if drawings definitely show that the weld is to be built out to obtain full throat thickness. AWS D1.1 requires that the minimum-effective length of a fillet weld be at least 4 times the nominal size, or else the weld must be considered not to exceed 25 percent of the effective length.

Subject to the preceding requirements, intermittent fillet welds maybe used in buildings to transfer calculated stress across a joint or faying surfaces when the strength required is less than that developed by a continuous fillet weld of the smallest permitted size. Intermittent fillet welds also may be used to join components of built-up members in buildings.

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

For groove welds, the weld lengths specified on drawings are effective weld lengths. They do not include distances needed for start and stop of welding. These welds must be started or stopped on run-off pads beyond the effective length. The effective length of straight fillet welds is the overall length of the full size fillet. No reduction in effective length need be taken in design calculations to allow for the start or stop weld crater.

The AISC Specification requires fillet weld terminations to be detailed in a manner that does not result in a notch in the base metal subject to applied tension loads. An accepted practice to avoid notches in base metal is to stop fillet welds short of the edge of the base metal by a length approximately equal to the size of the weld. In most welds the effect of stopping short can be neglected in strength calculations. A weld that is not stopped short of the edge is not cause for rejection unless the welding results in a harmful notch.

The AISC Specification also requires welds to allow deformation to accommodate assumed design conditions. Examples include

- Welds on the outstanding legs of beam clip angle connections are returned on the top of the outstanding leg and stopped no more than 4 times the weld size and not greater than half the leg width from the outer toe of the angle.
- Fillet welds connecting transverse stiffeners to webs of girders that are ¾ in thick or less are stopped 4 to 6 times the web thickness from the web toe of the flange-to web fillet weld, except where the end of the stiffener is welded to the flange. End returns should be indicated on design and detail drawings.

Fillet welds deposited on opposite sides of a common plane of contact between two parts must be interrupted at a corner common to both welds. An exception to this requirement must be made when seal welding parts prior to hot-dipped galvanizing.

If longitudinal fillet welds are used alone in end connections of flat-bar tension members, the length of each fillet weld should at least equal the perpendicular distance between the welds.

In material ⅝ in or less thick, the thickness of plug or slot welds should be the same as the material thickness. In material greater than ⅝ in thick, the weld thickness should be at least half the material thickness but not less than ⅝ in.

The diameter of the hole for a plug weld should be at least equal to the depth of the hole plus $\frac{5}{16}$ in, but the diameter should not exceed 2¼ times the thickness of the weld.

Thus, the hole diameter in ¾-in plate could be a minimum of ¾ + $\frac{5}{16}$ = $1\frac{1}{16}$ in. The depth of weld metal would be at least ⅝ in > (½ × ¾ = ⅜ in).

Plug welds may not be spaced closer center-to-center than 4 times the hole diameter.

The length of the slot for a slot weld should not exceed 10 times the thickness of the weld. The width of the slot should not be less than the thickness of the part containing it plus $\frac{5}{16}$ in rounded to the next larger $\frac{1}{6}$ in, but the width should not exceed 2¼ times the weld thickness.

Thus, the width of the slot in ¾-in plate could be a minimum of ¾ + $\frac{5}{16}$ = $1\frac{1}{16}$ in. The weld metal depth would be at least ⅝ in > (½ × ¾ = ⅜ in). The slot could be up to 10 × ⅝ = 6¼ in long.

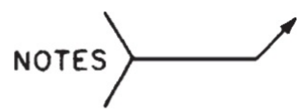
Slot welds may be spaced no closer than 4 times their width in a direction transverse to the slot length. In the longitudinal direction, center-to-center spacing should be at least twice the slot length.

1.3.2 Welding Symbols

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

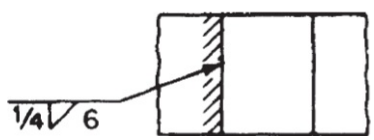
Extending from either end of the line, the arrow should point to the joint in the same manner as the electrode would be held to do the welding.

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

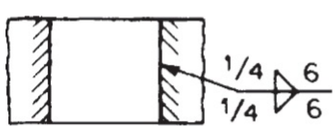


The type and length of weld are indicated above or below the line. If noted below the line, the symbol applies to a weld on the arrow side of the point, the side to which the arrow points. If noted above the line, the symbol indicates that the other side, the side opposite the one to which the arrow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

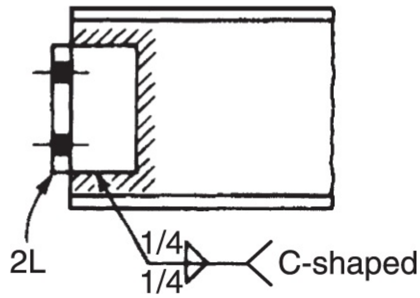
A fillet weld is represented by a right triangle extending above or below the line to indicate the side on which the weld is to be made. The vertical leg of the triangle is always on the left.



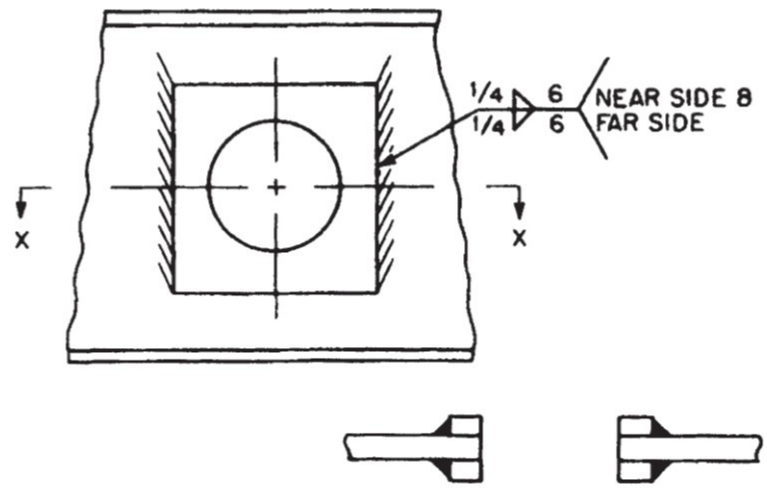
The preceding symbol indicates that a ¼-in fillet weld 6 in long is to be made on the arrow side of the assembly. The following symbol requires a ¼-in fillet weld 6 in long on both sides:



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

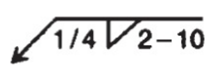


The length of the weld is not shown on the symbol in this case because the connection requires a continuous weld for the full length of each angle on both sides of the angle. Care must be taken not to omit the length unless a continuous full-length weld is wanted. “Continuous” should be written on the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

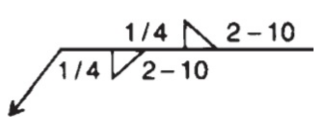


SECTION X – X

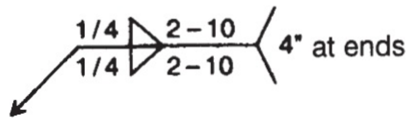
For many members, a stitch or intermittent weld is sufficient. It may be shown as



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

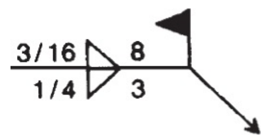


Usually, intermittent welds are started and finished with a weld at least twice as long as the length of the stitch welds. This information is given in a tail note:



In the previous three figures, intermittent fillets are shown as, for example, 2-10. This is the notation recommended by AWS, but it can lead to confusion on shop drawings, where dimensions are given in feet and inches as for instance, 2 ft-10, with no inch symbol. Therefore, 2-10 on a weld symbol could be mistaken as 2 ft, 10 in rather than 2 in at 10 in. It would be less ambiguous to use the "at" symbol, @, rather than the hyphen, -. Then the weld symbol would read 2 @ 10, which is unambiguous.

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



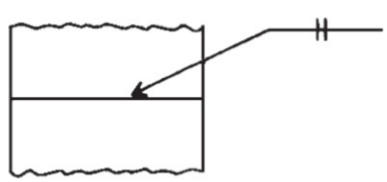
This is important in ensuring that the weld will be made as required. Often, a tail note is advisable for specifying field welds.

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

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

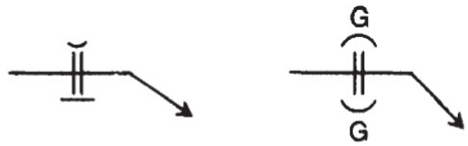
The preceding devices for providing information with fillet welds also apply to groove welds. In addition, groove-weld symbols must designate material preparation required. This often is best shown on a cross section of the joint.

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



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

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



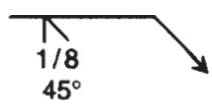
The root-opening size for a groove weld is written in within the symbol indicating the type of weld. For example, a ⅛-in root opening for a square-groove weld with a backing bar is specified by

Note that the “M” in the backing bar symbol indicates that the material to be used for backing is specified.

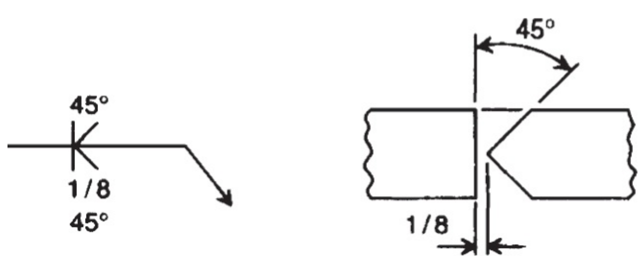
A ⅛-in root opening for a bevel weld, not to be ground, is indicated by

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

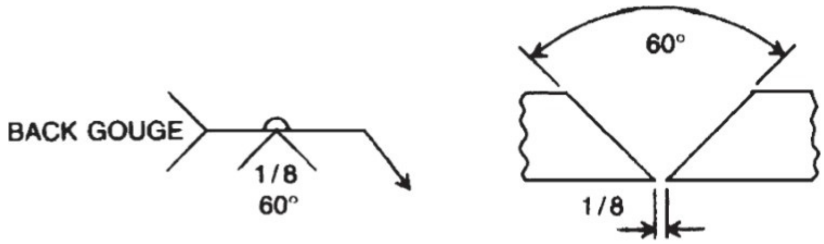
The angle at which the material is to be beveled should be indicated with the root opening:



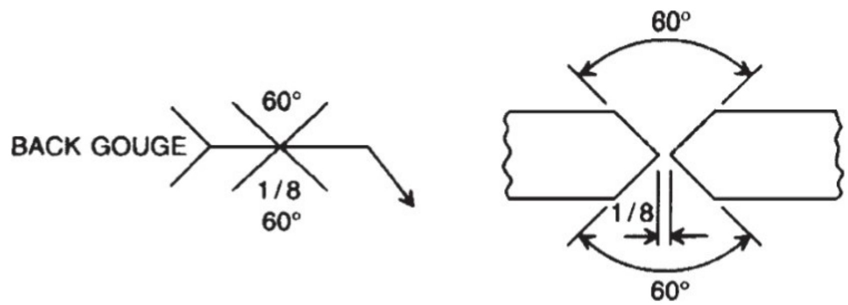
A double-bevel weld is specified by



A single-V weld is represented by



A double-V weld is indicated by



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

Where groove preparation is not symmetrical and complete, additional information should be given on the symbol. Also it may be necessary to give weld-penetration information, as in Fig. 1.11. For the weld shown, penetration from either side must be a minimum of $^3/_{16}$ in. The second side should be back-gouged before the weld there is made.

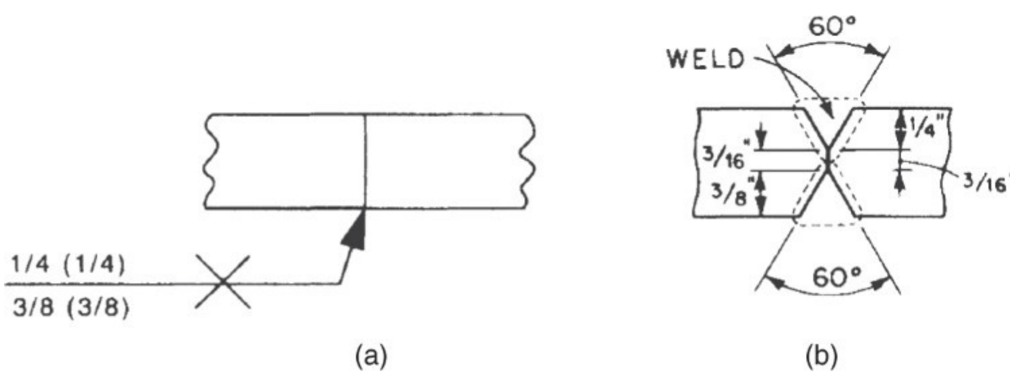


FIGURE 1.11 Penetration information is given on the welding symbol in (*a*) for the weld shown in (*b*). Penetration must be at least $^3/_{16}$ in. Second side must be back-gouged before the weld on that side is made.

Welds also may be a combination of different groove and fillet welds. While symbols can be developed for these, designers will save time by supplying a sketch or enlarged cross section. It is important to convey the required information accurately and completely to the workers who will do the job.

1.3.3 Welding Material

Weldable structural steels permissible in buildings are listed in AISC Specification A3. Matching electrodes are given in AWS D1.1 Table 3.1.

1.3.4 Welding Positions

The position of the stick electrode relative to the joint when a weld is being made *affects* welding economy and quality.

The basic welding positions are as follows:

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

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

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

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

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

AWS D1.1 requires that only the flat position be used for submerged-arc welding, except for certain sizes of fillet welds. Single-pass fillet welds may be made in the flat or the horizontal position in sizes up to $\frac{5}{16}$ in with a single electrode and up to ½ in with multiple electrodes. Other positions are prohibited.

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

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

In addition, adequate space must be provided for deposition of the required size of the fillet weld. For example, to provide an adequate landing c, in, for the fillet weld of size D, in, in Fig. 1.12, c should be at least $D + \frac{5}{16}$. In building column splices, however, $c = D + \frac{3}{16}$ often is used for welding splice plates to fillers.

FIGURE 1.12 Minimum landing for a fillet weld.

1.3.5 Weld Procedures

Welds should be qualified and should be made only by welders, welding operators, and tackers qualified as required in AWS D1.1 for buildings. Welding should not be permitted under any of the following conditions:

When the ambient temperature is below 0°F
When surfaces are wet or exposed to rain, snow, or high wind
When welders are exposed to inclement conditions

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

Parts to be fillet-welded should be in close contact. The gap between parts should not exceed $^3/_{16}$ in. If it is more than $^1/_{16}$ in, the fillet weld size should be increased by the amount of separation. The separation between faying surfaces for plug and slot welds and for butt joints landing on a backing should not exceed $^1/_{16}$ in. Parts to be joined at butt joints should be carefully aligned. Where the parts are effectively restrained against bending due to eccentricity in alignment, an offset not exceeding 10 percent of the thickness of the thinner part joined, but in no case more than ⅛ in, is permitted as a departure from theoretical alignment. When correcting misalignment in such cases, the parts should not be drawn in to a greater slope than ½ in in 12 in.

For permissible welding positions, see Sec. 1.3.4. Work should be positioned for flat welding whenever practicable.

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

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

With too rapid cooling, cracks might form in a weld. Possible causes are shrinkage of weld and heat-affected zone, austenite-martensite transformation, and entrapped hydrogen.

Preheating the base metal can eliminate the first two causes. Preheating reduces the temperature gradient between weld and adjacent base metal, thus decreasing the cooling rate and resulting stresses. Also, if hydrogen is present, preheating allows more time for this gas to escape. Use of low-hydrogen electrodes, with suitable moisture control, is also advantageous in controlling hydrogen content.

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

To avoid cracks and for other reasons, AWS specifications require that under certain conditions, before a weld is made the base metal must be preheated. Table 1.10 lists typical preheat and interpass temperatures. The table recognizes that as plate thickness, carbon content, or alloy content increases, higher preheats are necessary to lower cooling rates and to avoid microcracks or brittle heat-affected zones.

TABLE 1.10 Requirements of AWS D1.1 for Minimum Preheat and Interpass Temperatures, °F, for Welds in Buildings for Some Commonly Used Structural Steels*

	Shielded metal-arc with other than low-hydrogen electrodes	Shielded metal-arc with low-hydrogen electrodes; submerged-arc, gas metal-arc, or flux-cored arc	Shielded metal-arc with low-hydrogen electrodes; submerged-arc, gas metal-arc, or flux-cored arc	Shielded metal-arc; submerged-arc, gas metal-arc, or flux-cored arc with electrodes or electrode-flux combination capable of depositing weld metal with a maximum diffusible hydrogen content of 8 mL/100 g when tested in accordance with AWS A4.3
Thickness at thickest part at point of welding, in	**ASTM A36, A53 grade B, A501, A529O**	**ASTM A36, A53 grade B, A441, A501, A529 grades 50 and 55, A572 grades 42, 50, and 55, A588, A992**	**ASTM A572 grade 60 and 65**	**ASTM A913‡ grades 50, 60, and 65**
To ¾	32†	32†	50	32†
Over ¾ to 1½	150	50	150	32†
1½ to 2½	225	150	225	32†
Over 2½	300	225	300	32†

*In joints involving different base metals, preheat as specified for higher-strength base metal.

†When the base-metal temperature is below 32°F, the base metal shall be preheated to at least 70°F and the minimum interpass temperature shall be maintained during welding.

‡The heat input limitations of AWS D1.1 paragraph 5.7 shall not apply to A913.

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

Preheat and interpass temperatures should be sufficient to prevent crack formation. Temperatures above the minimums in Table 1.10 may be required for highly restrained welds.

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

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

Tack and other temporary welds are subject to the same quality requirements as final welds. For tack welds, however, preheat is not mandatory for single-pass welds that are remelted and incorporated into continuous submerged-arc welds. Also, defects such as undercut, unfilled craters, and porosity need not be removed before final submerged-arc welding. Welds not incorporated into final welds should be removed after they have served their purpose, and the surface should be made flush with the original surface.

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

Groove welds should be terminated at the ends of a joint in a manner that will ensure sound welds. Where possible, this should be done with the aid of weld tabs or runoff plates. AWS D1.1 does not require removal of weld tabs for statically loaded structures but does require it for dynamically loaded structures. The AISC Seismic Provisions (2005) also require their removal in zones of high seismicity. The ends of the welds then should be made smooth and flush with the edges of the abutting parts.

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

AWS D1.1 presents details of techniques acceptable for welding buildings. These techniques include handling of electrodes and fluxes.

1.3.6 Weld Quality

A basic requirement of all welds is thorough fusion of weld and base metal and of successive layers of weld metal. In addition, welds should not be handicapped by craters, undercutting, overlap, porosity, or cracks. (AWS D1.1 gives acceptable tolerances for these defects.) If craters, excessive concavity, or undersized welds occur in the effective length of a weld, they should be cleaned and filled to the full cross section of the weld. Generally, all undercutting (removal of base metal at the toe of a weld) should be repaired by depositing weld metal to restore the original surface. Overlap (a rolling over of the weld surface with lack of fusion at an edge), which may cause stress concentrations, and excessive convexity should be reduced by grinding away of excess material (Figs. 1.13 and 1.14). If excessive porosity, excessive slag inclusions, or incomplete fusion occur, the defective portions should be removed and rewelded. If cracks are present, their extent should be determined by acid etching, magnetic-particle inspection, or other equally positive means. Not only the cracks but also sound metal 2 in beyond their ends should be removed and replaced with the weld metal. Use of a small electrode for this purpose reduces the chances of further defects due to shrinkage. An electrode not more than $\frac{5}{32}$ in in diameter is desirable for depositing weld metal to compensate for size deficiencies.

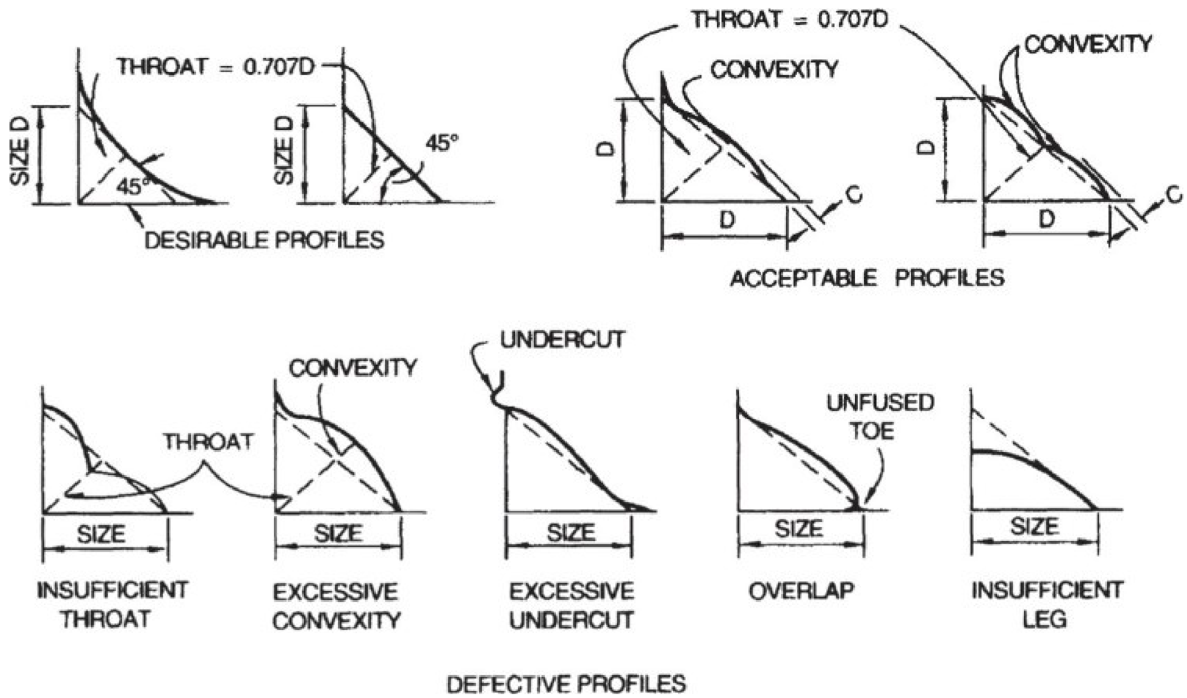


FIGURE 1.13 Profiles of fillet welds.

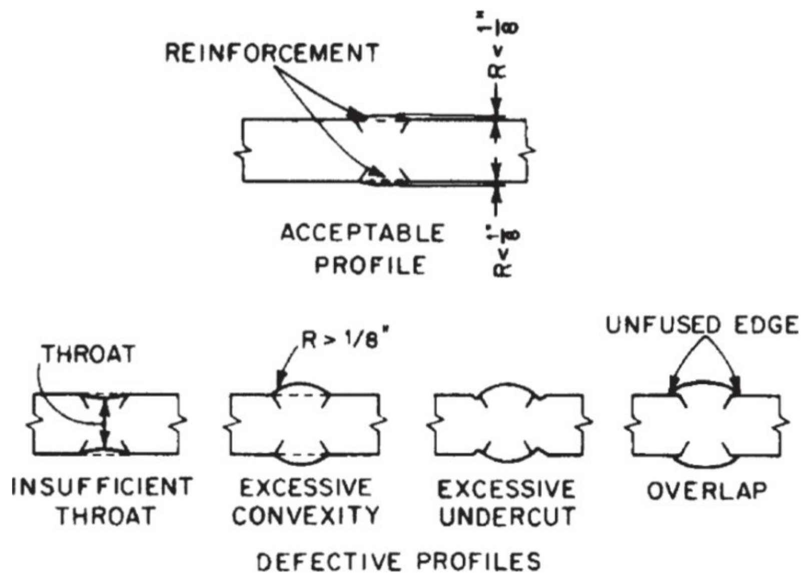


FIGURE 1.14 Profiles as groove welds.

AWS D1.1 limits convexity C to the values in Table 1.11.

TABLE 1.11 AWS D1.1 Limits on Convexity of Fillet Welds

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

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

Welds may be inspected by one or more methods: visual inspection; nondestructive tests, such as ultrasonic, x-ray, dye penetration, magnetic particle, and cutting of samples from finished welds. Designers should specify which welds are to be examined, extent of the examination, and methods to be used.

1.3.7 Methods for Determining Strength of Skewed Fillet Welds

It is often beneficial to utilize skewed single-plate or end-plate shear connections to carry members which run nonorthogonal to their supports. In such case the welds attaching the connection material to the support must be designed to accommodate this skew. There are two ways to do this. The AWS D1.1 Structural Welding Code provides a method to calculate the effective throat for skewed T joints with varying dihedral angles, which is based on providing equal strength in the obtuse and acute welds. This is shown in Fig. 1.15*a*. The AISC method is simpler, and simply increases the weld size on the obtuse side by the amount of the gap, as is shown in Fig. 1.15*c*.

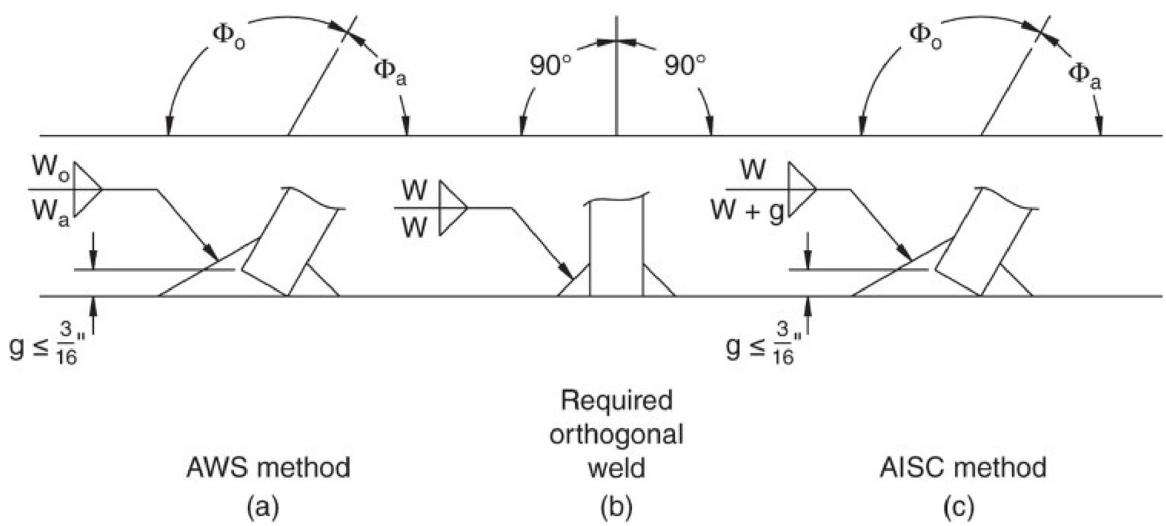


FIGURE 1.15 Skewed fillet weld sizes required to match strength of required orthogonal fillets of size *W*.

Both methods can be shown to provide a strength equal to or greater than the required orthogonal weld size of *W*. The main difference with regard to strength is that the AWS method, as given by the formulas in Fig. 1.16, maintains equal strength in both fillets, whereas the AISC method increases the strength on the acute side by maintaining a constant fillet size, $W_a = W$, while the increased size, $W_o = W + g$, on the obtuse side actually loses strength because of the gap, g. Nevertheless, it can be shown that the sum of the strengths of these two fillet welds, $W_a = W$ and $W_o = W + g$, is always greater than the $2W$ of the required orthogonal fillets.

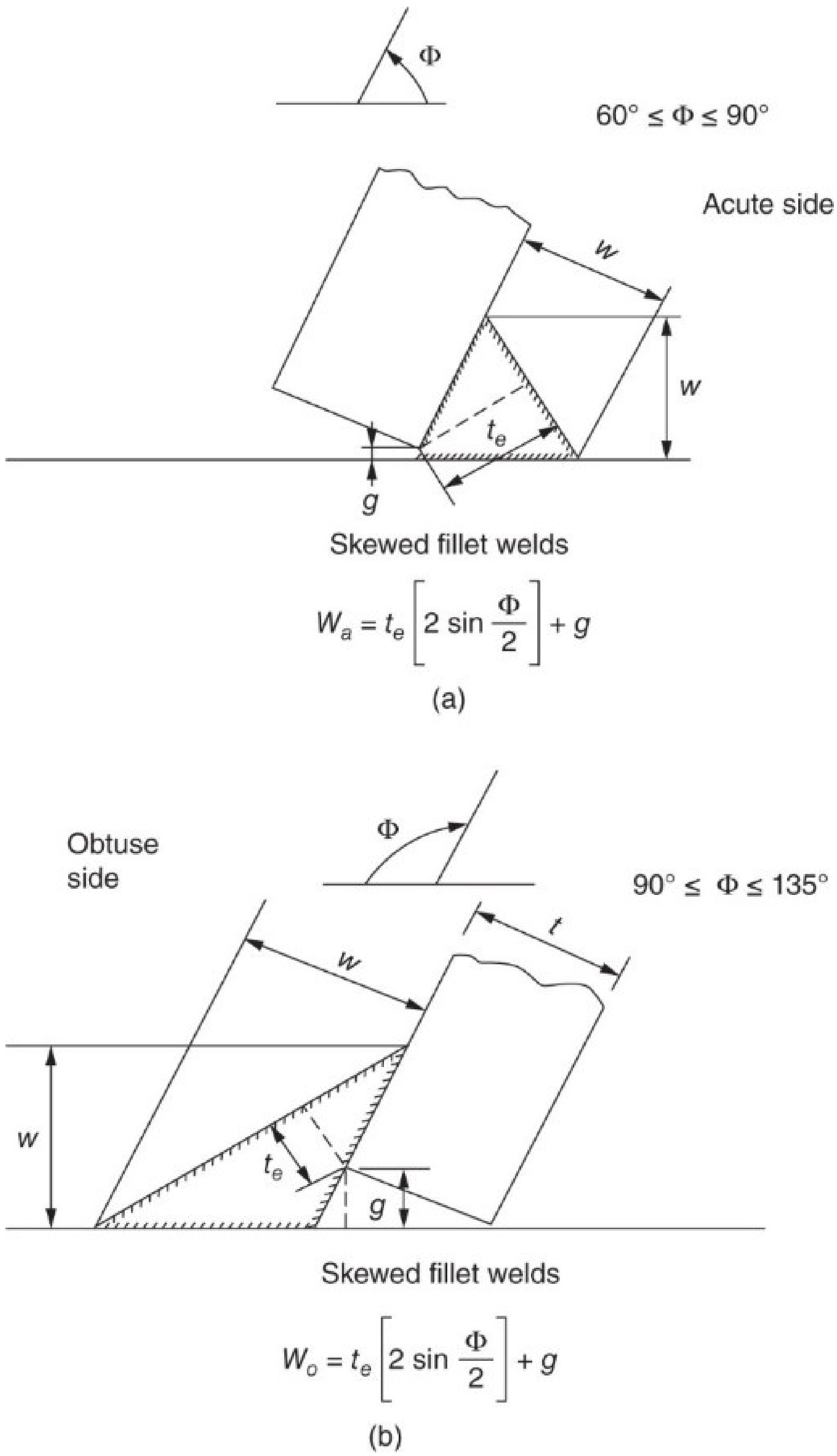


FIGURE 1.16 Geometry of skewed fillet welds. (Relationship of weld size to effective throat, t_e.) (*a*) Acute side, (*b*)

obtuse side. Note how the skewed fillet welds are to be measured. The contact leg length is *not* the weld size.

It should be noted that the gap, g, is limited to a maximum value of $\frac{3}{16}$ in for both methods.

The effects of the skew on the effective throat of a fillet weld can be very significant as shown in Fig. 1.16. Figure 1.16 also shows how fillet legs W_o and W_a are measured in the skewed configuration. On the acute side of the connection the effective throat for a given fillet weld size gradually increases as the connection intersection angle, Φ, changes from 90° to 60°. From 60° to 30°, the weld changes from a fillet weld to a partial joint penetration (PJP) groove weld (Fig. 1.17) and the effective throat, t_e, decreases due to the allowance, z, for the unwelded portion at the root. While this allowance varies based on the welding process and position, it can conservatively be taken as the throat less ⅛ in for 60° to 45° and less ¼ in for 45° to 30°. Joints less than 30° are not prequalified and generally should not be used.

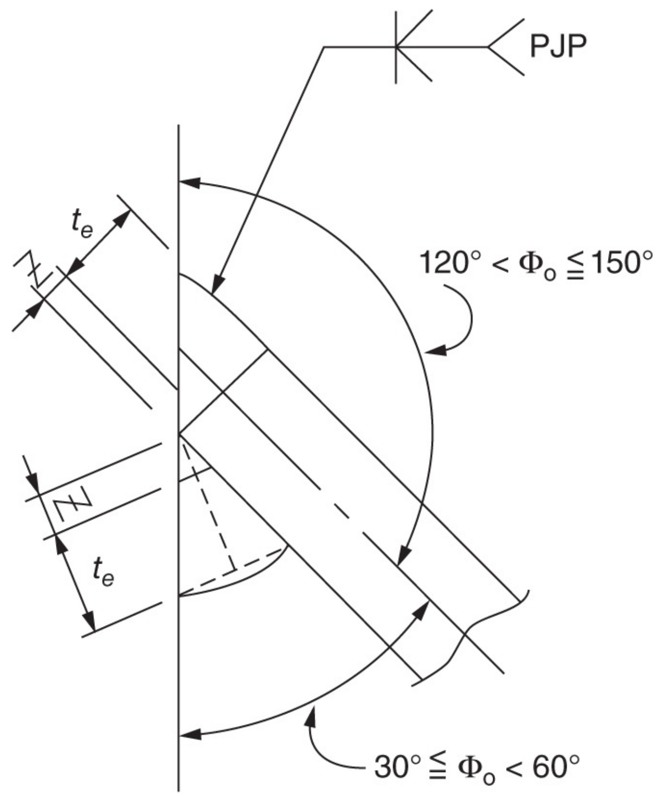


FIGURE 1.17 Acute angles less than 60° and obtuse angles greater than 120°.

1.3.8 Obliquely Loaded Concentric Fillet Weld Groups

The strength of a fillet weld is dependent on the direction of loading. Welds that are loaded in their longitudinal direction have a design strength of $0.6F_{EXX}$, while welds loaded transverse to their longitudinal axis have a design strength 1.5 times greater. The strength of welds loaded between these extremes can be found as

$$F_w = 0.6F_{EXX}\,(1.0 + 0.50\sin^{1.5}\theta)$$

This equation is easily applied to a single-line weld, or a group of parallel-line welds, but when applied to weld groups containing welds loaded at differing angles, such as that given in

Fig. 1.18, its application becomes much more complex. In such cases, deformation compatibility must also be satisfied. Since the transversely loaded welds are considerably less ductile than the longitudinally loaded welds, the transversely loaded welds will fracture before the longitudinally loaded welds reach their full capacity. This can easily be seen by examining Fig. 1.19 (taken from Fig. 8.5 AISC 2005). A weld loaded transverse to its longitudinal direction will fracture at a deformation equal to approximately 0.056 times the weld size. At this same deformation the longitudinally loaded weld has only reached about 83 percent of its maximum strength.

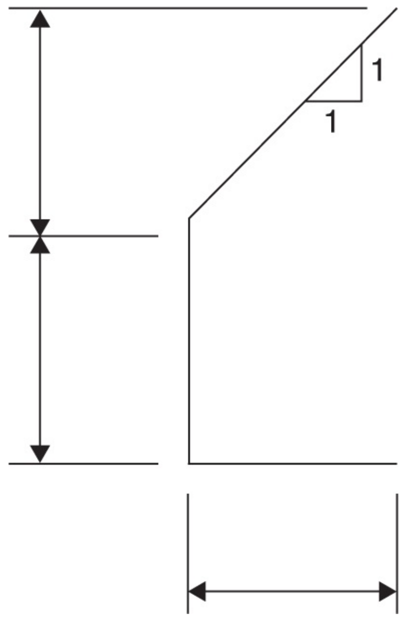


FIGURE 1.18 Obliquely loaded weld group.

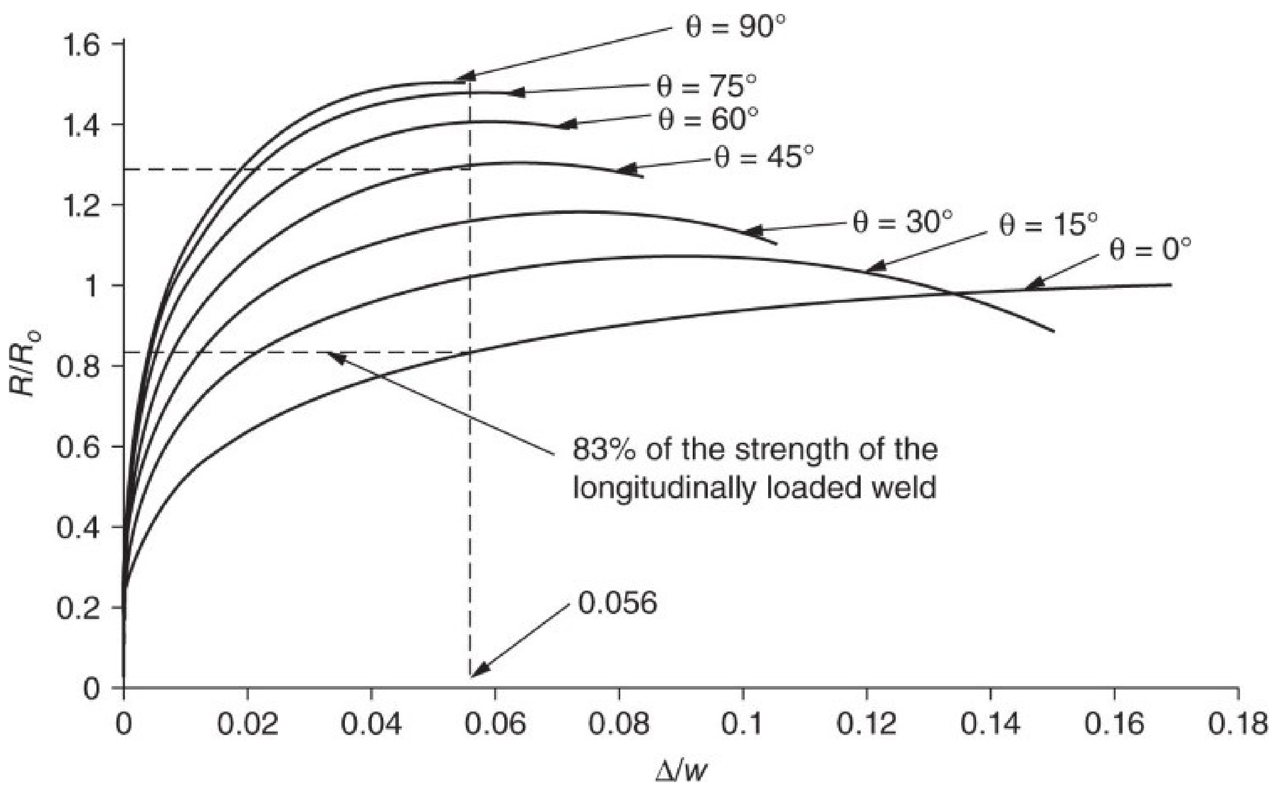


FIGURE 1.19 Graphical solution of the capacity of an obliquely loaded weld group. Alternately, if the welds are loaded only in the transverse and longitudinal directions, then the weld strength is permitted to taken as the greater of $R_n = R_{wl} + R_{wt}$

or $R_n = 0.85R_{wl} + 1.5R_{wt}$.

To account for this the strength of the weld is calculated as

$$R_{nx} = \sum F_{wix} A_{wi}$$
$$R_{ny} = \sum F_{wiy} A_{wi}$$

where A_{wi} = effective area of weld throat of any ith weld element, in^2

$F_{wi} = 0.6F_{EXX}(1 + 0.50 \sin^{1.5}\theta) f(p)$
F_{wi} = nominal stress in any ith weld element, ksi
F_{wix} = x component of stress F_{wi}
F_{wiy} = y component of stress F_{wi}
$F(p) = [p(1.9 - 0.9p)]^{0.3}$
$p = \Delta/\Delta_m$, ratio of element i deformation to its deformation at maximum stress
$\Delta_m = 0.209(\theta + 2)^{-0.32}w$, deformation of weld element at maximum stress, in (mm)
Δ_i = deformation of weld elements at intermediate *stress* levels, linearly proportioned to the critical deformation based on distance from the instantaneous center of rotation, r_i, in $= \Delta_i = r_i\Delta_u / r_{crit}$
$\Delta_u = 1.087(\theta + 6)^{-0.65}w \leq 0.17w$, deformation of weld element at ultimate stress (fracture), usually in element furthest from instantaneous center of rotation, in (mm)
w = leg size of the fillet weld, in
r_{crit} = distance from instantaneous center of rotation to weld element with minimum Δ_u/r_i ratio, in

This can be accomplished graphically using Fig. 1.19, the load-deformation curves. For example, to find the strength of the concentrically loaded weld group shown in Fig. 1.18, first the least ductile weld is determined. In this case it is the transversely loaded weld. By drawing a vertical line from the point of fracture, the strength increase or decrease for the remaining elements can be determined. In this case the strength of the weld group of Fig. 1.18, with I = 1 m, is found to be

$$\phi R_w = (D)(1.392)(1.5(1) + 1.29(1.41) + 0.83(1)) = 5.78D$$

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CHAPTER 2

DESIGN OF CONNECTIONS FOR AXIAL, MOMENT, AND SHEAR FORCES

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(*Courtesy of The Steel Institute of New York.*)

2.1 INTRODUCTION

Connection design is an interesting subject because it requires a great deal of rational analysis in arriving at a solution. There are literally an infinite number of possible connection configurations, and only a very small number of these have been subjected to physical testing.

Even within the small group that has been tested, changes in load directions, geometry, material types, fastener type, and arrangement very quickly result in configurations that have not been tested and therefore require judgment and rational analysis on the part of the designer. This chapter provides design approaches to connections based on test data, when available, supplemented by rational design or art and science in the form of equilibrium (admissible force states), limit states, and ductility considerations. The limit states are those of the AISC Specification (2016).

2.1.1 Philosophy

Connection design is both an art and a science. The science involves equilibrium, limit states, load paths, and the lower bound theorem of limit analysis. The art involves the determination of the most efficient load paths for the connection, and this is necessary because most connections are statically indeterminate.

The lower bound theorem of limit analysis states: If a distribution of forces within a structure (or connection, which is a localized structure) can be found, which is in equilibrium with the external load and which satisfies the limit states, then the externally applied load is less than or at most equal to the load that would cause connection failure. In other words, any solution for a connection that satisfies equilibrium and the limit states yields a safe connection. This is the science of connection design. The art involves finding the internal force distribution (or load paths) that maximizes the external load at which a connection fails. This maximized external load is also the true failure load when the internal force distribution results in satisfaction of compatibility (no gaps and tears) within the connection in addition to satisfying equilibrium and the limit states.

It should be noted that, strictly speaking, the lower bound theorem applies only to yield limit states in structures that are ductile. Therefore, in applying it to connections, limit states involving stability and fracture (lack of ductility) must be considered to preclude these modes of failure.

2.1.2 General Procedure

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

2.1.3 Economic Considerations

For any given connection situation, it is usually possible to arrive at more than one satisfactory solution. Where there is a possibility of using bolts or welds, let the economics of fabrication and erection play a role in the choice. Different fabricators and erectors in different parts of the country have their preferred ways of working, and as long as the principles of connection design are followed to achieve a safe connection, local preferences should be accepted. Some additional considerations that will result in more economical connections (Thornton, 1995b) are:

1. For shear connections, provide the actual loads and allow the use of single plate and single angle shear connections. Do not specify full-depth connections or rely on the AISC uniform load tables.
2. For moment connections, provide the actual moments and the actual shears. Also, provide a “breakdown” of the total moment, that is, give the gravity moment and lateral moment due to wind or seismic loads separately. This is needed to do a proper check for column web doubler plates. If stiffeners are required, allow the use of fillet welds in place of complete joint penetration welds. To avoid the use of stiffeners, consider redesigning with a heavier column to eliminate them.
3. For bracing connections, in addition to providing the brace force, also provide the beam shear and axial transfer force. The transfer force is the axial force that must be transferred to the opposite side of the column. The transfer force is not necessarily the beam axial force that is obtained from a computer analysis of the structure. See Thornton (1995b) and Muir and Thornton (2014) for a discussion of this. A misunderstanding of transfer forces can lead to both uneconomic and unsafe connections.

2.1.4 Types of Connections

There are three basic forces to which connections are subjected. These are axial force, shear force, and moment. Many connections are subject to two or more of these simultaneously. Connections are usually classified according to the major load type to be carried, such as shear connections, which carry primarily shear; moment connections, which carry primarily moment; and axial force connections, such as splices, bracing and truss connections, and hangers, which carry primarily axial force. Subsequent sections of this chapter will deal with these three basic types of connections.

2.1.5 Organization

This chapter will cover axial force connections first, then moment connections, and lastly shear connections. This is done to emphasize the ideas of load paths, limit states, and the lower bound theorem, which (except for limit states) are less obviously necessary to consider for the simpler connections.

This chapter is based on the limit states of the AISC Specification (AISC, 2016). The determination of loads, that is, required strengths, is dependent upon the specific building code required for the project, based on location, local laws, and so forth. At this time (2008), there is much transition taking place in the determination of seismic loads and connection requirements. Wherever examples involving seismic loads are presented in this chapter, the solutions presented are indicative of the author's experience in current practice with many structural engineers, and may need to be supplemented with additional requirements from local seismic codes. Chapter 5 deals with connections in high seismic regions and covers these additional requirements.

2.2 AXIAL FORCE CONNECTIONS

2.2.1 Bracing Connections

2.2.1.1 Introduction. The lateral force-resisting system in buildings may consist of a vertical truss. This is referred to as a braced frame and the connections of the diagonal braces to the beams and columns are the bracing connections. Figure 2.1 shows various bracing arrangements. For the bracing system to be a true truss, the bracing connections should be concentric, that is, the gravity axes of all members at any joint should intersect at a single point. If the gravity axes are not concentric, the resulting couples must be considered in the design of the members. The examples of this section will be of concentric type, but the nonconcentric type can also be handled as will be shown.

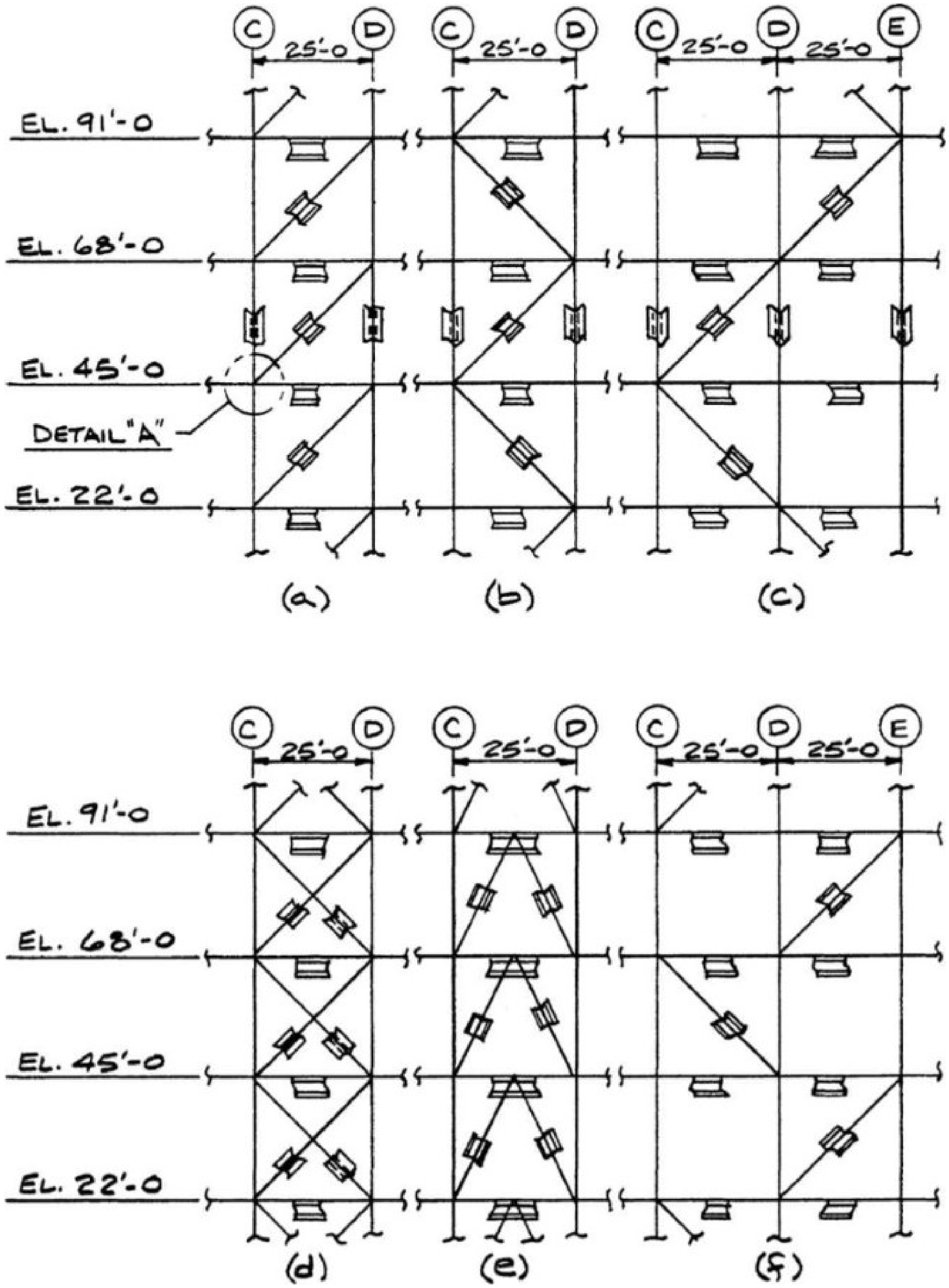


FIGURE 2.1 Various vertical bracing arrangements.

2.2.1.2 Example 1. Consider the bracing connection of Fig. 2.2. The brace load is 855 kips, the beam shear is 10 kips, and the beam axial force is 411 kips. The horizontal component of the brace force is 627 kips, which means that 627 – 411 = 216 kips is transferred to the opposite side of the column from the brace side. There must be a connection on this side to "pick up" this load, that is, provide a load path.

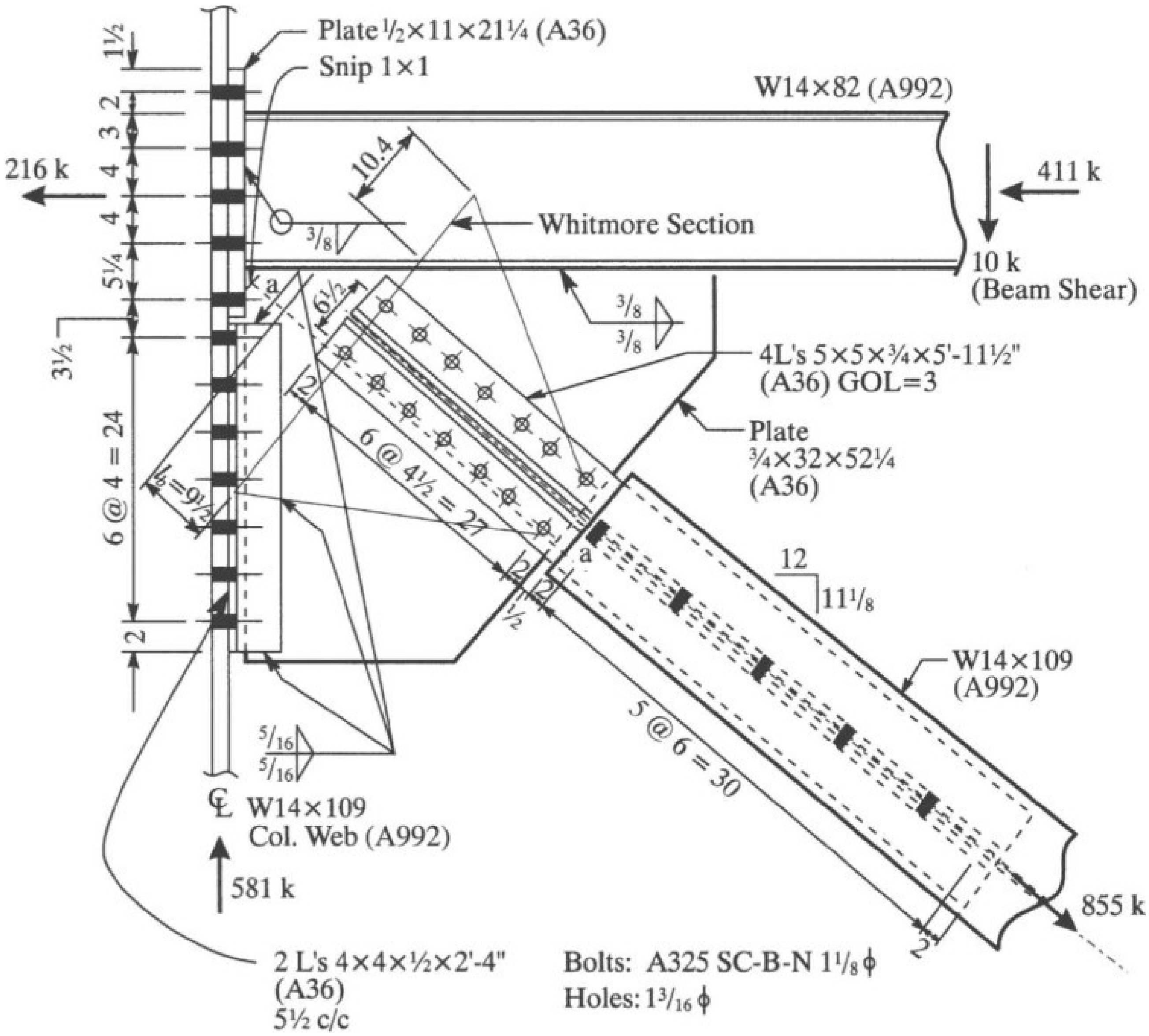


FIGURE 2.2 Example 1, bracing connection design.

The design of this connection involves the design of four separate connections. These are (1) the brace-to-gusset connection, (2) the gusset-to-column connection, (3) the gusset-to-beam connection, and (4) the beam-to-column connection. A fifth connection is the connection on the other side of the column, which will not be considered here.

1. *Brace-to-gusset*: This part of the connection is designed first because it provides a minimum size for the gusset plate which is then used to design the gusset-to-column and gusset-to-beam connections. Providing an adequate load path involves the following limit states:

   a. *Bolts (A325SC-B-N 1⅛-in-diameter 1-3/16 in holes (note that the 2016 Specification allows up to ⅛-in hole clearance for bolt greater than or equal to 1-in diameter), serviceability limit state)*: The above notation indicates that the bolts are slip critical, the surface class is B, and threads are not excluded from the shear planes. The slip-critical design strength per bolt is

$$\phi r_{str} = 1 \times 1.13 \times 0.5 \times 64 = 36.2 \text{ kips}$$

The specification requires that connections designed as slip critical must also be checked as bearing for the bearing condition. The bearing design strength per bolt is

$$\phi r_v = 0.75 \times \frac{\pi}{4} \times 1.125^2 \times 54 = 40.3 \text{ kips}$$

Since 36.2 < 40.3, use 36.2 kips as the design strength. The estimated number of bolts required is 855/(36.2 × 2) = 11.8. Therefore, try 12 bolts each side of the connection.

b. W14 × 109 *brace checks:*

(1) *Bolt shear, bearing, and tearout:* The proper check is one that considers bolt shear, bearing, and tearout for each bolt individually. The resistances of the individual bolts are then summed to determine a capacity for the bolt group.

The bolt shear strength has already been established as 36.2 kips per bolt.

The bearing strength per bolt is

$$\phi r_p = 0.75 \times 2.4 \times 1.125 \times 0.525 \times 65 = 69.1 \text{ kips}$$

The bolt tearout capacity of the edge bolts at the brace web is

$$\phi r_p = 0.75 \times 1.2 \times (2 - 0.594) \times 0.525 \times 65 = 43.1 \text{ kips}$$

Since tearout through the edge of the brace web is the critical condition and results in a capacity greater than the shear strength of the bolt, the full bearing capacity of the bolt can be developed. However, since the connection is to be designed as slip critical, the slip resistance will govern.

(2) *Block shear rupture:*

$$A_{gv} = (2 + 5 \times 6) \times 0.525 \times 2 = 33.6 \text{ in}^2$$

$$A_{nv} = 39.9 - 5.5 \times 1.25 \times 0.525 \times 2 = 26.4 \text{ in}^2$$

$$A_{nt} = 3.54 - 1 \times 1.25 \times 0.525 = 2.88 \text{ in}^2$$

Shear yielding = 39.9 × 0.6 × 50 = 1010 kips

Shear fracture = 31.4 × 0.6 × 65 = 1030 kips

Tension fracture = 2.88 × 65 = 187 kips

Since shear yielding is less than shear fracture, the failure mode is shear yielding and tension fracture; thus, the design block shear strength is

$$\phi R_{bs} = 0.75(1010 + 187) = 898 \text{ kips} > 855 \text{ kips, ok}$$

c. *Gusset checks:*

(1) *Bearing and tearout:* The bearing strength per bolt is

$$\phi r_p = 0.75 \times 2.4 \times 1.125 \times 0.75 \times 58 = 88.1 \text{ kips}$$

The bolt tearout capacity of the edge bolts at the gusset is

$$\phi r_p = 0.75 \times 1.2 \times (2 - 0.594) \times 0.75 \times 58 = 55.0 \text{ kips}$$

Again the bolt shear governs.

(2) *Block shear rupture:* These calculations are similar to those for the brace.

$$\begin{aligned}
A_{gv} &= 29.0 \times 0.75 \times 2 = 43.5 \text{ in}^2 \\
A_{nv} &= (29.0 - 6.5 \times 1.25) \times 0.75 \times 2 = 31.3 \text{ in}^2 \\
A_{nt} &= (6.5 - 1.0 \times 1.25) \times 0.75 = 3.94 \text{ in}^2 \\
\phi R_{bs} &= 0.75\ [F_u A_{nt} + \min\{0.6\ F_y A_{gv},\ 0.6\ F_u A_{nv}\}] \\
&= 0.75\ [58 \times 3.94 + \min\{0.6 \times 36 \times 43.5,\ 0.6 \times 58 \times 31.3\}] \\
&= 0.75\ [229 + \min\{940,\ 1090\}] \\
&= 876 \text{ kips} > 855 \text{ kips ok}
\end{aligned}$$

(3) *Whitmore section:* Since the brace load can be compression, this check is used to check for gusset buckling. Figure 2.2 shows the "Whitmore section" length, which is normally $l_w = (27 \tan 30) \times 2 + 6.5 = 37.7$ in, but the section passes out of the gusset and into the beam web at its upper side. Because of the fillet weld of the gusset to the beam flange, this part of the Whitmore section is not ineffective, that is, load can be passed through the weld to be carried on this part of the Whitmore section. The effective length of the Whitmore section is thus

$$l_{we} = (37.7 - 10.4) + 10.4 \times \frac{0.510}{0.75} \times \frac{50}{36} = 27.3 + 9.8 = 37.1 \text{ in}$$

The gusset buckling length is, from Fig. 2.1, $l_b = 9.5$ in, and the slenderness ratio is

$$\frac{Kl_b}{r} = \frac{0.5 \times 9.5 \times \sqrt{12}}{0.75} = 21.9$$

In this formula, the theoretical fixed-fixed factor of 0.5 is used rather than the usually recommended value of 0.65 for columns, because of the conservatism of this buckling check as determined by Gross (1990) from full-scale tests. From the AISC 2005 Specification Section J4.4, since $Kl_b/r \le 25$, the design buckling strength is

$$\phi F_{cr} = 0.9 \times 36 = 32.4 \text{ ksi}$$

and the Whitmore section buckling strength is thus

$$\phi R_{wb} = 32.4 \times 37.1 \times 0.75 = 902 \text{ kips} > 855 \text{ kips, ok}$$

The same result is achieved using the approach given by Dowswell (2006), where the required gusset thickness to prevent buckling is

$$t_{\beta} = 1.5\sqrt{\frac{F_y c^3}{El_1}} = 1.5\sqrt{\frac{36 \times 4.4^3}{29{,}000 \times 9.5}} = 0.158 < 0.75 \text{ yielding governs}$$

where c is the smaller of the distances from the connected edge of the gusset to the brace connection, and l_1 is the buckling length along the line of action of the brace.

d. *Brace-to-gusset connection angles*:

(1) *Gross and net area*: The gross area required is 855/(0.9 × 36) = 26.4 in^2

Try 4 *Ls* 5 × 5 × ¾, $A_{gt} = 6.94 \times 4 = 27.8$ in^2, ok

The net area is $A_{nt} = 27.8 - 4 \times 0.75 \times 1.25 = 24.1$ in^2

The effective net area is the lesser of 0.85 A_{gt} or UA_{nt},

where $U = 1 - \frac{1.51}{27} = 0.944$. Thus 0.85 A_{gt} = 0.85 × 27.8 = 23.6 and UA_{nt} = 0.944 × 24.1 = 22.8 and then A_e = 22.8. Therefore, the net tensile design strength is $\phi R_t = 0.75 \times 58 \times 22.8 = 992$ kips > 855 kips ok.

(2) *Bearing and tearout*: Comparing the strength of two ¾″ angles to the ¾″ gusset, it is clear that bolt bearing and tearout on the angles will not control.

(3) *Block shear rupture*: The length of the connection on the gusset side is the shorter of the two and is, therefore, the more critical. Per angle,

$$\begin{aligned} A_{gv} &= 29.0 \times 0.75 = 21.75 \text{ in}^2 \\ A_{nv} &= (29.0 - 6.5 \times 1.25) \times 0.75 = 15.66 \text{ in}^2 \\ A_{nt} &= (2.0 - 0.5 \times 1.25) \times 0.75 = 1.03 \text{ in}^2 \end{aligned}$$

$$\begin{aligned} \phi R_{bs} &= 0.75\ [F_u A_{nt} + \min\{0.6\ F_y A_{gv},\ 0.6\ F_u A_{nv}\}] \\ &= 0.75\ [58 \times 1.03 + \min\{0.6 \times 36 \times 21.75,\ 0.6 \times 58 \times 15.66\}] \times 4 \\ &= 1590 \text{ kips} > 855 \text{ kips ok} \end{aligned}$$

This completes the design checks for the brace-to-gusset connection. All elements of the load path, which consists of the bolts, the brace web, the gusset, and the connection angles, have been checked. The remaining connection interfaces require a method to determine the forces on them. Research (Thornton, 1991, 1995b) and practice (AISC, 2016) have shown that the best method for doing this is the uniform force method (UFM). The force distributions for this method are shown in Fig. 2.3.

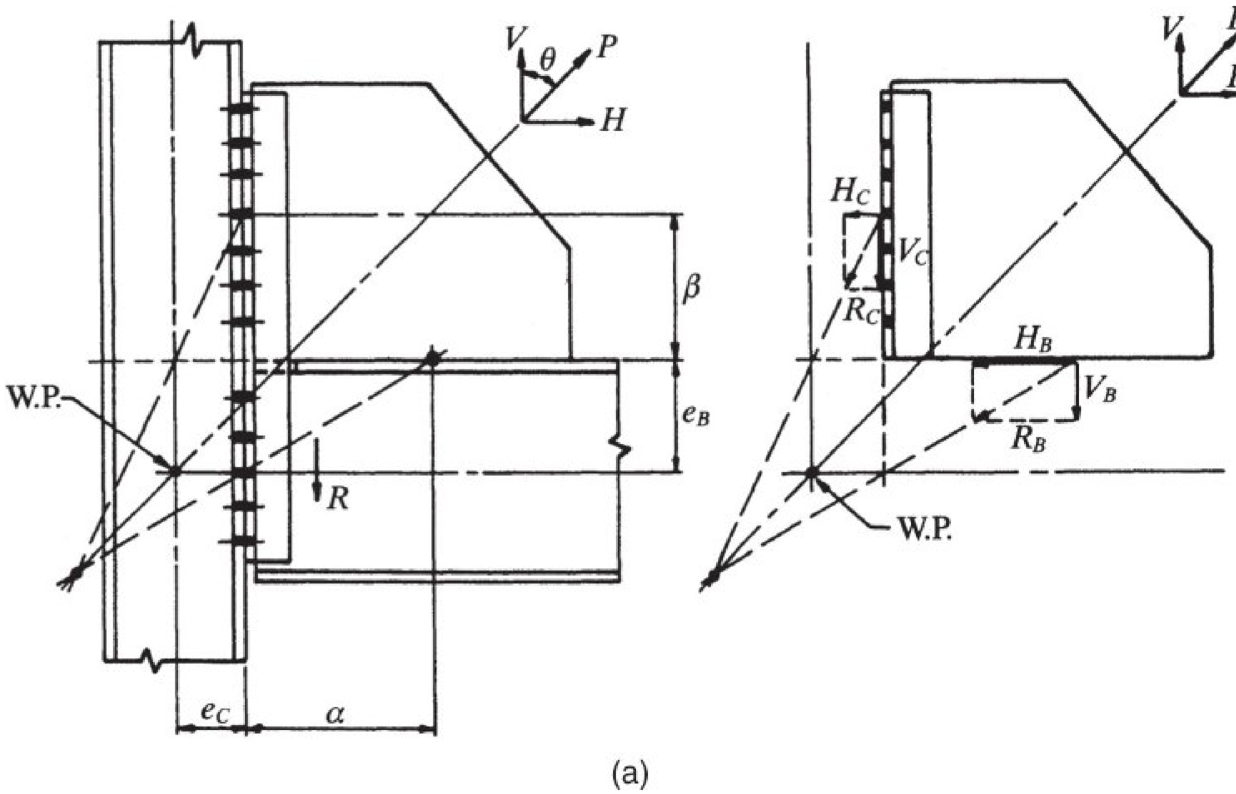


FIGURE 2.3*a* The uniform force method.

From the design of the brace-to-gusset connection, a certain minimum size of gusset is required. This is the gusset shown in Fig. 2.2. Usually, this gusset size, which is a preliminary size, is sufficient for the final design. From Fig. 2.2 and 2.3, the basic data are

$$\tan\theta = \frac{12}{11.125} = 1.08$$

$$e_B = \frac{14.3}{2} = 7.15$$

$$e_C = 0$$

The quantities α and β locate the centroids of the gusset edge connections, and in order for no couples to exist on these connections, α and β must satisfy the following relationship given in Fig. 2.3*b*,

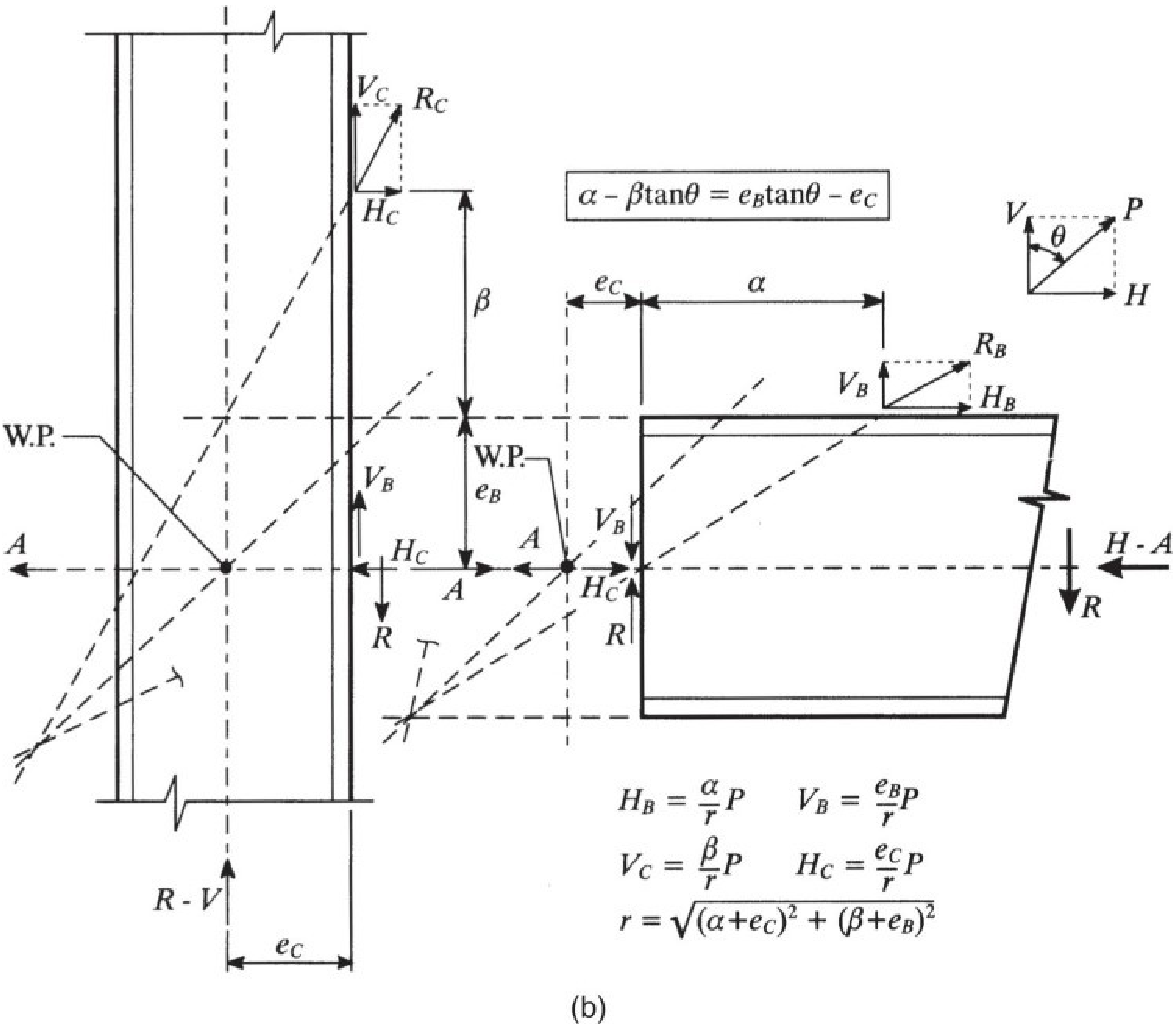


FIGURE 2.3*b* Force distribution for the uniform force method.

$$\alpha - \beta \tan \theta = e_B \tan\theta - e_C$$

Thus, $\alpha - 1.08\beta = 7.15 \times 1.08 - 0 = 7.72$.

From the geometry given in Fig. 2.2, a seven-row connection at 4-in pitch will give $\beta = 17.5$ in. Then $\alpha = 7.72 + 1.08 \times 17.5 = 26.6$ in and the horizontal length of the gusset is $(26.6 - 1) \times 2 = 51.2$ in. Choose a gusset length of 51¼ in.

With $\alpha = 26.6$ and $\beta = 17.5$,

$$r = \sqrt{(\alpha + e_C)^2 + (\beta + e_B)^2}$$

$$= \sqrt{(26.6 + 0)^2 + (17.5 + 7.15)^2} = 36.3\text{ in}$$

$$V_C = \frac{\beta}{r} P = \frac{17.5}{36.3} 855 = 413\text{ kips}$$

$$H_C = \frac{e_C}{r} P = \frac{0}{36.3} 855 = 0\text{ kip}$$

$$H_B = \frac{\alpha}{r} P = \frac{26.2}{36.3} 855 = 617\text{ kips}$$

$$V_B = \frac{e_B}{r} P = \frac{7.15}{36.3} 855 = 168\text{ kips}$$

2. *Gusset-to-column*: The loads are 412 kips shear and 0 kip axial.

a. *Bolts and clip angles:*

Bolts: A325SC-B-N 1⅛ ϕ; standard holes, serviceability criterion

Clip angles: try *Ls* 4 × 4 × ½

Shear per bolt is

$$V = 413/14 = 29.5\text{ kips} \le 36.2\text{ kips, ok}$$

The bearing strength of the clip angle is

$$\phi r_p = 0.75 \times 2.4 \times 58 \times 0.5 \times 1.125 = 58.7\text{ kips} > 36.2\text{ kips}$$

The bearing strength of the W14 × 109 column web is

$$\phi r_p = 0.75 \times 2.4 \times 65 \times 0.525 \times 1.125 = 69.1\text{ kips} > 36.2\text{ kips}$$

The bolt tearout capacity of the edge bolts at the clip angles is

$$\phi r_p = 0.75 \times 1.2 \times (2 - 0.594) \times 0.5 \times 58 = 36.7\text{ kips} > 36.2\text{ kips, ok}$$

There is no edge tearout condition at the column web, so it does not govern.

The net shear strength of the clips is

$$\phi R_n = 0.75 \times 0.6 \times 58\,(28 - 7 \times 1.25) \times 0.5 \times 2 = 502\text{ kips} > 412\text{ kips, ok}$$

The gross shear strength of the clips is

$$\phi R_n = 1.00 \times 0.6 \times 36 \times 28 \times 0.5 \times 2 = 605\text{ kips} > 412\text{ kips, ok}$$

Block shear on the clip angles

$$A_{gv} = 26 \times 0.5 = 13.0 \text{ in}^2$$
$$A_{nv} = (26 - 6.5 \times 1.25) \times 0.5 = 8.94 \text{ in}^2$$
$$A_{gt} = (1.5 - 0.5 \times 1.25) \times 0.5 = 0.438 \text{ in}^2$$
$$\phi R_{bs} = 0.75\ [0.438 \times 58 + \min\ \{0.6 \times 36 \times 13.0,\ 0.6 \times 58 \times 8.94\}] \times 2$$
$$= 459 \text{ kips} > 412 \text{ kips, ok}$$

b. *Fillet weld of clip angles to gusset*: The length of this clip angle weld is 28 in. From AISC 15th Edition Manual Table 8-8, $l = 28$, $kl = 3.0$, $k = 0.107$, $al = 4 - xl = 4 - 0.009 \times 28 = 3.75$, and $a = 0.134$. By interpolation, $c = 2.39$, and the required fillet weld size is $D = 412/(0.75 \times 2.39 \times 28 \times 2) = 4.11$, so the required fillet weld size is 5/16, and no proration is required because of the ¾-in-thick gusset. (See Table 1.9 in Chap. 1.)

3. *Gusset-to-beam*: The loads are 627 kips shear and 168 kips axial. The length of the gusset is 52.25 in. The 1-in snip can be ignored with negligible effect on the stress.

a. *Gusset stresses*:

$$f_v = \frac{627}{0.75 \times 51.25} = 16.3 \text{ ksi} < 1.0 \times 0.6 \times 36 = 21.6 \text{ ksi, ok}$$

$$f_a = \frac{168}{0.75 \times 51.25} = 4.37 \text{ ksi} < 0.9 \times 36 = 32.4 \text{ ksi, ok}$$

b. *Weld of gusset to beam bottom flange*: The resultant force per inch of weld is

$$f_r = \sqrt{16.3^2 + 4.37^2} \times \frac{0.75}{2} = 6.33 \text{ kips in}$$

To account for the directional strength increase on fillet welds

$$\phi = \tan^{-1}\left(\frac{4.37}{16.3}\right) = 15.0°$$

$$\mu = 1.0 + 0.5 \sin^{1.5}\phi = 1.0 + 0.5 \sin^{1.5}(15) = 1.07$$

The required weld size is

$$D = \frac{6.33}{1.392 \times 1.07} \times 1.25 = 5.31$$

which indicates that a ⅜-in fillet weld is required. The factor 1.25 is a ductility factor from the work of Richard (1986) as modified by Hewitt and Thornton (2004). Even though the stress in this weld is calculated as being uniform, it is well known that there will be local peak stresses, especially in the area where the brace-to-gusset connection comes close to the gusset-to-beam weld. An indication of high stress in this area is also indicated by the Whitmore section cutting into the beam web. Also, as discussed later, frame action will give rise to distortional forces that modify the force distribution given by the UFM.

c. *Checks on the beam web*: The 627-kip shear is passed into the beam through the gusset-to-beam weld. All of this load is ultimately distributed over the full cross-section of the W14 × 82, 411 kips passes to the right, and 216 kips are transferred across the column. The length of web required to transmit 627 kips of shear is l_{web}, where 627 = 1.0 × .6 × 50 × .510 × l_{web}. Thus

$$l_{\text{web}} = \frac{627}{1.0 \times 0.6 \times 50 \times 0.51} = 41.0\,\text{in}$$

which is reasonable. Note that this length can be longer than the gusset-to-beam weld, but probably should not exceed about half the beam span.

The vertical component can cause beam web yielding and crippling.

(1) *Web yielding*: The web yield design strength is

$$\phi R_{wy} = 1 \times 0.51 \times 50(51.25 + 2.5 \times 1.45) = 1400 \text{ kips} > 168 \text{ kips, ok}$$

(2) Web crippling: The web crippling design strength is

$$\phi R_{\text{wcp}} = 0.75 \times 0.8 \times t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_y t_f}{t_w}}$$

$$= 0.75 \times 0.8 \times 0.510^2 \left[1 + 3\left(\frac{51.25}{14.3}\right)\left(\frac{0.51}{0.855}\right)^{1.5}\right]$$

$$\times \sqrt{\frac{29{,}000 \times 50 \times 0.855}{0.51}} = 1450 \text{ kips} > 168 \text{ kips, ok}$$

The above two checks on the beam web seldom control but should be checked "just in case." The web crippling formula used is that for locations not near the beam end because the beam-to-column connection will effectively prevent crippling near the beam end. The physical situation is closer to that at some distance from the beam end rather than that at the beam end.

4. *Beam to column*: The loads are 216 kips axial, the specified transfer force and a shear which is the sum of the nominal minimum beam shear of 10 kips and the vertical force from the gusset-to-beam connection of 168 kips. Thus, the total shear is 10 + 168 = 178 kips.

a. *Bolts and end plate*: As established earlier in this example, the bolt design strength in shear is ϕr_{str} = 36.2 kips. In this connection, since the bolts also see a tensile load, there is an interaction between tension and shear that must be satisfied. If V is the factored shear per bolt, the design tensile strength is

$$\phi r_t' = 1.13 T_b \left(1 - \frac{V}{\phi r_{\text{str}}}\right) \leq 0.75 \times 90 A_b$$

This formula is obtained by inverting Specification formula J3-5a. T_b is the bolt pretension of 64 kips for A325 1⅛-in-diameter bolts and A_b is the bolt nominal area = $\pi/4 \times 1.125^2 = 0.994$ in^2.

For V = 179/10 = 17.9 kips < 36.2 kips, ok,

$$\phi r_t' = 1.13 \times 64(1 - 17.9/36.2) = 36.6 \text{ kips and } 0.75 \times 90 \times .994 = 67.1 \text{ kips.}$$

Thus $\phi r_t' = 36.6$ kips and $\phi R_t' = 10 \times 36.6 = 366$ kips > 216 kips, ok.

Section J3.8 of the Specification requires that slip critical connections must also be checked for bearing limit states, so the bearing interaction check is.

$$F_{nt}' = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \leq F_{nt}$$

$$= 1.3 \times 90 - \frac{90}{0.75 \times 54}\left(\frac{17.9}{0.994}\right) = 77.0 \text{ ksi}$$

$$\phi F_{nt}' = 0.75 \times 77.0 = 57.8 \text{ ksi} < 0.75 \times 90 = 67.5 \text{ ksi, ok}$$

$$\phi r_t' = 57.8 \times 0.994 = 57.5 \text{ ksi} > 36.6 \text{ kips, so bearing does not control.}$$

To determine the end plate thickness required, the critical dimension is the distance "b" from the face of the beam web to the center of the bolts. For 5½-in-cross centers, b = (5.5 – .5)/2 = 2.5 in. To make the bolts above and below the flanges approximately equally critical, they should be placed no more than 2½ in above and below the flanges. Figure 2.2 shows them placed at 2 in. Let the end plate be 11 in wide. Then a = (11 – 5.5)/2 = 2.75 < 1.25 × 2.5 = 3.125 ok. The edge distance at the top and bottom of the end plate is 1.5 in, which is more critical than 2.75 in, and will be used in the following calculations. The notation for a and b follows that of the AISC Manual as does the remainder of this procedure.

$$b' = b - \frac{d}{2} = 2.5 - \frac{1.125}{2} = 1.9375$$

$$a' = a + \frac{d}{2} = 1.5 + \frac{1.125}{2} = 2.0625$$

$$\rho = \frac{b'}{a'} = 0.94$$

$$\beta = \frac{1}{\rho}\left(\frac{\phi r_t'}{T} - 1\right)$$

where T = required tension per bolt = 21.6 kips.

$$\beta = \frac{1}{0.94}\left(\frac{57.5}{21.6} - 1\right) = 1.77$$

$$\alpha' = \min\left\{\frac{1}{\delta}\left(\frac{\beta}{1-\beta}\right), 1\right\}$$

where $\delta = 1 - d'/p = 1 - 1.1875/4 = 0.70$.

In the above expression, p is the tributary length of end plate per bolt. For the bolts adjacent to the beam web, this is obviously 4 in. For the bolts adjacent to the flanges, it is also approximately 4 in for p since at $b = 2.0$ in, a 45° spread from the center of the bolt gives $p = 4$ in. Note also that p cannot exceed one half of the width of the end plate.

$$\alpha' = 1.0, \text{ since } B > 1.0$$

The required end plate thickness is

$$t_{\text{req'd}} = \sqrt{\frac{4.44Tb'}{pF_u(1+\delta\alpha')}} = \sqrt{\frac{4.44 \times 21.6 \times 1.94}{4.0 \times 58(1 + 0.70 \times 1)}} = 0.472 \text{ in}$$

Use a ½-in end plate, 11 in wide and 14¼ + 2 + 2 + 1½ + 1½ = 21.25 in long.

b. *Weld of beam to end plate*: All of the shear of 179 kips exists in the beam web before it is transferred to the end plate by the weld of the beam to the end plate. The shear capacity of the beam web is

$$\phi R_v = 1.0 \times .6 \times 50 \times .510 \times 14.3 = 219 \text{ kips} > 178 \text{ kips, ok}$$

The weld to the end plate that carries this shear is the weld to the beam web plus the weld around to about the k_1 distance inside the beam profile and 2 k_1 on the outside of the flanges. This length is thus

$$2(d-2t_f)+4\left(k_1-\frac{t_w}{2}\right)+4k_1=2\times(14.31-2\times0.855)+4\times(1-0.510/2)+4\times1$$
$$=32.2\text{ in}$$

The force in this weld per inch due to shear is

$$f_v=\frac{178}{32.2}=5.53\text{ kips/in}$$

The length of weld that carries the axial force of 216 kips is the entire profile weld whose length is 4 × 10.13 – 2 × 0.51 + 2 × 14.3 = 68.0 in. The force in this weld per inch due to axial force is

$$f_a=\frac{216}{68.0}=3.18\text{ kips/in}$$

Also, where the bolts are close together, a “hot spot” stress should be checked. The most critical bolt in this regard is the one at the center of the W14 × 82. The axial force in the weld local to these bolts is

$$f_a'=\frac{2\times21.6}{8}=5.4\text{ kips/in}$$

The controlling resultant force in the weld is thus

$$f_R=\sqrt{5.53^2+5.40^2}=7.73\text{ kips/in}$$

To account for the directional strength increase on fillet welds

$$\theta=\tan^{-1}\left(\frac{5.40}{5.53}\right)=44.3°$$

$$\mu=1.0+0.5\sin^{1.5}(44.3)=1.29$$

The required weld size is

$$D=\frac{7.73}{1.392\times1.29}=4.30\text{ use a 5/16-in fillet weld}$$

As a final check, make sure that the beam web can deliver the axial force to the bolts. The tensile load for 2 bolts is 2 × 21.6 = 43.2 kips, and 4 in of the beam web must be capable of delivering this load, that is, providing a load path. The tensile capacity of 4 in of the beam web is 4 × 0.510 × 0.9 × 50 = 91.8 kips > 43.2 kips, ok.

2.2.1.3 Some Observations on the Design of Gusset Plates. It is a tenet of all gusset plate designs that it must be able to be shown that the stresses on any cut section of the gusset do not exceed the yield stresses on this section. Now, once the resultant forces on the gusset horizontal and vertical sections are calculated by the UFM, the resultant forces on any other cut section, such as section a-a of Fig. 2.2, are easy to calculate (see the appropriate free-body diagram incorporating this section, as shown in Fig. 2.4, where the resultant forces on section a-a are shown), but the determination of the stresses is not. The traditional approach to the determination of stresses, as mentioned in many books (Blodgett, 1966; Gaylord and Gaylord, 1972; Kulak et al., 1987) and papers (Whitmore, 1952; Vasarhelyi, 1971), is to use the formulas intended for long slender members, that is $f_a = P/A$ for axial stress, $f_b = Mc/I$ for bending stress, and $f_v = V/A$ for shear stress. It is well known that these are not correct for gusset plates (Timoshenko, 1970). They are recommended only because there is seemingly no alternative. Actually, the UFM, coupled with the Whitmore section and the block shear fracture limit state, is an alternative as will be shown subsequently.

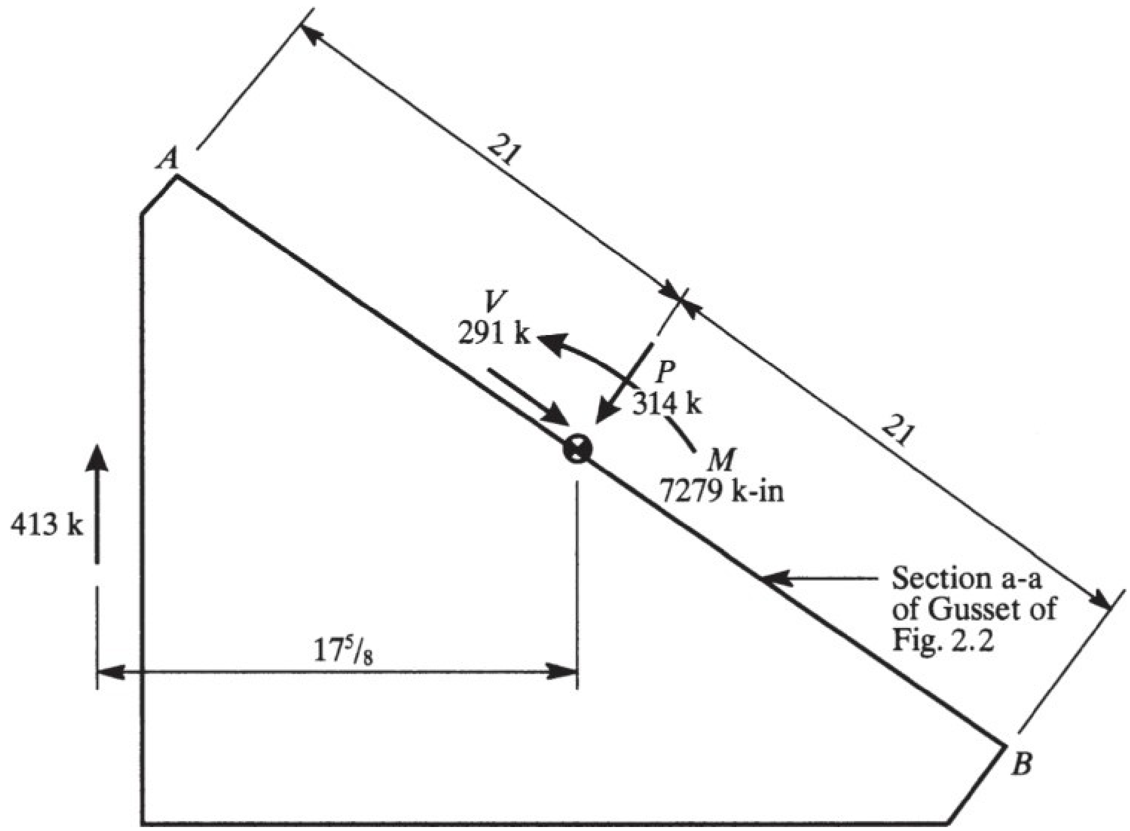


FIGURE 2.4 Free-body diagram of portion of gusset cut at section a-a of Fig. 2.2.

Applying the slender member formulas to the section and forces of Fig. 2.4, the stresses and stress distribution of Fig. 2.5 result. The stresses are calculated as

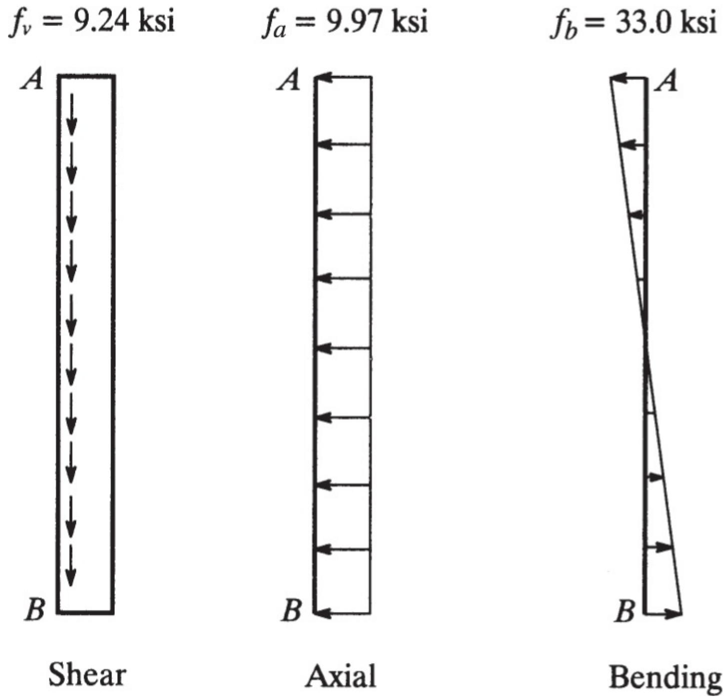


FIGURE 2.5 Traditional cut section stresses.

$$\text{shear:}\quad f_v = \frac{291}{0.75\times 42} = 9.24\text{ ksi}$$

$$\text{axial:}\quad f_a = \frac{314}{0.75\times 42^2} = 9.97\text{ ksi}$$

$$\text{bending:}\quad f_b = \frac{7280\times 6}{0.75\times 42^2} = 33.0\text{ ksi}$$

These are the basic "elastic"* stress distributions. The peak stress occurs at point A and is

$$\text{shear: } f_v = 9.24\text{ ksi}$$
$$\text{normal: } f_a + f_b = 9.97 + 33.0 = 43.0\text{ ksi}$$

The shear yield stress (design strength) is $\phi F_v = \phi(0.6\, F_y) = 1.0(0.6 \times 36) = 21.6$ ksi. Since 9.24 < 21.6, the section has not yielded in shear. The normal yield stress (design strength) is $\phi F_n = \phi F_y = 0.9\,(36) = 32.4$ ksi. Since 43.0 > 32.4, the yield strength has been exceeded at point A. At this point, it appears that the design is unsatisfactory (i.e., not meeting AISC requirements). But consider that the normal stress exceeds yield over only about 11 in of the 42-in-long section starting from point A. The remaining 42 – 11 = 31 in, have not yet yielded. This means that failure has not occurred because the elastic portion of the section will constrain unbounded yield deformations, that is, the deformation is "self-limited." Also, the stress of 43.0 ksi is totally artificial! It cannot be achieved in an elastic–perfectly plastic material with a design yield point of 32.4 ksi. What *will* happen is that when the design yield point of 32.4 ksi is reached, the stresses on the section will redistribute until the design yield point is reached at *every* point of the cross section. At this time, the plate will fail by unrestrained yielding if the applied loads are such that higher stresses are required for equilibrium.

To conclude on the basis of 43.0 ksi at point A, that the plate has failed is thus false. What must be done is to see if a redistributed stress state on the section can be achieved which nowhere exceeds the design yield stress. Note that if this can be achieved, all AISC requirements will have been satisfied. The AISC specifies that the design yield stress shall not be exceeded, but does *not* specify the formulas used to determine this.

The shear stress f_v and the axial stress f_a are already assumed uniform. Only the bending stress f_b is nonuniform. To achieve simultaneous yield over the entire section, the bending stress must be adjusted so that when combined with the axial stress, a uniform normal stress is achieved. To this end, consider Fig. 2.6. Here the bending stress is assumed uniform but of different magnitudes over the upper and lower parts of the section. Note that this can be done because **M** of Fig. 2.4, although shown at the centroid of the section, is actually a free vector that can be applied anywhere on the section or indeed anywhere on the free-body diagram. This being the case, there is no reason to assume that the bending stress distribution is symmetrical about the center of the section. Considering the distribution shown in Fig. 2.6, because the stress from A to the center is too high, the zero point of the distribution can be allowed to move down the amount e toward B. Equating the couple **M** of Fig. 2.4 to the statically equivalent stress distribution of Fig. 2.6 and taking moments about point D,

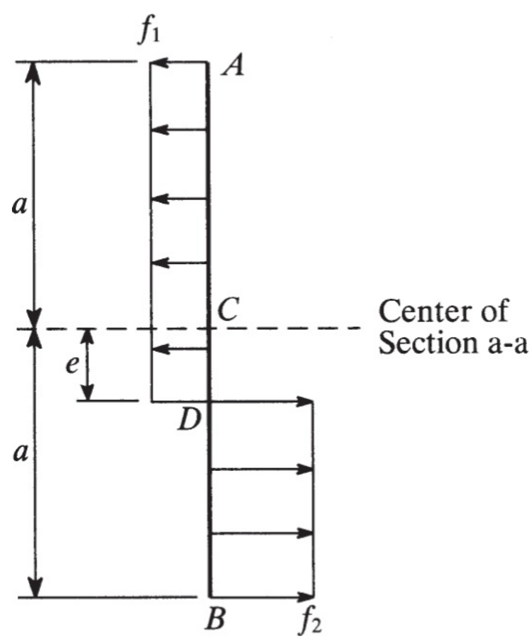


FIGURE 2.6 Admissible bending stress distribution of section a-a.

$$\mathbf{M} = \frac{t}{2}[f_1(a+e)^2 + f_2(a-e)^2]$$

where t is the gusset thickness. Also, from equilibrium

$$f_1\,(a+e)\,t = f_2\,(a-e)t$$

The above two equations permit a solution for f_1 and f_2 as

$$f_1 = \frac{\mathbf{M}}{at(a+e)}$$

$$f_2 = \frac{\mathbf{M}}{at(a-e)}$$

For a uniform distribution of normal stress,

$$f_1 + f_a = f_2 - f_a$$

from which e can be obtained as

$$e = \frac{1}{2}\left[\sqrt{\left(\frac{\mathbf{M}}{at\ f_a}\right)^2 + 4a^2} - \frac{\mathbf{M}}{at\ f_a}\right]$$

Substituting numerical values,

$$e = \frac{1}{2}\left[\sqrt{\left(\frac{7280}{(21)(0.75)(9.97)}\right)^2 + 4(21)^2} - \frac{7280}{(21)(0.75)(9.97)}\right] = 8.10\ \text{in}$$

Thus,

$$f_1 = \frac{7280}{(21)(0.75)(21+8.10)} = 15.9\ \text{ksi}$$

$$f_2 = \frac{7280}{(21)(0.75)(21-8.10)} = 35.8\ \text{ksi}$$

and the normal stress at point A is

$$f_{n_A} = f_1 + f_\text{a} = 15.9 + 9.97 = 25.9\ \text{ksi}$$

and at point B

$$f_{n_B} = f_2 - f_\text{a} = 35.8 - 9.97 = 25.9\ \text{ksi}$$

Now the entire section is uniformly stressed. Since

$$f_v = 9.24\ \text{ksi} < 21.6\ \text{ksi}$$
$$f_n = 25.9\ \text{ksi} < 32.4\ \text{ksi}$$

at all points of the section, the design yield stress is nowhere exceeded and the connection is satisfactory.

It was stated previously that there is an alternative to the use of the inappropriate slender beam formulas for the analysis and design of gusset plates. The preceding analysis of the special section a-a demonstrates the alternative that results in a true limit state (failure mode or mechanism) rather than the fictitious calculation of “hot spot” point stresses, which since their associated deformation is totally limited by the remaining elastic portions of the section, cannot correspond to a true failure mode or limit state. The UFM performs exactly the same analysis on the gusset horizontal and vertical edges, and on the associated beam-to-column connection. It is capable of producing forces on all interfaces that give rise to uniform stresses. Each interface is designed to just fail under these uniform stresses. Therefore, true limit states are achieved at every interface. For this reason, the UFM achieves a good approximation to the greatest lower bound solution (closest to the true collapse solution) in accordance with the lower bound theorem of limit analysis.

The UFM is a complete departure from the so-called traditional approach to gusset analysis using slender beam theory formulas. It has been validated against all known full-scale gusseted bracing connection tests (Thornton, 1991, 1995b). It does not require the checking of gusset sections such as that studied in this section (section a-a of Fig. 2.4). The analysis at this section was done to prove a point. But the UFM does include a check in the brace-to-gusset part of the calculation that is closely related to the special section a-a of Fig. 2.4. This is the block shear rupture of Fig. 2.7 (Hardash and Bjorhovde, 1985; Richard, 1983), which is included in section J4 of the AISC Specification (AISC, 2005). The block shear capacity was previously calculated as 877 kips.

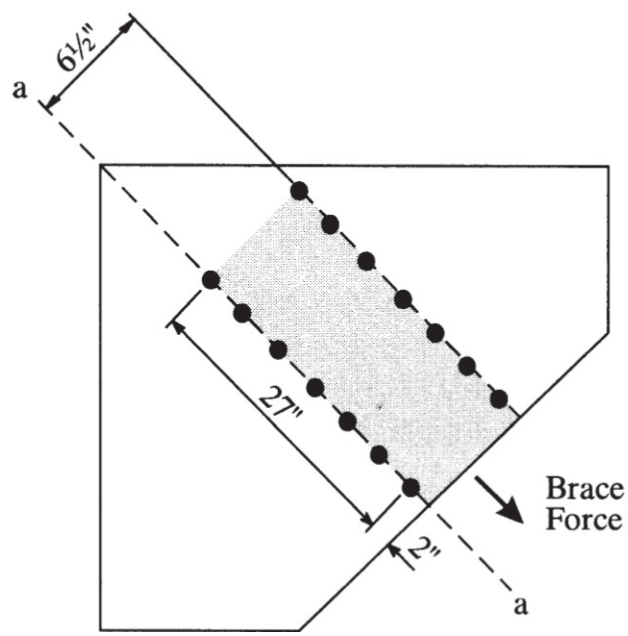


FIGURE 2.7 Block shear rupture and its relation to gusset section a-a.

Comparing the block shear limit state to the special section a-a limit state, a reserve capacity in block shear $= \frac{877-855}{855} 100 = 2.57\%$ is found, and the reserve capacity of the special section $= \frac{32.4-25.9}{25.9} 100 = 25.1\%$, which shows that block shear gives a conservative prediction of the capacity of the closely related special section.

A second check on the gusset performed as part of the UFM is the Whitmore section check. From the Whitmore section check performed earlier, the Whitmore area is

$$A_w = (37.7 - 10.4) \times 0.75 + 10.4 \times 0.510 \times \frac{50}{36} = 27.8\,\text{in}^2$$

and the Whitmore section design strength in tension is

$$\phi F_w = \phi(F_y \times A_w) = 0.9(36 \times 27.8) = 90\ 1\ \text{kips}$$

The reserve capacity of the Whitmore section in tension is $\frac{901-855}{855} \times 100 = 5.38\%$, which again gives a conservative prediction of capacity when compared to the special section a-a.

With these two limit states, block shear rupture and Whitmore, the special section limit state is closely bounded and rendered unnecessary. The routine calculations associated with block shear and Whitmore are sufficient in practice to eliminate the consideration of any sections other than the gusset-to-column and gusset-to-beam sections.

2.2.1.4 Example 2: Example Bracing Connection. This connection is shown in Fig. 2.8. The member on the right of the joint is a "collector" that adds load to the bracing truss. The brace consists of two MC12 × 45s with toes 1½ in apart. The gusset thickness is thus chosen to be 1½ in and is then checked. The completed design is shown in Fig. 2.8. In this case, because of the high specified beam shear of 170 kips, it is proposed to use a special case of the UFM which sets the vertical component of the load between the gusset and the beam, V_B, to zero. Figure 2.9 shows the resultant force distribution. This method is called "special case 2" of the UFM and is discussed in the AISC books (AISC, 1992, 1994).

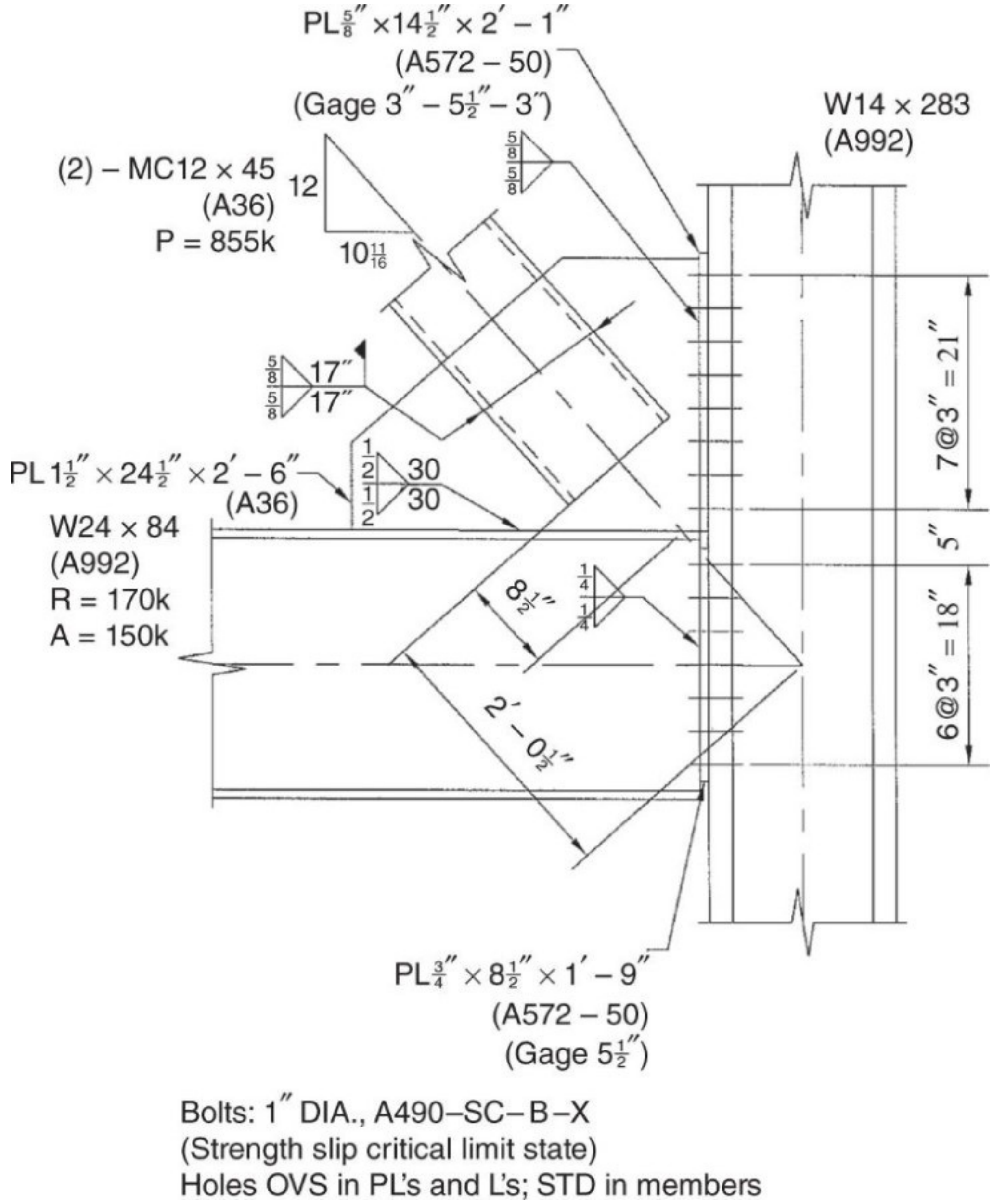


FIGURE 2.8 Example 2, bracing connection design.

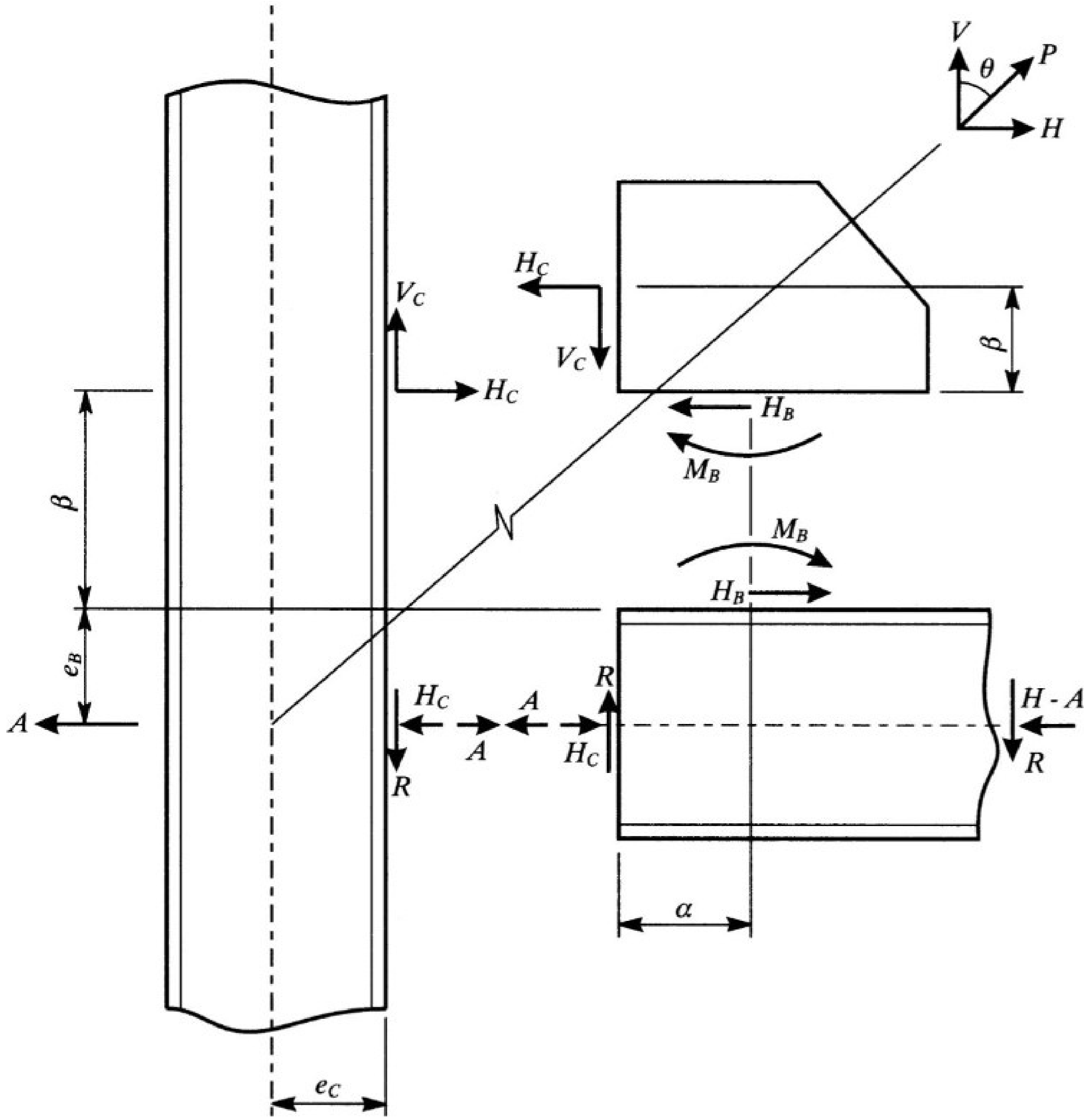


FIGURE 2.9 Force distribution for special case 2 of the uniform force method.

1. *Brace-to-gusset connection:*

   a. *Weld:* The brace is field welded to the gusset with fillet welds. Because of architectural constraints, the gusset size is to be kept to 31 in horizontally and 24½ in vertically. From the geometry of the gusset and brace, about 17 in of fillet weld can be accommodated. The weld size is

$$D = \frac{855}{4 \times 17 \times 1.392} = 9.03$$

   A ⅝-in fillet weld is indicated, but the flange of the MC12 × 45 must be checked to see if an adequate load path exists. The average thickness of 0.700 in occurs at the center of the flange, which is 4.012 in wide. The thickness at the toe of the flange, because of the usual inside flange slope of 2/12 or 16⅔%, is 0.700 – 2/12 × 2.006 = 0.366 in (see Fig. 2.10). The thickness at the toe of the fillet is 0.366 + 2/12 × 0.625 = 0.470 in. The design shear rupture strength of the MC12 flange at the toe of the fillet is

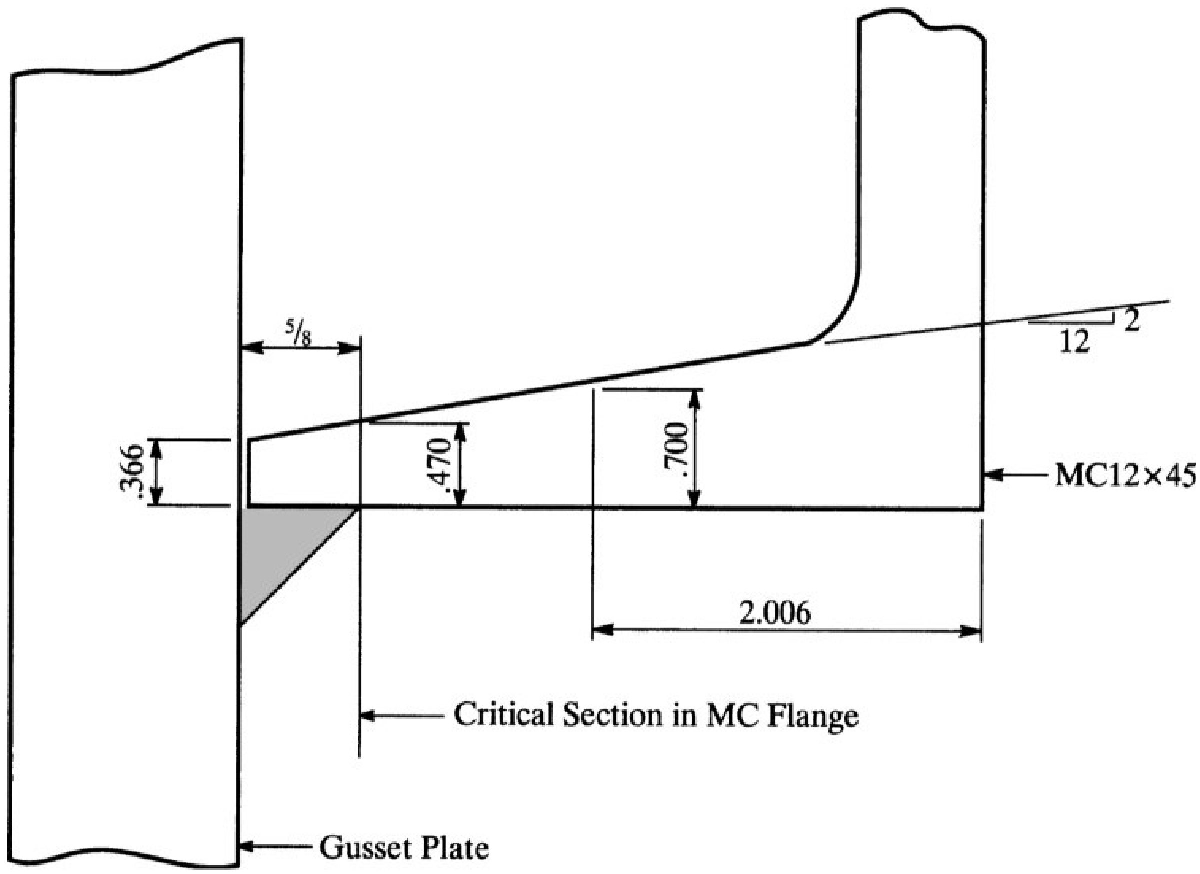


FIGURE 2.10 Critical section at toe of fillet weld.

$$\phi R_v = 0.75 \times 0.6 \times 58 \times 0.470 \times 17 \times 4 = 834 \text{ kips}$$

The design tensile rupture strength of the toe of the MC flange under the fillet is

$$\phi R_t = 0.75 \times 36 \left(\frac{0.366 + 0.470}{2} \right) 0.625 \times 4 = 28 \text{ kips}$$

Thus the total strength of the load path in the channel flange is 834 + 28 = 862 kips > 855 kips, ok.

b. *Gusset-to-brace block shear:*

shear yeilding:

$$\phi R_v = 0.90 \times 0.6 \times 36 \times 1.5 \times 17 \times 2 = 991 \text{ kips}$$

tension fracture

$$\phi R_t = 0.75 \times 58 \times 1.5 \times 12 = 783 \text{ kips}$$

$$\phi R_{bs} = 991 + 783 = 1770 \text{ kips} > 855 \text{ kips, ok}$$

c. *Whitmore section*: The theoretical length of the Whitmore section is (17 tan 30)2 + 12 = 31.6 in. The Whitmore section extends into the column by 5.40 in. The column web is stronger than the gusset since 1.29 × 50/36 = 1.79 > 1.5 in. The Whitmore also extends into the beam web by 6.80 in, but since 0.470 × 50/36 = 0.653 < 1.5 in, the beam web is not as strong as the gusset. The effective Whitmore section length is

$$l_{\text{weff}} = (31.6 - 6.80) + 6.80 \times \frac{0.470}{1.5} \times \frac{50}{36} = 27.8 \text{ in}$$

The effective length is based on $F_y = 36$ and the gusset thickness of 1.5 in.

Since the brace force can be tension or compression, compression will control. The slenderness ratio of the unsupported length of gusset is

$$\frac{Kl}{r} = \frac{0.5 \times 8.5\sqrt{12}}{1.5} = 9.82$$

The use of K = 0.5 comes from the work of Gross (1990).

Since $Kl/r < 25$

$$\phi F_a = 0.9F_y = 0.9 \times 36 = 32.4 \text{ ksi}$$

and the buckling strength of the gusset is

$$\phi R_{wb} = 27.8 \times 1.5 \times 32.4 = 1350 > 855 \text{ kips, ok}$$

The same result is achieved using the approach given by Dowswell (2006), where the required gusset thickness to prevent buckling is

$$t_\beta = 1.5\sqrt{\frac{F_y c^3}{El_1}} = 1.5\sqrt{\frac{36 \times 2.25^3}{29000 \times 8.5}} = 0.061 < 1.5 \text{ yielding governs}$$

where c is the smaller of the distances from the connected edge of the gusset to the brace connection, and l_1 is the buckling length along the line of action of the brace.

This completes the brace-to-gusset part of the design. Before proceeding, the distribution of forces to the gusset edges must be determined. From Fig. 2.8,

$$e_B = \frac{24.10}{2} = 12.05 \quad e_C = 8.37 \quad \bar{\beta} = 12.5 \quad \bar{\alpha} = 15.0$$

$$\theta = \tan^{-1}\left(\frac{10.6875}{12}\right) = 41.6°$$

$$V_C = P\cos\theta = 855 \times 0.747 = 638 \text{ kips}$$

$$H_C = \frac{V_C e_C}{e_B + \beta} = \frac{638 \times 8.37}{12.05 + 12.5} = 218 \text{ kips}$$

$$H_B = P\sin\theta - H_C = 855 \times 0.665 - 218 = 351 \text{ kips}$$

$$M_B = H_B e_B = 351 \times 12.05 = 4230 \text{ kips-in}$$

Note that, in this special case 2, the calculations can be simplified as shown here. The same results can be obtained formally with the UFM by setting $\beta = \bar{\beta} = 12.5$ and proceeding as follows. With tan θ = 0.8906,

$$\alpha - 0.8906\beta = 12.05 \times 0.8906 - 8.37 = 2.362$$

Setting $\beta = \bar{\beta} = 12.5$, α = 13.5. Since $\bar{\alpha}$ is approximately 15.0, there will be a couple, M_B, on the gusset-to-beam edge. Continuing

$$r = \sqrt{(13.5 + 8.37)^2 + (12.5 + 12.05)^2} = 32.9$$

$$\frac{P}{r} = \frac{855}{32.9} = 26.0$$

$$H_B = \frac{\alpha}{r} P = 351 \text{ kips}$$

$$H_C = \frac{e_C}{r} P = 218 \text{ kips}$$

$$V_B = \frac{e_B}{r} P = 313 \text{ kips}$$

$$V_C = \frac{\beta}{r} P = 325 \text{ kips}$$

$$M_B = |V_B(\alpha - \bar{\alpha})| = 470 \text{ kips-in}$$

This couple is clockwise on the gusset edge. Now, introducing special case 2, in the notation of the AISC *Manual of Steel Construction* (2015), set $\Delta V_B = V_B = 313$ kips. This reduces the vertical force between the gusset and beam to zero, and increases the gusset-to-column shear, V_C, to 313 + 325 = 638 kips and creates a counterclockwise couple on the gusset-to-beam edge of $\Delta V_B \bar{\alpha} = 313 \times 15.0 = 4700$ kips-in. The total couple on the gusset-to-beam edge is thus M_B = 4700 – 470 = 4230 kips-in. It can be seen that these gusset interface forces are the same as those obtained from the simpler method.

2. *Gusset-to-column connection*: The loads are 638 kips shear and 218 kips axial.

a. *Gusset stresses*:

$$f_v = \frac{638}{1.5 \times 24.5} = 17.4 \text{ ksi} < 1.0 \times 0.6 \times 36 = 21.6 \text{ ksi, ok}$$

$$f_a = \frac{2/8}{1.5 \times 24.5} = 5.93 \text{ ksi} < 0.9 \times 36 = 32.4 \text{ ksi, ok}$$

b. *Weld of gusset to end plate*: Using AISC LRFD, Table 8-4, $P_u = \sqrt{638^2 + 218^2} = 674$ kips and the angle from the longitudinal weld axis is $\tan^{-1}$ (218/638) = 18.9°, so using the table for 15° with k = a = 0.0, c = 3.84. Thus,

$$D = \frac{674}{0.75 \times 3.84 \times 24.5} = 9.55$$

which indicates that a ⅝ fillet is required. No ductility factor is used because the flexibility of the end plate will enable redistribution of nonuniform weld stresses.

(1) *Check bolt capacity*

The bolts are A490 SC-B-X in OVS holes. The slip-critical strength criterion is used because slip into bearing in this building could cause excessive P-Δ effects. Thus, from Table 7-3

$$\phi r_v = 18.4 \times 1.67 = 30.7 \text{ kips//bolt}$$

and from Table 7-2

$$\phi r_t = 66.6 \text{ kips/bolt}$$

(2) *Bolt shear*

$$\phi R_v = 30.7 \times 8 \times 4 = 982 \text{ kips} > 638 \text{ kips, ok}$$

(3) *Bolt tension*

Since only the two inside columns of bolts are effective in carrying the tension,

$$\phi R_t = 66.6 \times 8 \times 2 = 1070 \text{ kips} > 218 \text{ kips, ok}$$

(4) *Bolt shear/tension interaction*

The interaction equation for slip-critical bolts is given in Specification Section

J3.9 as,

$$\phi r_v' = \phi r_v \left(1 - \frac{T_u}{D_u T_b}\right)$$

where $\phi r_v'$ = reduced shear strength
T_u = tension load per bolt
D_u = 1.13
T_b = specified bolt pretension, 64 kips for 1-in A490 bolts

therefore,

$$\phi r_v' = 30.7\left(1 - \frac{218/16}{1.13 \times 64}\right) = 24.9 \text{ kips/bolt} > \frac{638}{32}$$

$$= 20.0 \text{ kips/bolt, ok}$$

(5) *End plate thickness required and prying action*

In previous editions of this handbook the interaction equation above was rearranged to produce:

In spite of its mathematical relationship the rearrangement does not accurately represent the physical behavior of slip-critical connections. The Specification Equation J3-5a is written in terms of a reduced shear stress is as follows: while T_u affects slip-critical connection shear strength per bolt, the applied shear, V_u, does not affect the tensile strength of the bolt in quite the same manner. The reason for this lies in the physical behavior of slip-critical connections. Connection shear, V_u, is carried by the faying surface through friction—rather than by the bolt shank—until slip occurs. Thus, the bolt itself "sees" no shear until the connection slips, and its tensile strength is consequently unaffected until slip. Once slip occurs, bearing interaction Equation J3-3a from the Specification and the prying action model as shown in the Manual must be used (Thornton, 2012).

In order to demonstrate the effect on the final design, the previous method will be presented and then the more appropriate model will be used:

$$\phi r_t' = 1.13 \times 64\left(1 - \frac{20.0}{30.7}\right) = 25.2 \text{ kips/bolt} > 13.6 \text{ kips/bolt, ok}$$

Try a ⅝-in-thick end plate of A572-Grade 50 steel. Following the notation of the Manual.

$$b = \frac{5.5 - 1.5}{2} = 2 \text{ in} \qquad a = \frac{14.5 - 5.5}{2} = 4.5 \text{ in}$$

Check $a \leq 1.25b = 1.25 \times 2.00 = 2.50$. Therefore, use $a = 2.50$ in.

In this problem, "*a*" should not be taken as larger than the bolt gage of 3 in.

$$b' = 2.00 - \frac{1.00}{2} = 1.5 \text{ in} \qquad a' = 2.50 + \frac{1.00}{2} = 3.00 \text{ in}$$

$$\rho = \frac{b'}{a'} = \frac{1.50}{3.00} = 0.50$$

$$\delta = 1 - \frac{d'}{p} = 1 - \frac{1.25}{3.00} = 0.583$$

$$B = \phi r_t' = 25.2$$

$$t_c = \sqrt{\frac{4.44Bb'}{pF_u}} = \sqrt{\frac{4.44 \times 25.2 \times 1.5}{3 \times 65}} = 0.928 \text{ in}$$

$$\alpha' = \frac{1}{\delta(1+p)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] = \frac{1}{0.583 \times 1.50}\left[\left(\frac{0.928}{0.625}\right)^2 - 1\right] = 1.38$$

$$Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta\alpha') = \left(\frac{0.625}{0.928}\right)^2 (1 + 0.583) = 0.718$$

$$T_{\text{avail}} = QB = 0.718 \times 25.2 = 18.0 \text{ kips/bolt } > 13.6 \text{ kips/bolt} \therefore \text{ bolts}$$

Now determine the available tensile strength of the bolt, considering the effects of the applied shear, based on the bearing strength:

$$F_{nt}' = 1.3F_{nt} - \frac{F_{nt}}{\varphi F_{nv}} f_{nv} \le F_{nt}$$

$$F_{nt}' = 1.3 \times 113 - \frac{113}{0.75 \times 84} \times 25.5 \le 113$$

$$F_{nt}' = 101 \text{ ksi} \le 113 \text{ ksi}$$

Therefore,

$$\varphi r_t' = 0.75 \times 101 \times 0.5^2 \times \pi = 59.5 \text{ kips/bolt}$$

$$t_c = \sqrt{\frac{4.44Bb'}{pF_u}} = \sqrt{\frac{4.44 \times 59.5 \times 1.5}{3 \times 65}} = 1.43$$

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] = \frac{1}{0.583 \times 1.5}\left[\left(\frac{1.43}{0.625}\right)^2 - 1\right] = 4.84$$

$$Q = \left(\frac{t}{t_c}\right)^2 (1+\delta) = \left(\frac{0.625}{1.43}\right)^2 (1+0.583) = 0.302$$

$$T_{\text{avail}} = QB = 0.302 \times 59.5 = 18.0 > 13.6 \text{ kips/bolt}$$

Use ⅝-in end plate.

In this case bending in the plate governs. This is indicated both by the fact that both methods produce the same result and by the fact that α' is greater than 1.

Use ¾-in end plate.

Check clearance

From Table 7-16

$$C_3 = 1 \text{ in} < \frac{5.5 - 1.5}{2} - 0.625 = 1.375 \text{ in} \therefore \text{ok}$$

(6) *Check column flange prying*

Since $t_f = 2.07$ in and $t_w = 1.29$ in, it is obvious that this limit state will not govern. In addition to the prying check, the end plate should be checked for gross shear, net shear, and block shear. These will not govern in this case.

c. *Checks on column web:*

(1) *Web yielding (under normal load H_c)*:

$$\phi R_{wy} = 1.0 \times 50 \times 1.290\left(24.5 + 5 \times 2\frac{3}{4}\right)$$

$$= 2470 \text{ kips} > 218 \text{ kips, ok}$$

(2) *Web crippling (under normal load H_c)*:

$$\phi R_{wcp} = 0.75 \times 0.8 \times 1.29^2\left[1 + 3\left(\frac{24.5}{16.7}\right)\left(\frac{1.29}{2.07}\right)^{1.5}\right]$$

$$\times \sqrt{\frac{29{,}000 \times 50 \times 2.07}{1.29}} = 4820 \text{ kips} > 218 \text{ kips, ok}$$

(3) *Web shear:* The horizontal force, H_c, is transferred to the column by the gusset-to-column connection and back into the beam by the beam-to-column connection. Thus, the column web sees H_e = 218 kips as a shear. The column shear capacity is

$$\phi R_v = 1.0 \times 0.6 \times 50 \times 1.29 \times 16.7 = 646 \text{ kips} > 218 \text{ kips, ok}$$

3. *Gusset-to-beam connection*: The loads are 351 kips shear and a 4230-kips-in couple.

a. *Gusset stresses*:

$$f_v = \frac{351}{1.5 \times 30} = 7.80 \text{ ksi} < 21.6 \text{ ksi, ok}$$

$$f_b = \frac{4230 \times 4}{1.5 \times 30^2} = 12.5 \text{ ksi} < 32.4 \text{ ksi, ok}$$

b. *Weld of gusset-to-beam flange*:

$$f_{\text{peak}} = \sqrt{7.80^2 + 12.5^2} \times \frac{1.5}{2} = 11.0 \text{ kips/in}$$

$$\theta_w = \tan^{-1}\left(\frac{12.5}{7.80}\right) = 58.0°$$

$$f_{\text{ave}} = \frac{1}{2}[\sqrt{7.80^2 + 12.5^2} + \sqrt{7.80^2 + 12.5^2}]\frac{1.5}{2} = 11.0 \text{ kips/in}$$

Since 11.0/11.0 = 1.0 < 1.25, the weld size based on the average force in the weld, $f_{\text{ave}} \times$ 1.25, therefore

$$D = \frac{11.0 \times 1.25}{1.392(1 + 0.5 \sin^{1.5}(58.2))} = 7.09$$

A ½ fillet weld is indicated. The 1.25 is the ductility factor; see Hewitt and Thornton (2004).

An alternate method for calculating the weld size required is to use Table 8-38 of the AISC *Manual of Steel Construction* (2005), special case k = 0, P_u = 349, and al = 4205/349 = 12.05 in; thus a = 12.05/30 = 0.40 and c = 2.00, and the required weld size is

$$D = \frac{349}{2.0 \times 30} = 5.8$$

A ⅜ fillet is indicated. This method does not give an indication of peak and average stresses, but it will be safe to use the ductility factor. Thus, the required weld size would be

$$D = 5.8 \times 1.25 = 7.25$$

Thus, by either method, a ½ fillet is indicated.

c. *Checks on beam web:*

(1) *Web yield:* Although there is no axial component, the couple M_B = 4230 kips-in is statically equivalent to equal and opposite vertical shears at a lever arm of one-half the gusset length or 15 in. The shear is thus

$$V_s = \frac{4230}{15} = 282 \text{ kips}$$

This shear is applied to the flange as a transverse load over 15 in of flange. It is convenient for analysis purposes to imagine this load doubled and applied over the contact length N = 30 in. The design web yielding strength is

$$\phi R_{wy} = 1.0 \times 50 \times 0.47(30 + 2.5 \times 1.27) = 780 \text{ kips} > 282 \times 2$$
$$= 564 \text{ kips, ok}$$

(2) *Web crippling:*

$$\phi R_{\text{wcp}} = 0.75 \times 0.8 \times 0.47^2 \left[1 + 3\left(\frac{30}{24.1}\right)\left(\frac{0.47}{0.77}\right)^{1.5}\right]$$
$$\times \sqrt{\frac{29{,}000 \times 50 \times 0.77}{0.47}} = 568 \text{ kips} > 564 \text{ kips, ok}$$

(3) *Web shear:*

$$\phi P_v = 1.0 \times 0.6 \times 50 \times 0.47 \times 24.1 = 340 \text{ kips} > 282 \text{ kips, ok}$$

The maximum shear due to the couple is centered on the gusset 15 in from the beam end. It does not reach the beam-to-column connection where the beam shear is 170 kips. Because of the total vertical shear capacity of the beam and the gusset acting together, there is no need to check the beam web for a combined shear of V_s and R of 282 + 170 = 452 kips.

4. *Beam-to-column connection*: The shear load is 170 kips and the axial force is H_c +/− A = 218 +/− 150 kips. Since the W18 × 50 is a collector, it adds load to the bracing system. Thus, the axial load is 218 + 150 = 368 kips. However, the AISC book on connections (AISC, 1992) addresses this situation and states that because of frame action (distortion), which will always tend to reduce H_c, it is reasonable to use the larger of H_c and A as the axial force. Thus the axial load would be 218 kips in this case. It should be noted however that when the brace load is not due to primarily lateral loads frame action might not occur.

a. *Bolts and end plate:* Though loads caused by wind and seismic forces are not considered cyclic (fatigue) loads and bolts in tension are not required to be designed as slip critical, the bolts are specified to be designed as A490 SC-B-X 1-in diameter to accommodate the use of oversize 1¼-in-diameter holes. As mentioned earlier the slip-critical strength criterion in used. Thus, for shear

$$\phi r_v = 30.7 \text{ kips/bolt}$$

and for tension

$$\phi r_t = 66.6 \text{ kips/bolt}$$

The end plate is ¾ in thick with seven rows and 2 columns of bolts. Note that the end plate is 14½ in wide for the gusset to column connection and 8½ in wide for the beam-to-column connection.

For shear

$$\phi R_v = 30.7 \times 14 = 430 \text{ kips} > 170 \text{ kips, ok}$$

For tension

$$\phi R_t = 66.6 \times 14 = 932 \text{ kips} > 218 \text{ kips, ok}$$

For tension, the bolts and end plate are checked together for prying action.

Since all of the bolts are subjected to tension simultaneously, there is interaction between tension and shear. The reduced tensile capacity is

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\varphi F_{nv}} f_{rv} = 1.3 \times 113 - \frac{113}{0.75 \times 84}\left(\frac{170}{14 \times \pi \times 0.625^2}\right) = 119, \text{ use } 13 \text{ ksi}$$

$$B = 66.6 \text{ kips/bolt}$$

Prying action is now checked using the method and notation of the AISC *Manual of Steel Construction* (2005), pages 9-10 through 9-13:

$$b = \frac{5.5 - 0.47}{2} = 2.52$$

$$a = \frac{8.5 - 5.5}{2} = 1.50$$

Check 1.25b = 1.25 × 2.52 = 3.15. Since 3.15 > 1.50, use a = 1.50.

$$b' = 2.52 - \frac{1.0}{2} = 2.02$$

$$a' = 1.50 + \frac{1.0}{2} = 2.00$$

$$\rho = 1.01$$

$$\delta = 1 - \frac{1.25}{3} = 0.583$$

$$t_c = \sqrt{\frac{4.44 \times 66.6 \times 2.02}{3 \times 58}} = 1.85$$

$$\alpha' = \frac{1}{0.583 \times 2.01}\left[\left(\frac{1.85}{0.75}\right)^2 - 1\right] = 4.34$$

Use $\alpha' = 1.00$

The design strength per bolt including prying is

$$\phi T = 66.7\left(\frac{0.75}{1.85}\right)^2 (1 + 0.583 \times 1.00) = 17.3 \text{ kips is about equal to } 17.4 \text{ kips, say ok}$$

In addition to the prying check, the end plate should also be checked for gross shear net shear and block shear. These will not control in this case.

b. *Weld of end plate to beam web:* The weld is a double line weld with length l = 21 in, k = a = 0. From the AISC *Manual of Steel Construction* (2005), Table 8-4. Since $\tan^{-1}$ 220/170 = 52.3°, use the chart for 45°. With C = 4.64 a ¼ fillet weld has a capacity of ϕR_w = 0.75 × 4.64 × 4 × 21 = 292 kips. Thus, since 292 kips $\sqrt{218^2 + 170^2} = 278$ kips, the ¼ fillet weld is ok.

c. *Bending of the column flange*: As was the case for the gusset to column connection, since t_f = 2.07 in is much greater than the end plate thickness of ¾ in, the check can be ignored. The following method can be used when t_f and t_p are of similar thicknesses. Because of the axial force, the column flange can bend just as the clip angles. A yield-line analysis derived from Mann and Morris (1979) can be used to determine an effective tributary length of column flange per bolt. The yield lines are shown in Fig. 2.11. From Fig. 2.11,

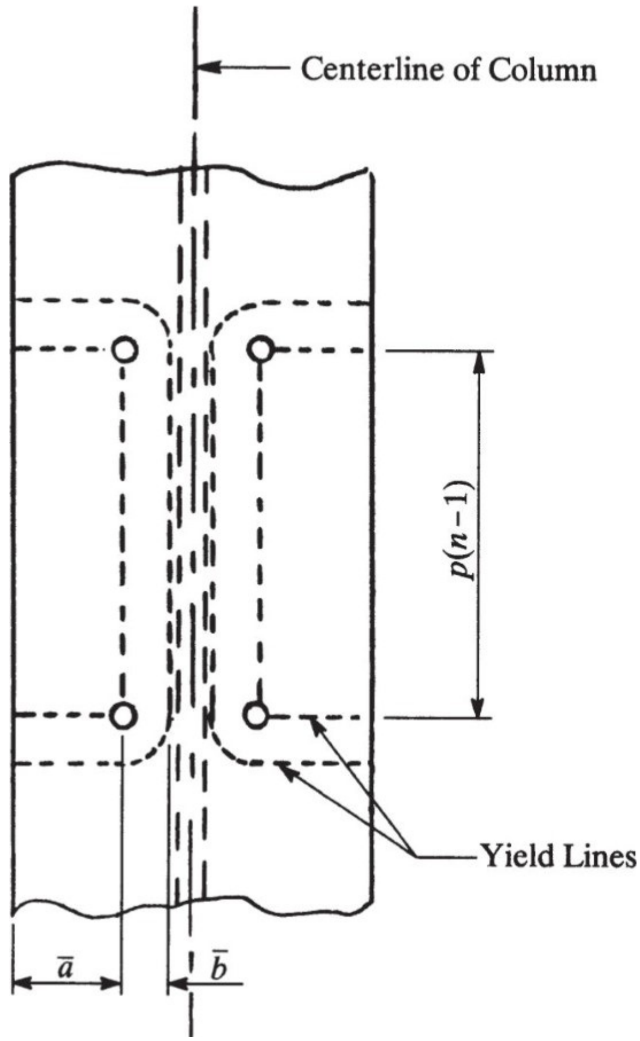


FIGURE 2.11 Yield lines for flange bending.

$$p_{\text{eff}} = \frac{(n-1)p + \pi\bar{b} + 2\bar{a}}{n}$$

where $\bar{b} = (5.5 - 1.29)/2 = 2.11$
$\bar{a} = (16.1 - 5.5)/2 = 5.31$
$p = 3$
$n = 7$

Thus,

$$p_{\text{eff}} = \frac{6\times 3 + \pi\times 2.11 + 2\times 5.31}{7} = 5.03$$

Using p_{eff} in place of p, and following the AISC procedure,

$$b = \bar{b} = 2.11$$

$$b' = 2.11 - \frac{1.0}{2} = 1.61$$

$$a = \min\left(\frac{4+4+0.47-5.5}{2}, 5.31, 1.25 \times 2.11\right)$$

$$= \min(1.48, 5.31, 2.63) = 1.49$$

$$a' = 1.49 + 0.5 = 1.99$$

$$p = \frac{b'}{a'} = 0.81$$

$$\delta = 1 - \frac{1.06}{5.03} = 0.79$$

Note that standard holes are used in the column flange.

$$t_c = \sqrt{\frac{4.44 \times 66.6 \times 1.61}{5.03 \times 65}} = 1.2$$

$$\alpha' = \frac{1}{0.79 \times 1.81}\left[\left(\frac{1.21}{2.07}\right)^2 - 1\right] = -0.460$$

Since $\alpha' < 0$, use $\alpha' = 0$

$$\phi T = 66.6 \text{ kips/bolt} > 15.7 \text{ kips/bolt, ok}$$

When $\alpha' < 1$, the bolts, and not the flange, control the strength of the connection.

2.2.1.5 Frame Action. The method of bracing connection design presented here, the uniform force method (UFM), is an equilibrium-based method. Every proper method of design for bracing connections, and in fact for every type of connection, must satisfy equilibrium. The set of forces derived from the UFM, as shown in Fig. 2.3, satisfy equilibrium of the gusset, the column, and the beam with axial forces only. Such a set of forces is said to be “admissible.” But equilibrium is not the only requirement that must be satisfied to establish the true distribution of forces in a structure or connection. Two additional requirements are the constitutive equations that relate forces to deformations and the compatibility equations that relate deformations to displacements.

If it is assumed that the structure and connection behave elastically (an assumption as to constitutive equations) and that the beam and the column remain perpendicular to each other (an assumption as to deformation–displacement equations), then an estimate of the moment in the beam due to distortion of the frame (frame action) (Thornton, 1991) is given by

$$M_D = 6\frac{P}{Abc}\frac{I_b I_c}{\left(\frac{I_b}{b}+\frac{2I_c}{c}\right)}\frac{(b^2+c^2)}{bc}$$

where D = distortion
I_b = moment of inertia of beam = 2370 in^4
I_c = moment of inertia of column = 3840 in^4
P = brace force = 855 kips
A = brace area = 26.4 in^2
b = length of beam to inflection point (assumed at beam midpoint) = 175 in
c = length of columns to inflection points (assumed at column midlengths) = 96 in

With $\frac{2I_c}{c}=80$ and $\frac{I_b}{b}=13.5$

$$M_D = \frac{6\times 855\times 2370\times 3840}{26.4\times 175\times 96\times(13.5+80)}\frac{175^2+96^2}{(175\times 96)} = 2670 \text{ kips-in}$$

This moment M_D is only an estimate of the actual moment that will exist between the beam and column. The actual moment will depend on the strength of the beam-to-column connection. The strength of the beam-to-column connection can be assessed by considering the forces induced in the connection by the moment M_D as shown in Fig. 2.12. The distortional force F_D is assumed to act as shown through the gusset edge connection centroids. If the brace force P is a tension, the angle between the beam and column tends to decrease, compressing the gusset between them, so F_D is a compression. If the brace force P is a compression, the angle between the beam and column tends to increase and F_D is a tension. Figure 2.12 shows how the distortional force F_D is distributed throughout the connection. From Fig. 2.12, the following relationships exist between F_D, its components H_D and V_D, and M_D:

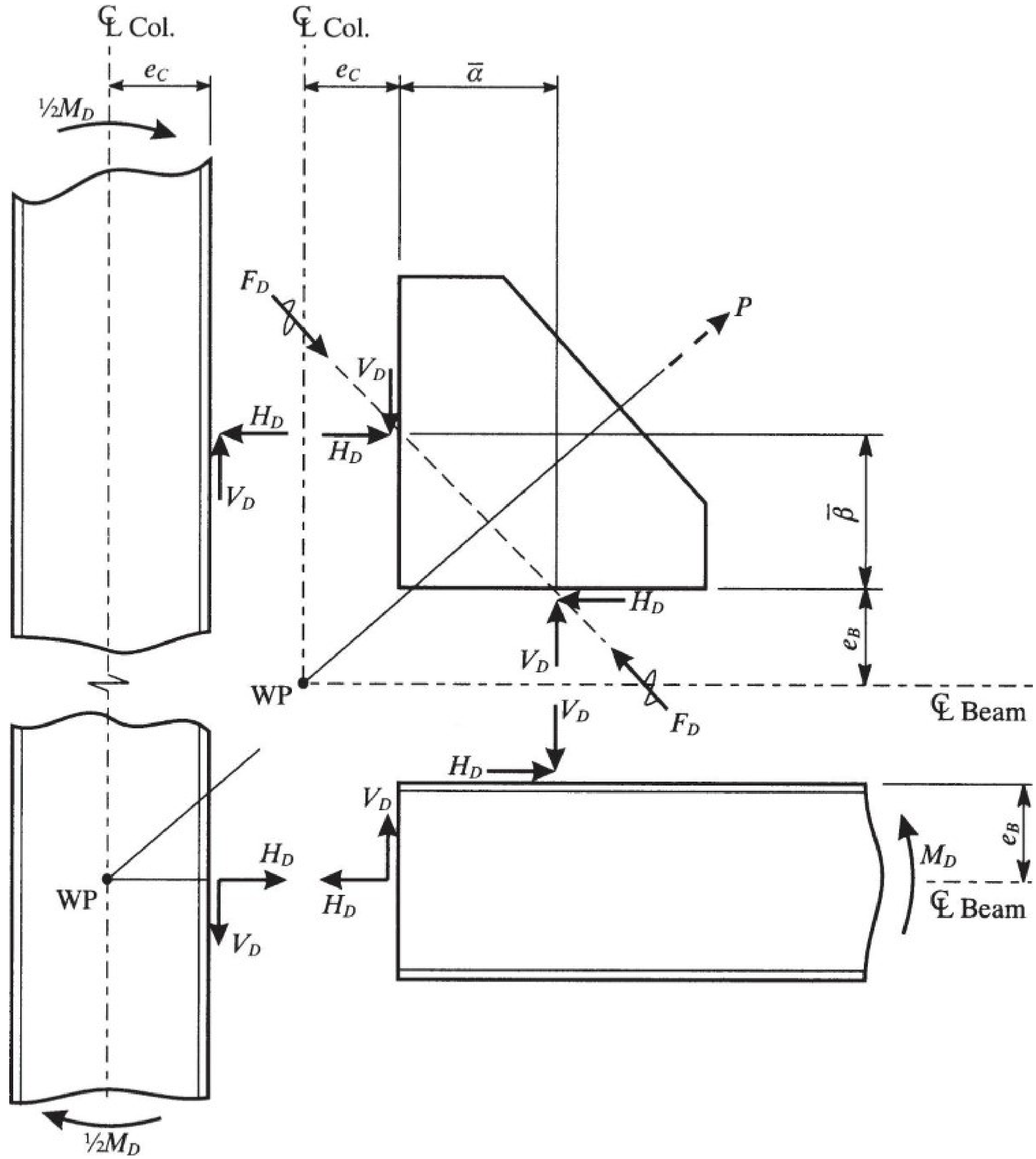


FIGURE 2.12 Distribution of distortion forces.

$$F_D = \sqrt{H_D^2 + V_D^2}$$

$$\overline{\beta} H_D = \overline{\alpha} V_D$$

$$H_D = \frac{M_D}{(\overline{\beta} + e_B)}$$

For the elastic case with no angular distortion

$$H_D = \frac{2670}{(12.25 + 12.05)} = 110 \text{ kips}$$

$$V_D = \frac{\overline{\beta}}{\overline{\alpha}} H_D = \frac{12.25}{15} \times 110 = 89.8 \text{ kips}$$

It should be remembered that these are just estimates of the distortional forces. The actual distortional forces will be dependent also upon the strength of the connection. But it can be seen that these estimated distortional forces are not insignificant. Compare, for instance, H_D to H_c. H_c is 218 kips tension when H_D is 110 kips compression. The net axial design force would then be 218 – 110 = 108 kips rather than 218 kips.

The strength of the connection can be determined by considering the strength of each interface, including the effects of the distortional forces. The following interface forces can be determined from Figs. 2.3 and 2.12.

For the gusset-to-beam interface:

$$T_B(\text{tangential force}) = H_B + H_D$$
$$N_B(\text{normal force}) = V_B - V_D$$

For the gusset-to-column interface:

$$T_C = V_C + V_D$$
$$N_C = H_c - H_D$$

For the beam-to-column interface:

$$T_{BC} = |V_B - V_D| + R$$
$$N_{BC} = |H_c - H_D| \pm A$$

The only departure from a simple equilibrium solution to the bracing connection design problem was in the assumption that frame action would allow the beam-to-column connection to be designed for an axial force equal to the maximum of H_c and A, or max (218, 150) = 218 kips. Thus, the design shown in Fig. 2.8 has its beam-to-column connection designed for N_{BC} = 218 kips and T_{BC} = 170 kips. Hence

$$N_{BC} = |218 - H_D| + 150 = 128$$

means that H_D = 150 kips and

$$V_D = \frac{12.25}{15} \times 150 = 122.5 \text{ kips}$$

From

$$T_{BC} = |V_B - 122.5| + 170 = 170$$
$$V_B = 122.5 \text{ kips}$$

Note that in order to maintain the beam to column loads of 170 kips shear and 218 kips tension, the gusset-to-beam-shear V_B must increase from 0 to 122.5 kips. Figure 2.13 shows the transition from the original load distribution to the final distribution as given in Fig. 2.13*d*. Note also that N_{BC} could have been set as 17.1 × 14 = 239 kips, rather than 218 kips, because this is the axial capacity of the connection at 170 kips shear. The N_{BC} value of 218 kips is used to cover the case when there is no excess capacity in the beam-to-column connection. Now, the gusset-to-beam and gusset-to-column interfaces will be checked for the redistributed loads of Fig. 2.13*d*.

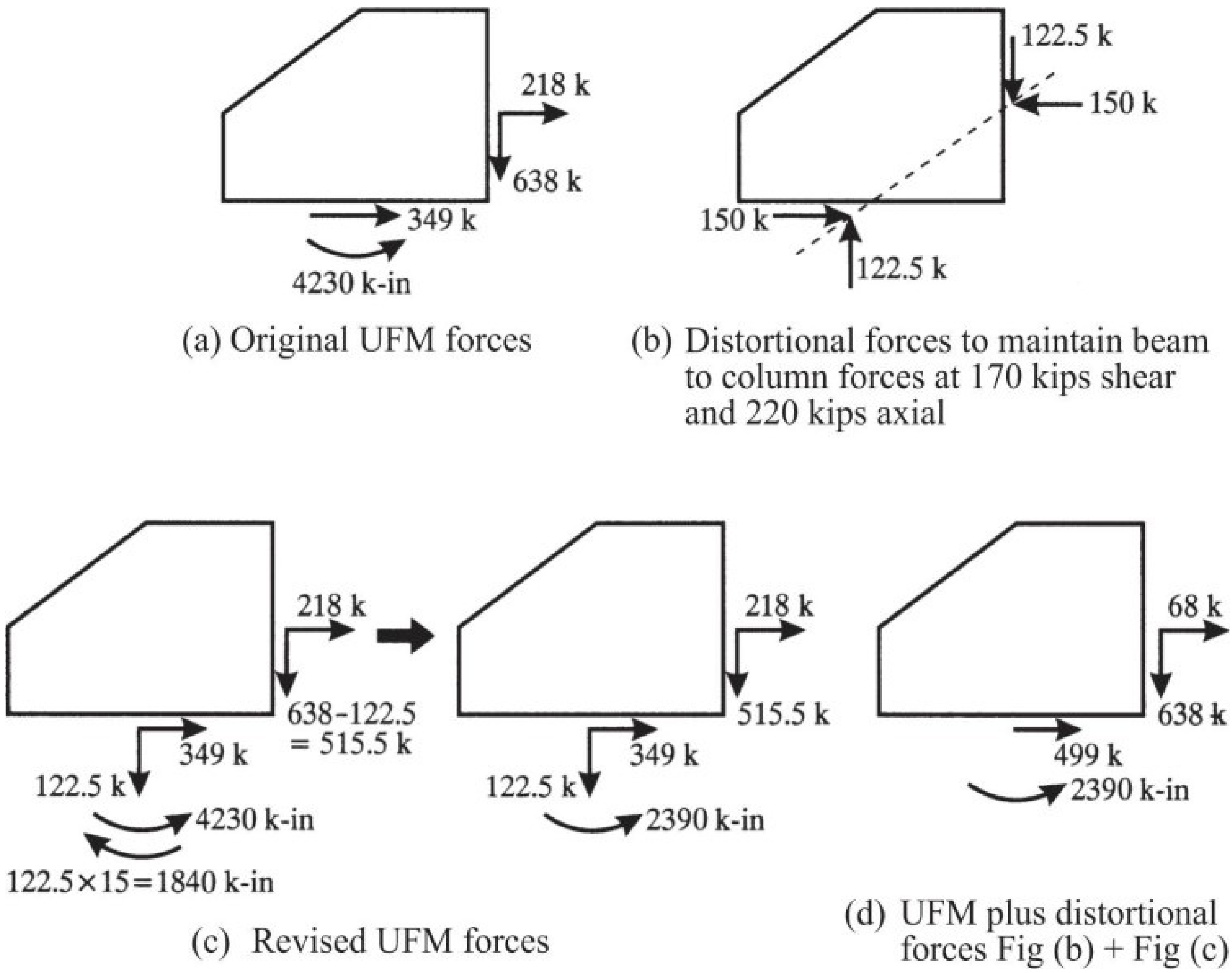


FIGURE 2.13 Admissible combining of UFM and distortional forces.

Gusset to Beam.

1. *Gusset stresses*:

$$f_v = \frac{499}{1.5 \times 30} = 11.1\ \text{ksi} < 21.6\ \text{ksi, ok}$$

$$f_b = \frac{2390 \times 4}{1.5 \times 30^2} = 7.08\ \text{ksi} < 32.4\ \text{ksi, ok}$$

2. *Weld of gusset to beam flange*:

$$f_R = \sqrt{11.1^2 + 7.08^2} \times \frac{1.5}{2} = 9.87$$

$$\Theta = \tan^{-1}(7.08 / 11.1)$$

$$= 32.5°$$

$$\mu = 1 + 0.5 \sin^{1.5}(32.5)$$

$$= 1.20°$$

$$D = \frac{9.85}{1.392 \times 1.20} = 5.90$$

A ⅜-in fillet weld is indicated, which is less than what was provided. No ductility factor is used here because the loads include a redistribution.

Gusset to Column. This connection is ok without calculations because the loads of Fig. 2.13*d* are no greater than the original loads of Fig. 2.13*a*.

Discussion. From the foregoing analysis, it can be seen that the AISC-suggested procedure for the beam-to-column connection, where the actual normal force

$$N_{BC} = |H_c - H_D| \pm A$$

is replaced by

$$N_{BC} = \max\,(H_c, A)$$

is justified.

It has been shown that the connection is strong enough to carry the distortional forces of Fig. 2.13*b*, which are larger than the elastic distortional forces.

In general, the entire connection could be designed for the combined UFM forces and distortional forces, as shown in Fig. 2.13*d* for this example. This set of forces is also admissible. The UFM forces are admissible because they are in equilibrium with the applied forces. The distortional forces are in equilibrium with zero external forces. Under each set of forces, the parts of the connection are also in equilibrium. Therefore, the sum of the two loadings is admissible because each individual loading is admissible. A safe design is thus guaranteed by the lower bound theorem of limit analysis. The difficulty is in determining the distortional forces. The elastic distortional forces could be used, but they are only an estimate of the true distortional forces. The distortional forces depend as much on the properties of the connection, which are inherently inelastic and affect the maintenance of the angle between the members, as on the properties and lengths of the members of the frame. For this example, the distortional forces are [(150 – 110)/110] × 100 = 36% greater than the elastic distortional forces. In full-scale tests by Gross (1990) as reported by Thornton (1991), the distortional forces were about 2½ times the elastic distortional forces while the overall frame remained elastic. Because of the difficulty in establishing values for the distortional forces, and because the UFM has been shown to be conservative when they are ignored (Thornton, 1991, 1995b), they are not included in bracing connection design, except implicitly as noted here to justify replacing $|H_c - H_D| \pm A$ with max (H_c, A).

2.2.1.6 Load Paths Have Consquences. The UFM produces a load path that is consistent with the gusset plate boundaries. For instance, if the gusset-to-column connection is to a column web, no horizontal force is directed perpendicular to the column web because unless it is stiffened, the web will not be able to sustain this force. This is clearly shown in the physical test results of Gross (1990) where it was reported that bracing connections to column webs were unable to mobilize the column weak axis stiffness because of web flexibility.

A mistake that is often made in connection design is to assume a load path for a part of the connection, and then to fail to follow through to make the assumed load path capable of carrying the loads (satisfying the limit states). Note that load paths include not just connection elements, but also the members to which they are attached. As an example, consider the connection of Fig. 2.14*a*. This is a configuration similar to that of Fig. 2.1*b* with minimal transfer force into and out of the braced bay. It is proposed to consider the welds of the gusset to the beam flange and to the ½-in end plate as a single L-shaped weld. This will be called the L weld method, and is similar to model 4, the parallel force method, which is discussed by Thornton (1991). This is an apparently perfectly acceptable proposal and will result in very small welds because the centroid of the weld group will lie on or near the line of action of the brace. In the example of Fig. 2.14*a*, the geometry is arranged to cause the weld centroid to lie exactly on the line of action to simplify the calculation. This makes the weld uniformly loaded, and the force per inch is $f = 300/(33 + 20) = 5.66$ kips/in in a direction parallel to the brace line of action, which has horizontal and vertical components of $5.66 \times 0.7071 = 4.00$ kips/in. This results in free-body diagrams for the gusset, beam, and column as shown in Fig. 2.14*b*. Imagine how difficult it would be to obtain the forces on the free-body diagram of the gusset and other members if the weld were not uniformly loaded! Every inch of the weld would have a force of different magnitude and direction. Note that while the gusset is in equilibrium under the parallel forces alone, the beam and the column require the moments as shown to provide equilibrium. For comparison, the free-body diagrams for the UFM are given in Fig. 2.14*c*. These forces are always easy to obtain and no moments are required in the beam or column to satisfy equilibrium.

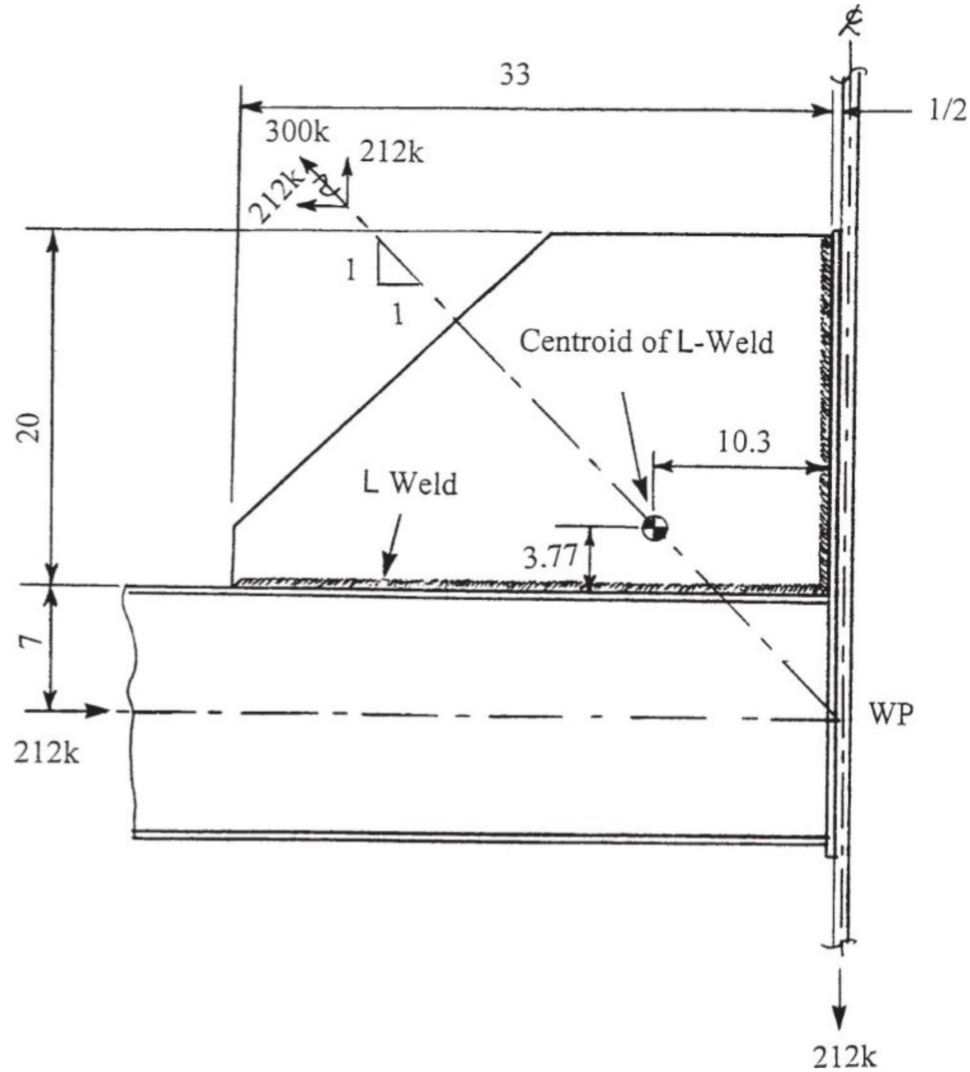


FIGURE 2.14*a* Bracing connection to demonstrate the consequences of an assumed load path.

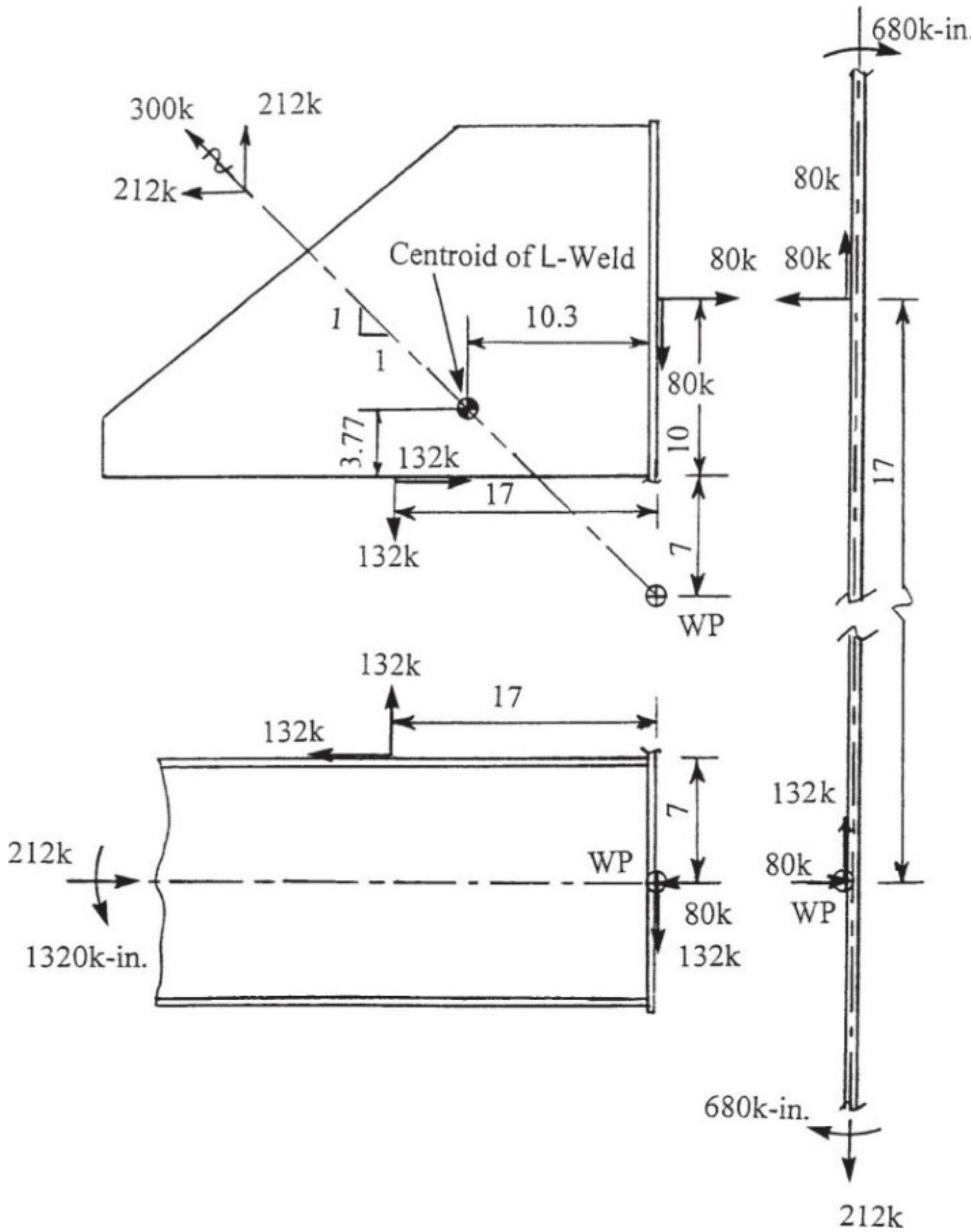


FIGURE 2.14*b* Free body diagrams for L weld method.

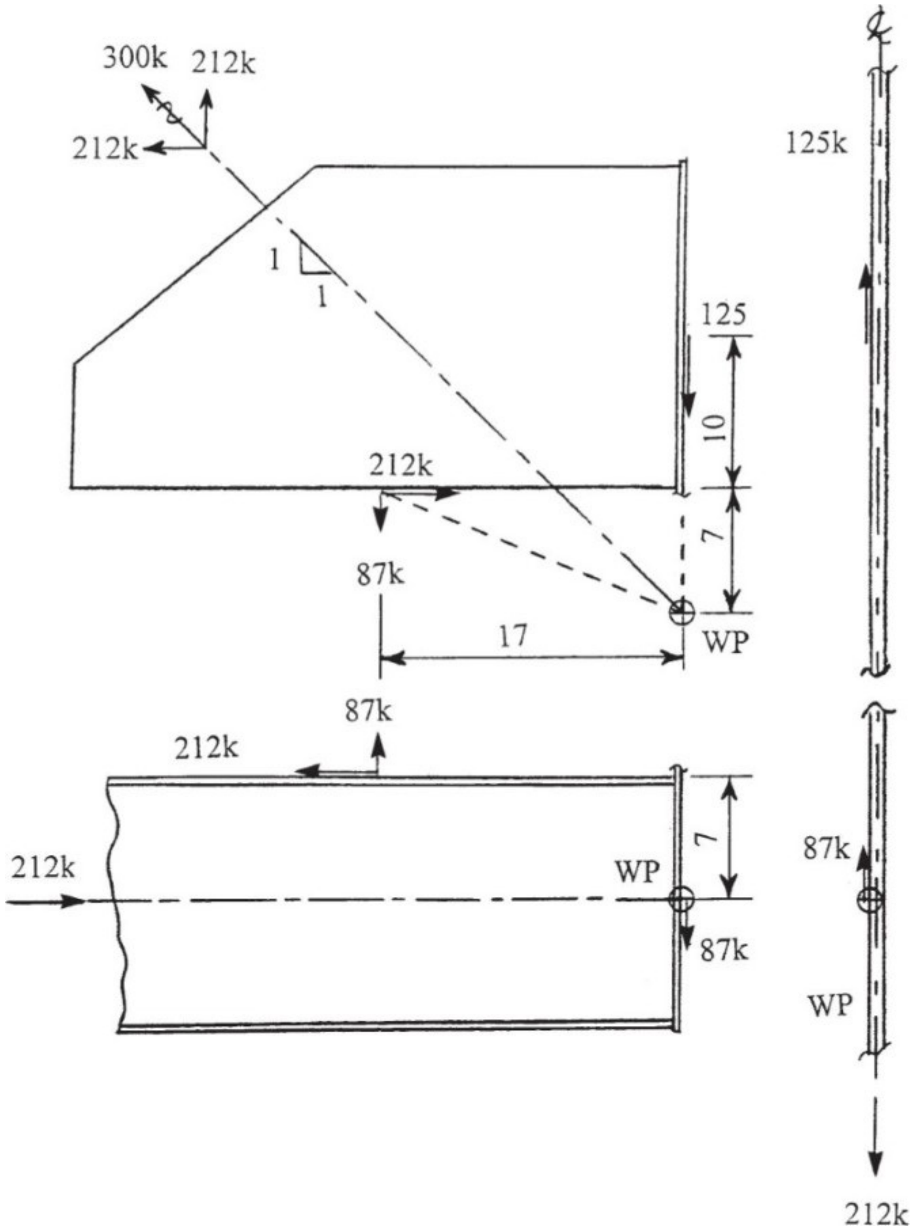


FIGURE 2.14*c* Free body diagrams for uniform force method.

From the unit force $f = 5.66$ kips/in, the gusset-to-beam and gusset-to-end plate weld sizes are $D = 5.66/(2 \times 1.392) = 2.03$ sixteenths, actual required size. For comparison, the gusset-to-beam weld for the UFM would be

$$D = \frac{\sqrt{87^2 + 212^2}}{2 \times 33 \times 1.392} \times 1.25 = 3.12$$

actual required size, a 54% increase over the L weld method weld of $D = 2.03$. While the L weld method weld is very small, as expected with this method, now consider the load paths through the rest of the connection.

Gusset to Column

BOLTS. The bolts are A325N, ⅞-in diameter, with $\phi r_v = 24.3$ kips and $\phi r_t = 40.6$ kips. The shear per bolt is $80/12 = 6.67$ kips < 24.3 kips, ok. The tension per bolt is $80/12 = 6.67$ kips, but ϕr_t must be reduced due to interaction. Thus

$$\phi r_t' = 0.75\left[1.3\times 90 - \left(\frac{90}{0.75\times 54}\right)\left(\frac{6.67}{0.601}\right)\right]0.601$$

$$= 41.6 \text{ kips} > 40.6 \text{ kips}$$

so use $\phi r_t' = 40.6$ kips. Since 40.6 > 6.67, the bolts are ok for shear and tension.

END PLATE. This involves the standard prying action calculations as follows:

$$b = (5.5 - 0.375)/2 = 2.56,\ a = (8 - 5.5)/2 = 1.25 < 1.25b$$

so use

$$a = 1.25;\ b' = 2.56 - 0.875/2 = 2.12,\ a' = 1.25 + 0.875/2$$
$$= 1.69,\ \rho = b'/a' = 1.25,\ \delta = 1 - 0.9375/3 = 0.69,\ p = 3;$$
$$t_c = \sqrt{((4.44\times 40.6\times 2.12/(3\times 58))} = 1.48;$$

try an end plate ½ in thick.

Calculate

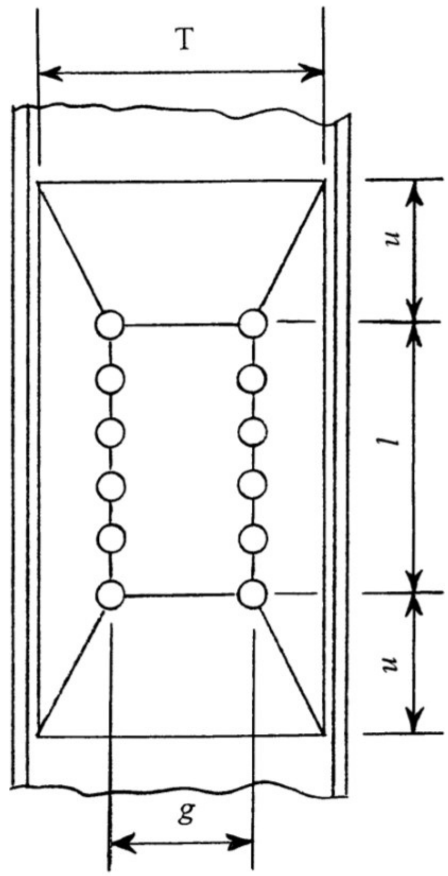
$$a' = \frac{1}{0.698(1+1.25)}\left[\left(\frac{1.48}{0.5}\right)^2 - 1\right] = 4.94$$

Since $\alpha' > 1$, use $\alpha' = 1$, and the design tension strength is

$$T_d = 40.6\times\left(\frac{0.5}{1.48}\right)^2\times 1.69 = 7.83 \text{ kips} \geq 6.67 \text{ kips, ok}$$

The ½-in end plate is ok.

COLUMN WEB. The column web sees a transverse force of 80 kips. Figure 2.14*d* shows a yield-line analysis (Anand and Bertz, 1981) of the column web. The normal force ultimate strength of the yield pattern shown is



$$u = \sqrt{\frac{T(T-g)}{2}}$$

FIGURE 2.14*d* Deformation method for yield-line analysis of column web.

$$P_u = 8m_p\left\{\sqrt{\frac{2T}{T-g}} + \frac{l}{2(T-g)}\right\}$$

where $m_p = ¼\, F_y t_w^2$. For the present problem, $m_p = 0.25 \times 50 \times (0.44)^2 = 2.42$ kips-in/in, T = 11.25 in, g = 5.5 in, and l = 15 in, so

$$P_u = 8 \times 2.42\left(\sqrt{\frac{2 \times 11.25}{(11.25-5.5)}} + \frac{15}{2(11.25-5.5)}\right) = 63.5 \text{ kips}$$

Thus $\phi P_u = 0.9 \times 63.5 = 57.2$ kips < 80 kips, no good, and the column web is unable to sustain the horizontal force from the gusset without stiffening or a column web-doubler plate. Figure 2.15 shows a possible stiffening arrangement.

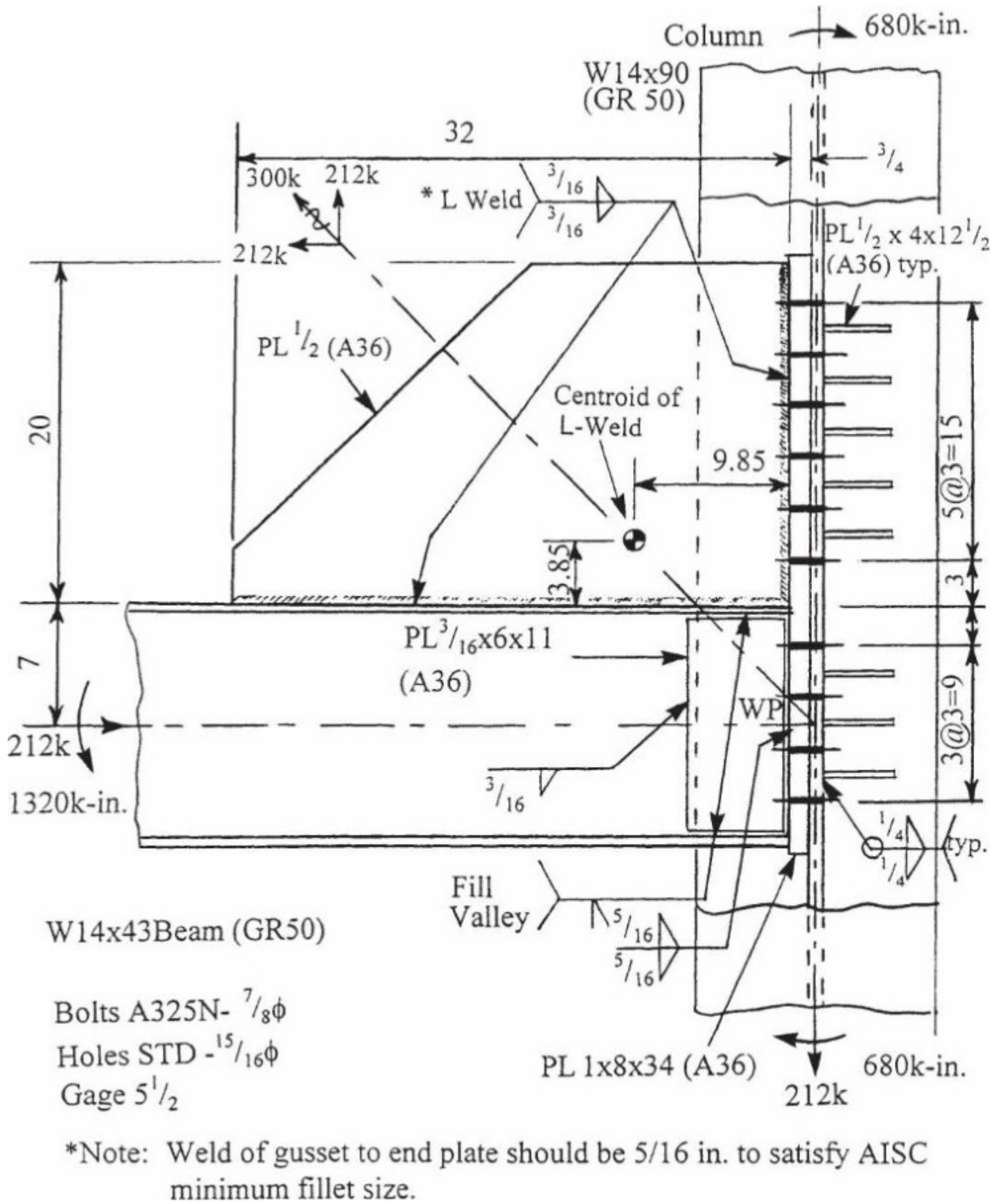


FIGURE 2.15 Design by L weld method.

It should be noted that the yield-line pattern of Fig. 2.14*d* compromises the foregoing end plate/prying action calculation. That analysis assumed double curvature with a prying force at the toes of the end plate a distance *a* from the bolt lines. But the column web will bend away as shown in Fig. 2.14*d* and the prying force will not develop. Thus, single curvature bending in the end plate must be assumed, and the required end plate thickness is given by AISC 2016.

$$t_{\text{req}} = \sqrt{\frac{4.44Tb'}{pF_u}} = \sqrt{\frac{4.44 \times 6.67 \times 2.12}{3 \times 58}} = 0.600 \text{ in}$$

and a ⅝-in-thick end plate is required.

Gusset to Beam. The weld is already designed. The beam must be checked for web yield and crippling, and web shear.

WEB YIELD. $\phi R_{wy} = 10 \times 0.305 \times 50\,(32 + 2.5 \times 1.12) = 531$ kips > 132 kips, ok

WEB CRIPPLING

$$\phi R_{wcp} = 0.75 \times 0.8 \times 0.305^2 \left[1 + \left(\frac{32}{13.7}\right)\left(\frac{0.305}{0.530}\right)^{1.5}\right]\sqrt{\frac{50 \times 29{,}000 \times 0.530}{0.305}}$$

$$= 179 \text{ kips} > 132 \text{ kips, ok}$$

WEB SHEAR. The 132-kip vertical load between the gusset and the beam flange is transmitted to the beam-to-column connection by the beam web. The shear design strength is

$$\phi R_{vw} = 1.0 \times 0.6 \times 0.305 \times 13.7 \times 50 = 125 \text{ kips} < 132 \text{ kips, no good}$$

To carry this much shear, a web-doubler plate is required. Starting at the toe of the gusset plate, 132/33 = 4.00 kips of shear is added per inch. The doubler must start at a distance x from the toe, where $4.00x = 125$, $x = 31.0$ in. Therefore, a doubler of length 34 – 31.0 = 2 in is required, measured from the face of the end plate. The doubler thickness t_d required is $1.0 \times 0.6 \times 50 \times (t_d + 0.305) \times 13.4 = 132$, $t_d = 0.02$ in, so use a minimum thickness 3/16-in plate of grade 50 steel. If some yielding before ultimate load is reached is acceptable, grade 36 plate can be used. The thickness required would be $t_d = 0.02 \times 50/36 = 0.028$ in, so a 3/16-in A36 plate is also ok.

Beam to Column. The fourth connection interface (the first interface is the brace-to-gusset connection, not considered here), the beam-to-column, is the most heavily loaded of them all. The 80 kips horizontal between the gusset and column must be brought back into the beam through this connection to make up the beam (strut) load of 212 kips axial. This connection also sees the 132 kips vertical load from the gusset-to-beam connection.

BOLTS. The shear per bolt is 132/8 = 16.5 kips < 24.3 kips, ok. The reduced tension design strength is

$$\phi r_t' = 0.75\left[1.3 \times 90 - \left(\frac{90}{0.75 \times 54}\right)\left(\frac{16.5}{0.601}\right)\right]0.601$$

$$= 25.2 \text{ kips, } 40.6 \text{ kips}$$

so use $\phi r_t' = 25.2$ kips. Since 25.2 kips > 80/8 = 10.0 kips, the bolts are ok for tension and shear.

END PLATE. As discussed for the gusset-to-column connection, there will be no prying action and hence double curvature in the end plate, so the required end plate thickness is

$$t_{\text{req}} = \sqrt{\frac{4.44 \times 10.0 \times 2.12}{3 \times 58}} = 0.736 \text{ in}$$

A ¾-in end plate is required. This plate will be run up to form the gusset-to-column connection, so the entire end plate is a ¾-in plate (A36).

COLUMN WEB. Using the yield-line analysis for the gusset-to-column connection, $T =$

11.25, g = 5.5, l = 9

$$\phi P_u = 0.9 \times 8 \times 2.42\left[\sqrt{\frac{2\times 11.25}{5.75}} + \frac{9}{2\times 5.75}\right]$$

$$= 48 \text{ kips}, < 80 \text{ kips, no good}$$

Again, the column web must be stiffened as shown in Fig. 2.15, or a doubler must be used.

STIFFENER. If stiffeners are used, the most highly loaded one will carry the equivalent tension load of three bolts or 30.0 kips to the column flanges. The stiffener is treated as a simply supported beam 12½ in long loaded at the gage lines. Figure 2.15 shows the arrangement. The shear in the stiffener is 30.0/2 = 15.0 kips, and the moment is 15.0 × (12.5 – 5.5)/2 = 52.5 kips-in. Try a stiffener of A36 steel ½ × 4:

$$f_v = \frac{15.0}{0.5\times 4} = 7.50 \text{ ksi} < 21.6 \text{ ksi, ok}$$

$$f_b = \frac{52.5\times 4}{0.5\times 4^2} = 26.3 \text{ ksi} < 32.4 \text{ ksi, ok}$$

The ½ × 4 stiffener is ok. Check buckling, b/t = 4/0.5 = 8 < 15, ok.

Weld of Stiffener to Column Web. Assume about 3 in of weld at each gage line is effective, that is 1.5 × 1 × 2 = 3. Then

$$D = \frac{10.0 + 0.5\times 10.0}{2\times 3\times 1.5\times 1.392} = 1.19 \qquad \text{use } \frac{3}{16} \text{ fillet welds}$$

Weld of Stiffener to Column Flange

$$D = \frac{15.0}{2\times(4 - 0.75)\times 1.392} = 1.66 \qquad \text{use } \frac{3}{16} \text{ fillet welds}$$

Weld of End Plate to Beam Web and Doubler Plate. The doubler is 3/16 in thick and the web is 0.305 in thick, so 0.1875/0.4925 = 0.38 or 38% of the load goes to the doubler and 42% goes to the web. The load $\sqrt{132^2 + 80^2} = 154$ kips. The length of the weld is 13.66 – 2 × 0.530 = 12.6 in. The weld size to the doubler is D = 0.38 × 154/(2 × 12.6 × 1.392) = 1.67 and that to the web is D = 0.42 × 154/(2 × 12.6 × 1.392) = 1.84, so 3/16 in minimum fillets are indicated.

Additional Discussion. The 80-kip horizontal force between the gusset and the column must be transferred to the beam-to-column connections. Therefore, the column section must be capable of making this transfer. The weak axis shear capacity (design strength) of the column is

$$\phi R_v = 1.0 \times 0.6 \times 50 \times 0.710 \times 14.5 \times 2 = 618 \text{ kips} > 80 \text{ kips, ok}$$

It was noted earlier that the column and the beam require couples to be in equilibrium. These couples could act on the gusset-to-column and gusset-to-beam interfaces, since they are free vectors, but this would totally change these connections. Figure 2.14*b* shows them acting in the members instead, because this is consistent with the L weld method. For the column, the moment is 80 × 17 = 1360 kips-in and is shown with half above and half below the connection. The bending strength of the column is ϕM_{py} = 0.9 × 50 × 133 = 5985 kips-in so the 1360/2 = 680 kips-in is 11% of the capacity, which probably does not seriously reduce the column's weak axis bending strength. For the beam, the moment is 132 × 17 – 132 × 7 = 1320 kips-in (should be equal and opposite to the column moment since the connection is concentric—the slight difference is due to numerical roundoff). The bending strength of the beam is ϕM_{px} = 0.9 × 50 × 69.6 = 3146 kips-in so the 1320 kips-in couple uses up 42% of the beam's bending strength. This will greatly reduce its capacity to carry 212 kips in compression and is probably not acceptable.

This completes the design of the connection by the L weld method. The reader can clearly see how the loads filter through the connection, that is, the load paths involved. The final connection as shown in Fig. 2.15 has small welds of the gusset to the beam and the end plate, but the rest of the connection is very expensive. The column stiffeners are expensive, and also compromise any connections to the opposite side of the column web. The ¾-in end plate must be flame cut because it is generally too thick for most shops to shear. The web-doubler plate is an expensive detail and involves welding in the beam k-line area, which may be prone to cracking (AISC, 1997). Finally, although the connection is satisfactory, its internal admissible force distribution that satisfies equilibrium requires generally unacceptable couples in the members framed by the connection.

As a comparison, consider the design that is achieved by the UFM. The statically admissible force distribution for this connection is given in Fig. 2.14*c*. Note that all elements (gusset, beam, and column) are in equilibrium with no couples. Note also how easily these internal forces are computed. The final design for this method, which can be verified by the reader, is shown in Fig. 2.16. There is no question that this connection is less expensive than its L weld counterpart in Fig. 2.15, and it does not compromise the strength of the column and strut. To summarize, the L weld method seems a good idea at the outset, but a complete "trip" through the load paths ultimately exposes it as a fraud, that is, it produces expensive and unacceptable connections. As a final comment, a load path assumed for part of a connection affects every other part of the connection, including the members that frame to the connection.

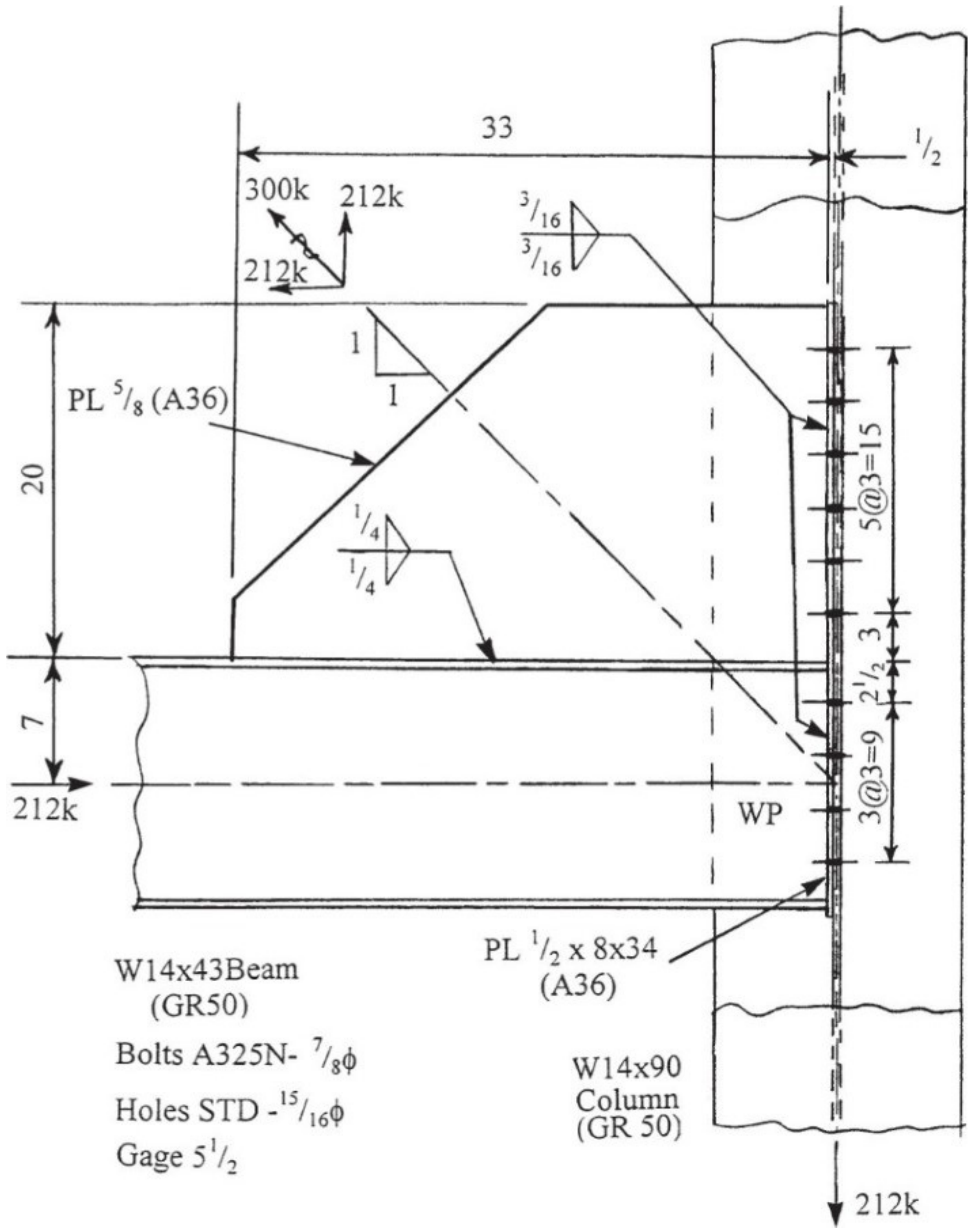


FIGURE 2.16 Design by uniform force method.

2.2.1.7 Bracing Connections Utilizing Shear Plates. All of the bracing connection examples presented here have involved connections to the column using end plates or double clips, or are direct welded. The UFM is not limited to these attachment methods. Figures 2.17 and 2.18 show connections to a column flange and web, respectively, using shear plates. These connections are much easier to erect than the double-angle or shear plate type because the beams can be brought into place laterally and easily pinned. For the column web connection of Fig. 2.18, there are no common bolts that enhance erection safety. The connections shown were used on an actual job and were designed for the tensile strength of the brace to resist seismic loads in a ductile manner.

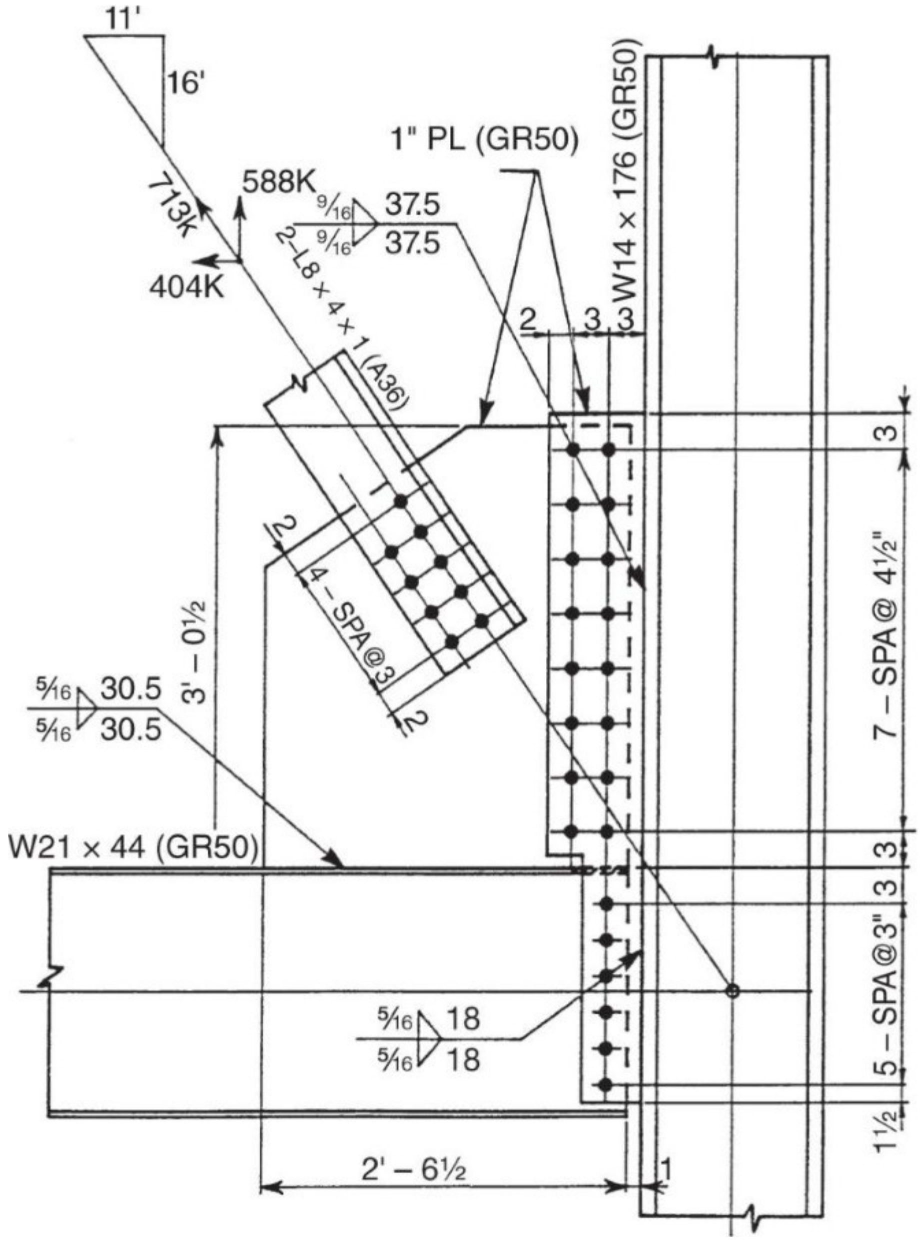


FIGURE 2.17 Bracing connection to a column flange utilizing a shear plate.

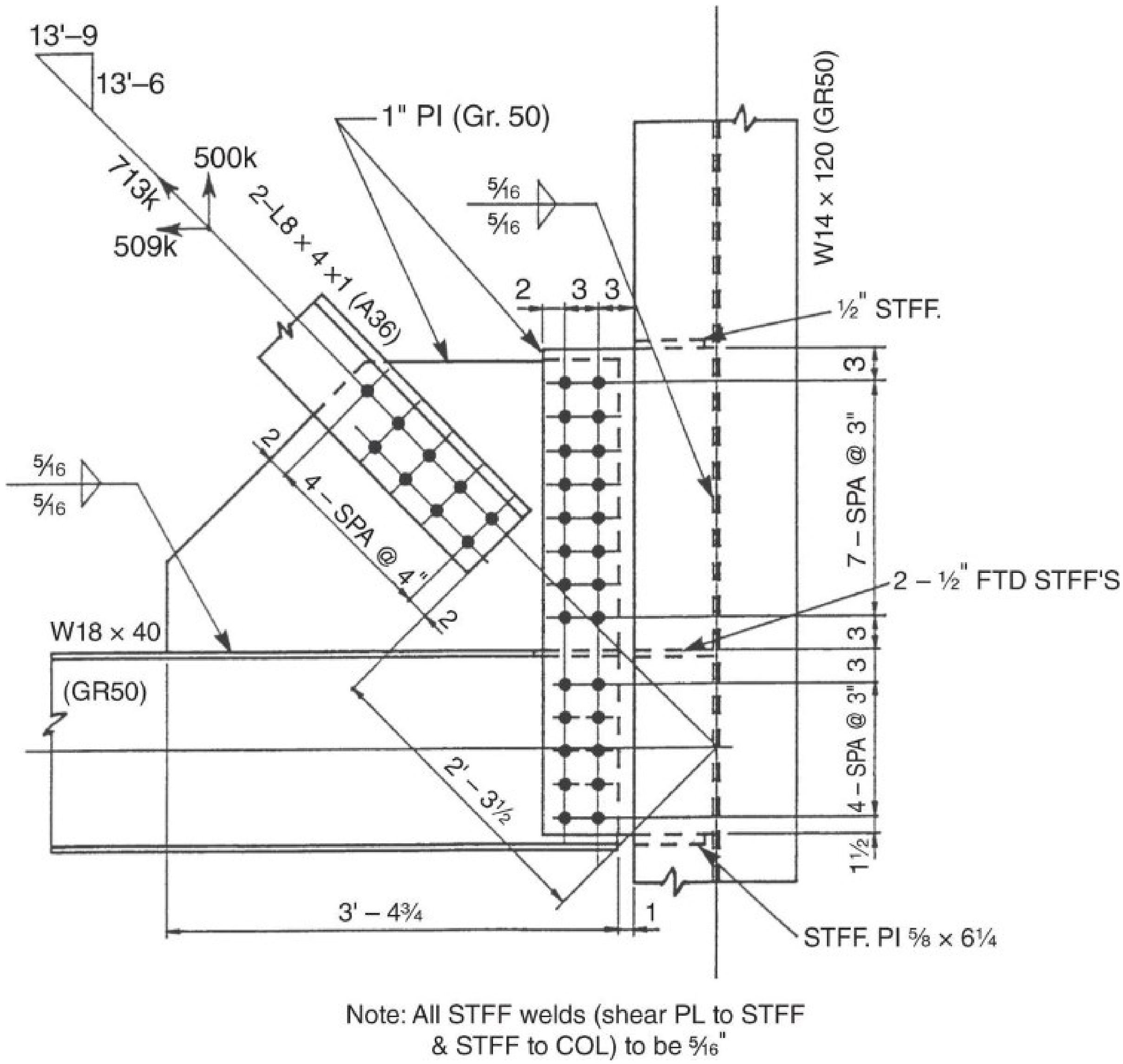


FIGURE 2.18 Bracing connection to a column web utilizing a shear plate.

2.2.1.8 Connections with Non-Concentric Work Points. The UFM can be easily generalized to this case as shown in Fig. 2.19*a*, where x and y locate the specified non-concentric work point (WP) from the intersection of the beam and column flanges. All of the forces on the connection interfaces are the same as for the concentric UFM, except that there is an extra moment on the gusset plate $M = Pe$, which can be applied to the stiffer gusset edge. It should be noted that this non-concentric force distribution is consistent with the findings of Richard (1986), who found very little effect on the force distribution in the connection when the work point is moved from concentric to non-concentric locations. It should also be noted that a non-concentric work point location induces a moment in the structure of $M = Pe$, and this may need to be considered in the design of the frame members. In the case of Fig. 2.19*a*, since the moment $M = Pe$ is assumed to act on the gusset-to-beam interface, it must also be assumed to act on the beam outside of the connection, as shown. In the case of a connection to a column web, this will be the actual distribution (Gross, 1990), unless the connection to the column mobilizes the flanges, as for instance is done in Fig. 2.15 by means of stiffeners.

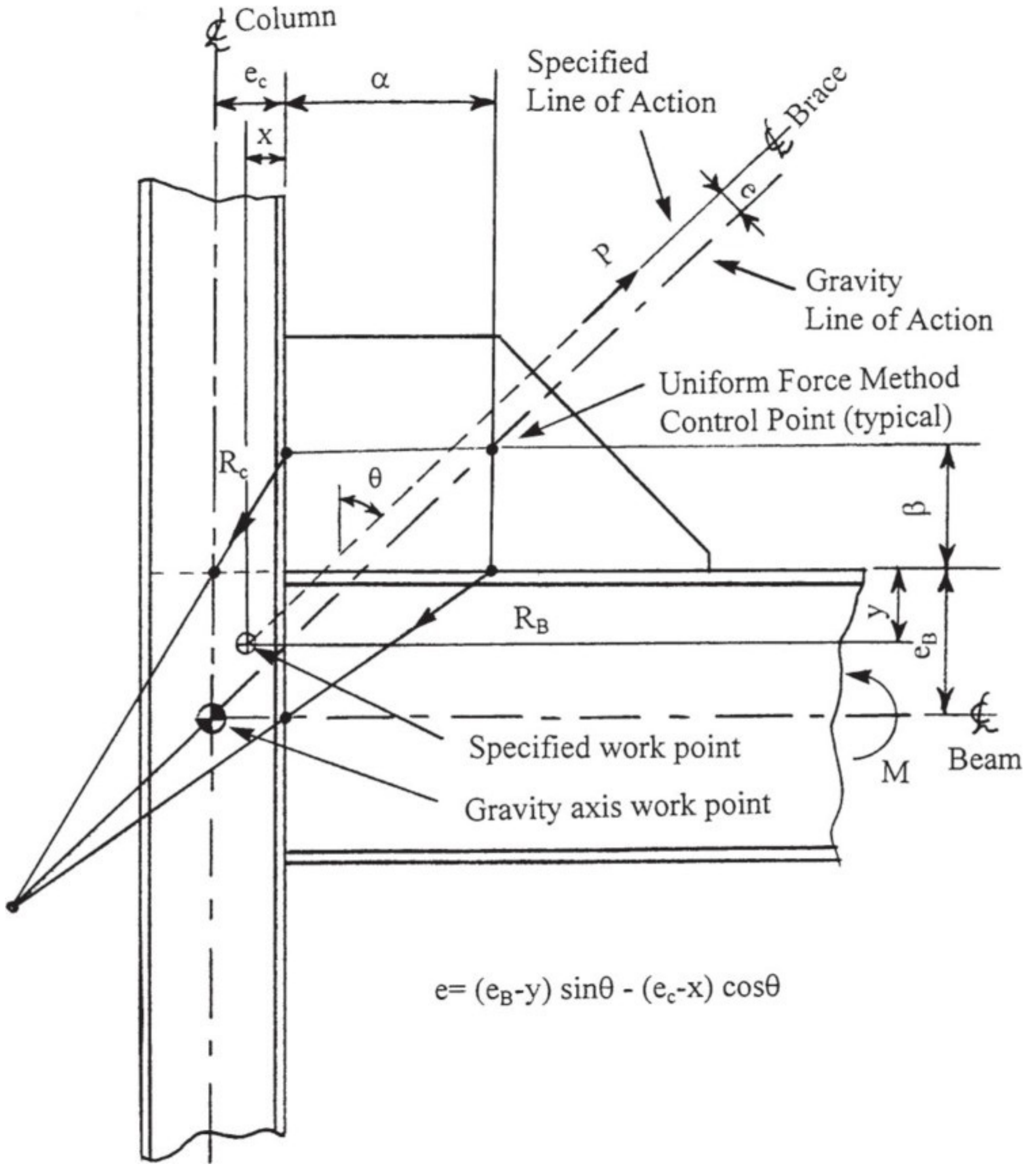


FIGURE 2.19*a* Nonconcentric uniform force method.

An alternate analysis, where the joint is considered rigid, that is, a connection to a column flange, the moment *M* is distributed to the beam and column in accordance with their stiffnesses (the brace is usually assumed to remain an axial force member and so is not included in the moment distribution), can be performed. If η denotes the fraction of the moment that is distributed to the beam, then horizontal and vertical forces, *H*′ and *V*′, respectively, acting at the gusset to beam, gusset-to-column, and beam-to-column connection centroids due to the distribution of *M* are

$$H' = \frac{(1-\eta)M}{\bar{\beta} + e_B}$$

$$V' = \frac{M - H'\bar{\beta}}{\bar{\alpha}}$$

These forces, shown in Fig. 2.19*b*, are to be added algebraically to the concentric UFM forces acting at the three connection interfaces. Note that for connections to column webs, $\eta = 1$, $H' = 0$, and $V' = M/\bar{\alpha}$, unless the gusset-to-column web and beam-to-column web connections positively engage the column flanges, as for instance in Fig. 2.15.

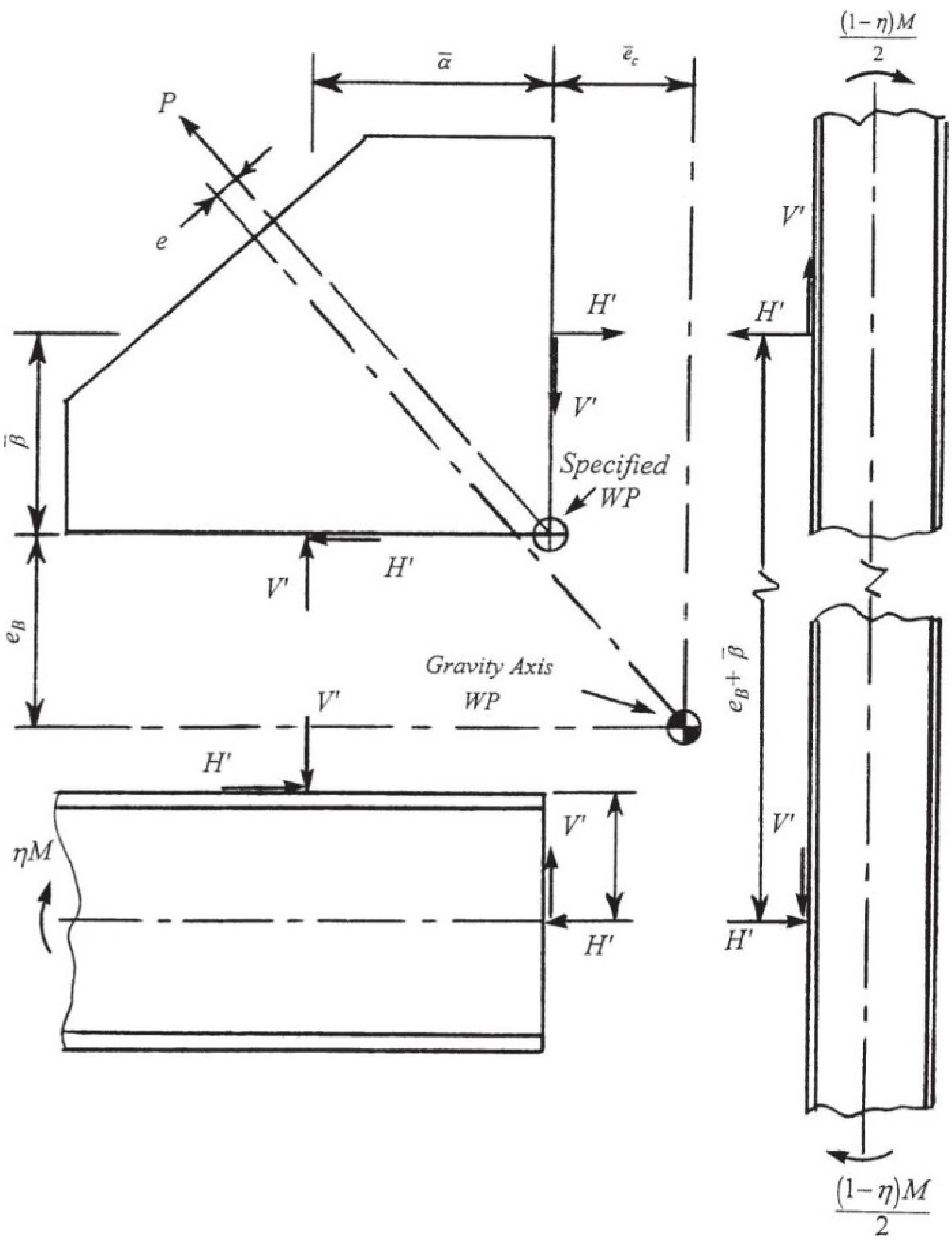


FIGURE 2.19*b* Extra forces due to nonconcentric work point.

Example Consider the connection of Sec. 2.2.1.4 as shown in Fig. 2.8, but consider that the brace line of action passes through the corner of the gusset rather than to the gravity axis intersection of the beam and the column. Using the data of Fig. 2.8, $e_C = 8.37$, $e_B = 12.05$, $\bar{\alpha} = 15.0$, $\bar{\beta} = 12.25$,

$$\theta = \tan^{-1}\left[\left(10\frac{11}{16}\right)/12\right] = 41.7°$$

Since the specified work point is at the gusset corner, $x = y = 0$, and $e = 12.05 \sin 41.7° - 8.37 \cos 41.7° = 1.76$ in. Thus, $M = Pe = 855 \times 1.77 = 1510$ kips-in and using the frame data of Sec. 2.2.1.5,

$$\eta = 13.5/(13.5 + 80) = 0.144$$

$$H' = \frac{(1 - 0.144)1510}{(12.25 + 12.05)} = 53 \text{ kips}$$

$$V' = \frac{1510 - 53 \times 12.25}{15} = 58 \text{ kips}$$

These forces are shown on the gusset in Fig. 2.19*c*. This figure also shows the original UFM forces of Fig. 2.13*a*. The design of this connection will proceed in the same manner as shown in Sec. 2.2.1.4, but the algebraic sum of the original forces and the additional forces due to the non-concentric work point are used on each interface.

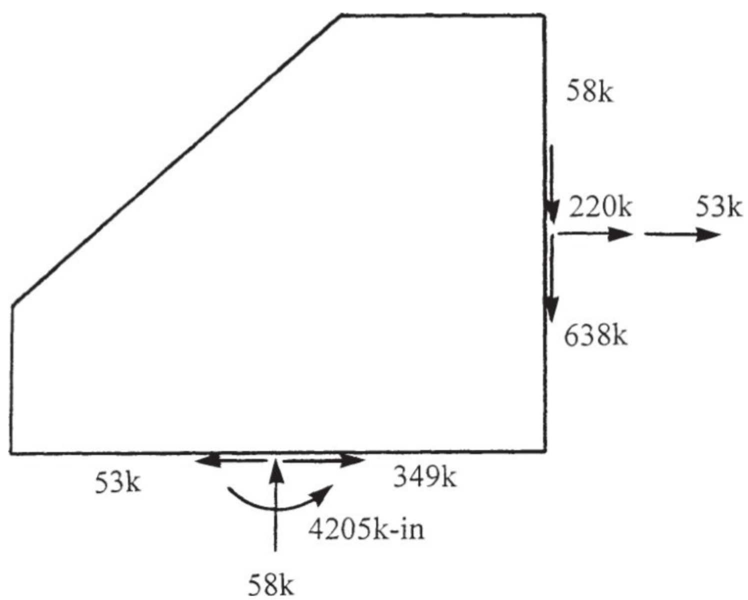


FIGURE 2.19*c* Uniform force method and nonconcentric forces combined.

2.2.2 Truss Connections

2.2.2.1 Introduction. The UFM as originally formulated can be applied to trusses as well as to bracing connections. After all, a vertical bracing system is just a truss as seen in Fig. 2.1, which shows various arrangements. But bracing systems generally involve orthogonal members, whereas trusses, especially roof trusses, often have a sloping top chord. In order to handle this situation, the UFM has been generalized as shown in Fig. 2.20 to include nonorthogonal members. As before, α and β locate the centroids of the gusset edge connections and must satisfy the constraint shown in the box on Fig. 2.20. This can always be arranged when designing a connection, but in checking a given connection designed by some other method, the constraint may not be satisfied. The result is gusset edge couples, which must be considered in the design.

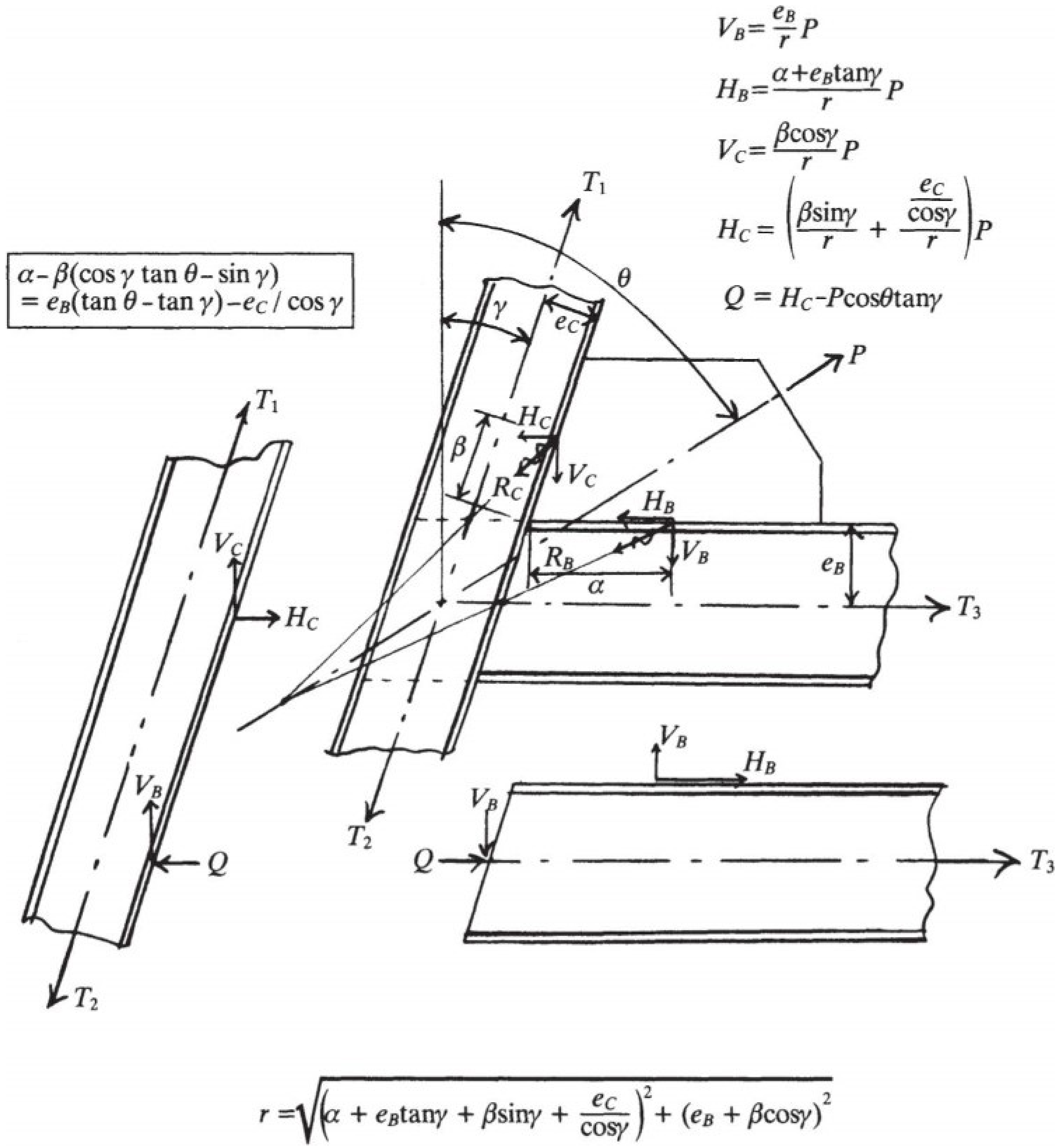


FIGURE 2.20 Generalized uniform force method.

2.2.2.2 A Numerical Example. As an application of the UFM to a truss, consider the situation of Fig. 2.21. This is a top chord connection in a large aircraft hangar structure. The truss is cantilevered from a core support area. Thus, the top chord is in tension. The design shown in Fig. 2.21 was obtained by generalizing the KISS method (Thornton, 1995b) shown in Fig. 2.22 for orthogonal members to the nonorthogonal case. The KISS method is the simplest admissible design method for truss and bracing connections. On the negative side, however, it generates large, expensive, and unsightly connections. The problem with the KISS method is the couples required on the gusset edges to satisfy equilibrium of all parts. In the Fig. 2.21 version of the KISS method, the truss diagonal, horizontal, and vertical components are placed at the gusset edge centroids as shown. The couples 15,860 kips-in on the top edge and 3825 kips-in on the vertical edge are necessary for equilibrium of the gusset, top chord, and truss vertical, with the latter two experiencing only axial forces away from the connection. It is these couples that require the ¾-in chord doubler plate, the 7/16-in fillets between the gusset and chord, and the 38-bolt ⅞-in end plate on the vertical edge.

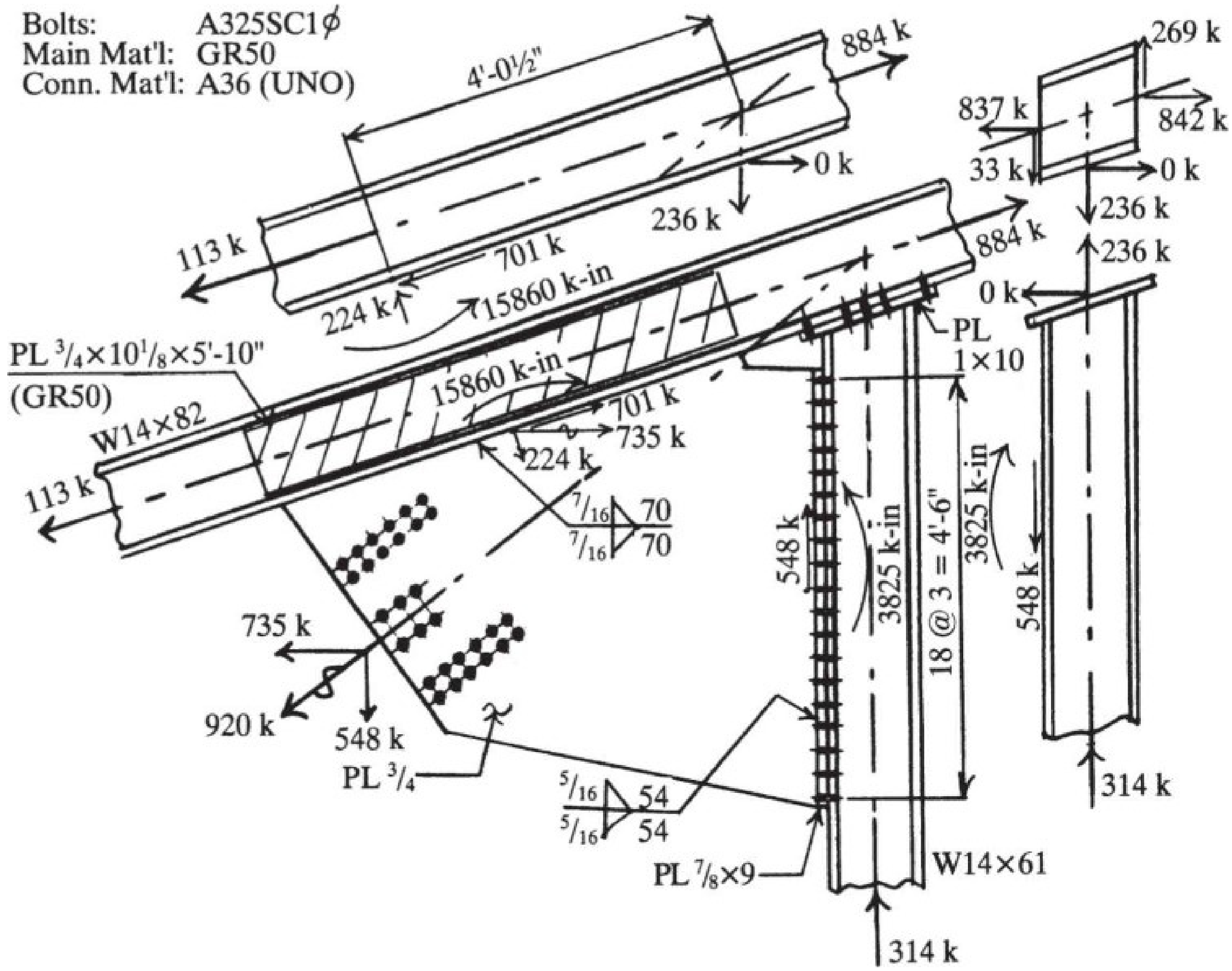


FIGURE 2.21 KISS method—gusset forces arc brace components.

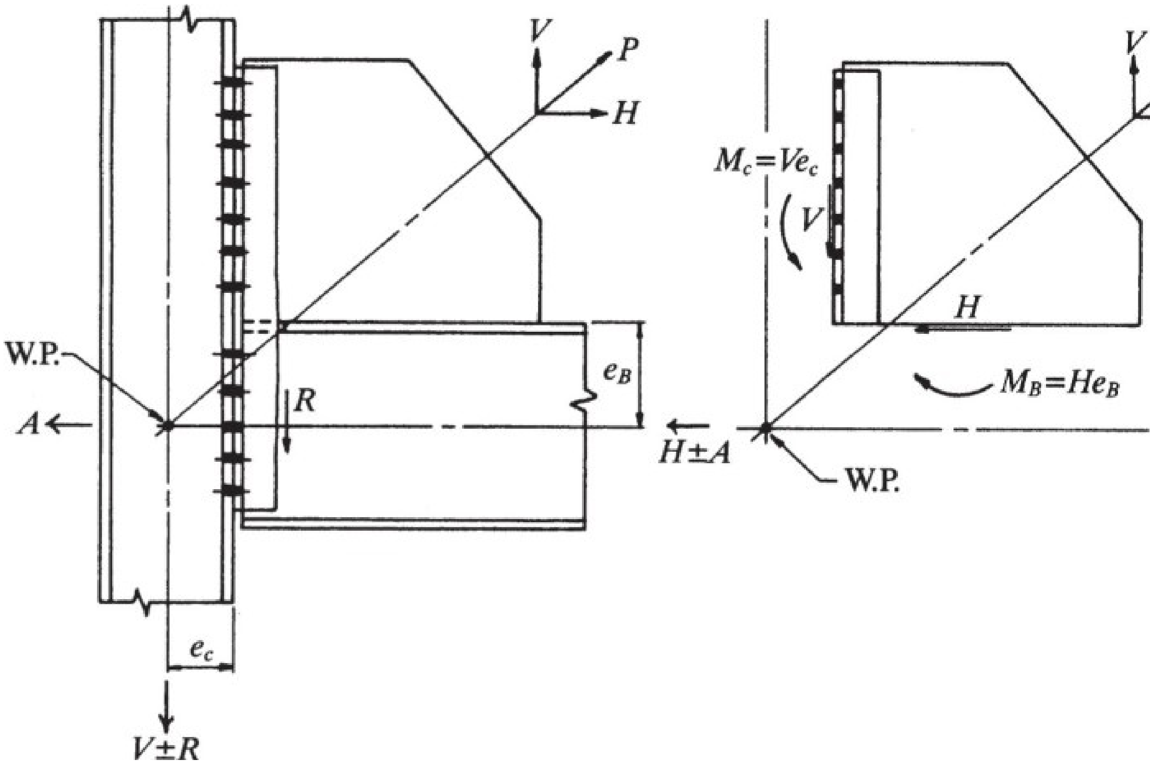


FIGURE 2.22 The KISS method.

The design shown in Fig. 2.23 is also obtained by the KISS method with the brace force resolved into tangential components on the gusset edges. Couples still result, but are much smaller than in Fig. 2.21. The resulting connection requires no chord doubler plate, 5/16-in fillets of the gusset to the chord, and a 32-bolt ¾-in end plate on the vertical edge. This design is much improved over that of Fig. 2.21.

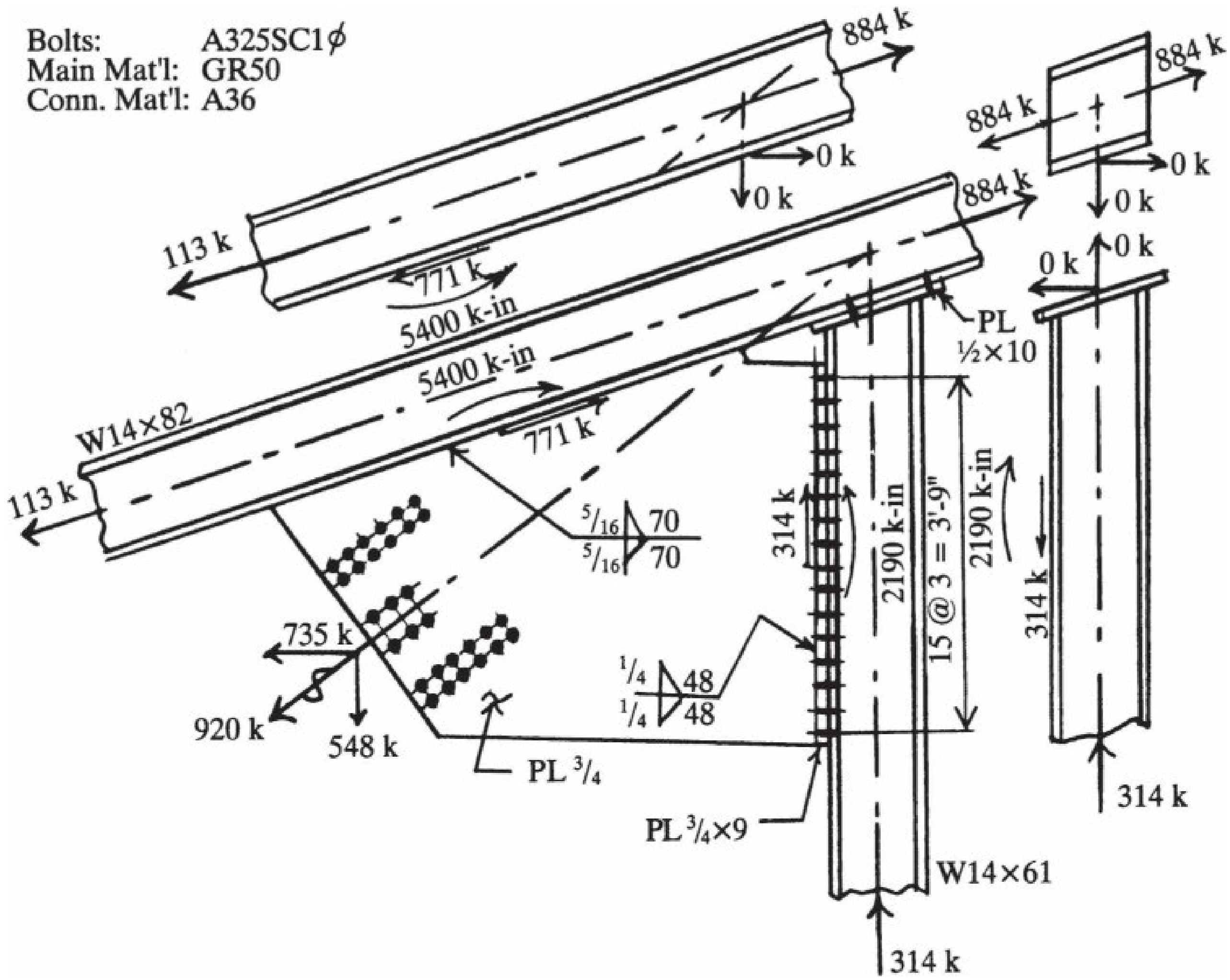


FIGURE 2.23 KISS method—brace components are tangent to gusset edges.

When the UFM of Fig. 2.20 is applied to this problem, the resulting design is as shown in Fig. 2.24. The vertical connection has been reduced to only 14 bolts and a ½-in end plate.

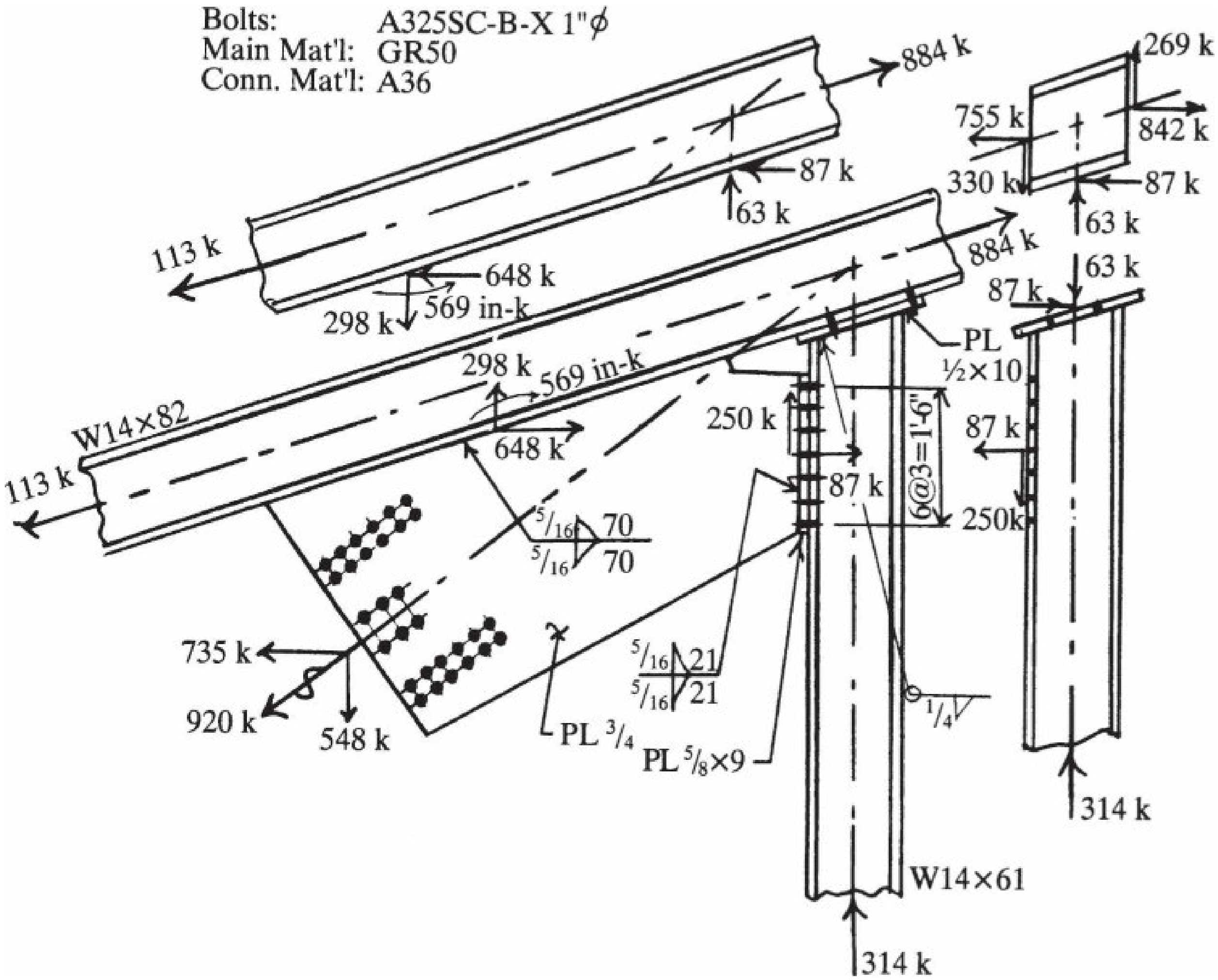


FIGURE 2.24 Uniform force method.

The designs of Figs. 2.21, 2.23, and 2.24 are all satisfactory for some admissible force system. For instance, the design of Fig. 2.21 will be satisfactory for the force systems of Figs. 2.23 and 2.24, and the design of Fig. 2.23 will be satisfactory for the force system of Fig. 2.24. How can it be determined which is the "right" or "best" admissible force system to use? The lower bound theorem of limit analysis provides an answer. This theorem basically says that for a given connection configuration, that is, Figs. 2.21, 2.23, or 2.24, the statically admissible force distribution that maximizes the capacity of the connection is closest to the true force distribution. As a converse to this, for a given load, the smallest connection satisfying the limit states is closest to the true required connection. Of the three admissible force distributions given in Figs. 2.21, 2.23, and 2.24, the distribution of Fig. 2.24, based on the UFM, is the "best" or "right" distribution.

2.2.2.3 A Numerical Example. To demonstrate the calculations required to design the connections of Figs. 2.21, 2.23, and 2.24, for the statically admissible forces of these figures, consider for instance the UFM forces and the resulting connection of Fig. 2.24.

The geometry of Fig. 2.24 is arrived at by trial and error. First, the brace-to-gusset connection is designed, and this establishes the minimum size of gusset. For calculations for this part of the connection, see Sec. 2.2.1.2. Normally, the gusset is squared off as shown in Fig. 2.23, which gives 16 rows of bolts in the gusset-to-truss vertical connection. The gusset-to-top chord connection is pretty well constrained by geometry to be about 70 in long plus about 13½ in for the cutout. Starting from the configuration of Fig. 2.23, the UFM forces are calculated from the formulas of Fig. 2.20 and the design is checked. It will be found that Fig. 2.23 is a satisfactory design via the UFM, even though it fails via the KISS method forces of Fig. 2.21. Although the gusset-to-top chord connection cannot be reduced in length because of geometry, the gusset-to-truss vertical is subject to no such constraint. Therefore, the number of rows of bolts in the gusset-to-truss vertical is sequentially reduced until failure occurs. The last-achieved successful design is the final design as shown in Fig. 2.24.

The calculations for Fig. 2.24 and the intermediate designs and the initial design of Fig. 2.23 are performed in the following manner. The given data for all cases are

$$P = 920 \text{ kips}$$
$$e_B = 7 \text{ in}$$
$$e_C = 7 \text{ in}$$
$$\gamma = 17.7°$$
$$\theta = 36.7°$$

The relationship between α and β is

$$\alpha - \beta(0.9527 \times 0.7454 - 0.3040) = 7(0.7454 - 0.3191) - 7/0.9527$$
$$\alpha - 0.4061\beta = -4.363$$

This relationship must be satisfied for there to be no couples on the gusset edges. For the configuration of Fig. 2.24 with seven rows of bolts in the gusset-to-truss vertical connection (which is considered the gusset-to-beam connection of Fig. 2.20) $\bar{\alpha} = 18.0$ in. Then,

$$\beta = \frac{18 + 4.363}{0.4061} = 55.07 \text{ in}$$

From Fig. 2.24, the centroid of the gusset to top chord (which is the gusset-to-column connection of Fig. 2.20) is $\bar{\beta} = 13.5 + 70/2 = 48.5$ in. Since $\bar{\beta} \neq \beta$, there will be a couple on this edge unless the gusset geometry is adjusted to make $\bar{\beta} = \beta = 55.07$. In this case, we will leave the gusset geometry unchanged and work with the couple on gusset-to-top chord interface.

Rather than choosing $\bar{\alpha} = 18.0$ in, we could have chosen $\bar{\beta} = 48.5$ and solved for $\alpha \neq \bar{\alpha}$. In this case, a couple will be required on the gusset-to truss vertical interface unless gusset geometry is changed to make $\alpha = \bar{\alpha}$.

Of the two possible choices, the first is the better one because the rigidity of the gusset-to-top chord interface is much greater than that of the gusset-to-truss vertical interface. This is so because the gusset is direct welded to the center of the top chord flange and is backed up by the chord web, whereas the gusset-to-truss vertical involves a flexible end plate and the bending flexibility of the flange of the truss vertical. Thus, any couple required to put the gusset in equilibrium will tend to migrate to the stiffer gusset-to-top chord interface.

With $\alpha = 18.0$ and $\beta = 55.07$,

$$r = [(18.0 + 7 \times 0.3191 + 55.07 \times 0.3040 + 7/0.9527)^2 + (7 + 55.07 \times 0.9527)^2]^{1/2}$$
$$= 74.16 \text{ in}$$

and from the equations of Fig. 2.20,

$$V_C = 648 \text{ kips}$$
$$H_c = 298 \text{ kips}$$
$$V_B = 87 \text{ kips}$$
$$H_B = 250 \text{ kips}$$

For subsequent calculations, it is necessary to convert the gusset-to-top chord forces to normal and tangential forces as follows: the tangential or shearing component is

$$T_C = V_C \cos\gamma + H_C \sin\gamma = (\beta + e_C \tan\gamma)\frac{P}{r}$$

The normal or axial component is

$$N_C = H_C \cos\gamma - V_C \sin\gamma = \frac{e_C P}{r}$$

The couple on the gusset-to-top chord interface is then

$$M_C = |N_C(\beta - \bar{\beta})|$$

Thus

$$T_C = (55.07 + 7 \times .3191)\frac{920}{74.16} = 711^{\text{k}}$$

$$N_C = 7 \times \frac{920}{74.16} = 86.6^{\text{k}}$$

$$M_C = 86.6 \times (55.07 - 48.5) = 569 \text{ kips-in}$$

Each of the connection interfaces will now be designed.

1. *Gusset to top chord.*
   a. *Weld*: Weld length is 70 in.

$$f_v = \frac{711}{2 \times 70} = 5.08 \text{ kips/in}$$

$$f_a = \frac{86.6}{2 \times 70} = 0.61 \text{ kips/in}$$

$$f_b = \frac{569 \times 2}{70^2} = 0.23 \text{ kips/in}$$

$$f_R = \sqrt{(5.08)^2 + (0.61 + 0.23)^2} = 5.1 \text{ kips/in}$$

$$D = \frac{5.1}{1.392} = 3.7\ 16^{\text{ths}}$$

Check ductility

$$f_{\text{ave}} = \sqrt{5.08^2 + \left(0.61 + \frac{0.23}{2}\right)^2} = 5.1 \text{ kips/in}$$

Since $1.25 \times f_{\text{ave}} = 6.4 > 5.1$, size weld for ductility requirement

$$D = \frac{6.4}{1.392} = 4.60$$

Use 5/16 fillet weld.

b. *Gusset stress:*

$$f_v = \frac{711}{0.75 \times 70} = 13.5 \text{ ksi} < 21.6 \text{ ksi, ok}$$

$$f_a + f_b = \frac{86.6}{0.75 \times 70} + \frac{569 \times 4}{0.75 \times 70^2} = 2.27 \text{ ksi} < 32.4 \text{ ksi, ok}$$

c. *Top chord web yield:* The normal force between the gusset and the top chord is T_c = 86.6 kips and the couple is M_c = 569 kips-in. The contact length N is 70 in. The couple M_c is statically equivalent to equal and opposite normal forces $V_s = M_c/(N/2) = 569/35 = 16.2$ kips. The normal force V_s acts over a contact length of $N/2$ = 35 in. For convenience, an equivalent normal force acting over the contact length N can be defined as

$$N_{\text{C,equiv.}} = N_C + 2 \times V_s = 86.6 + 2 \times 16.2 = 119 \text{ kips}$$

Now, for web yielding

$$\phi R_{wy} = 1.0 \times (5k + N)\, F_{yw} t_w = 1.0 \times (5 \times 1.625 + 70) \times 50 \times 0.510$$
$$= 1992 \text{ kips} > 119 \text{ kips, ok}$$

d. *Top chord web crippling:*

$$\phi R_{wcp} = 0.75 \times 0.8 \times t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

$$= 0.75 \times 0.8 \times .510^2 \left[1 + 3\left(\frac{70}{14.3}\left(\frac{0.510}{0.855}\right)^{1.5}\right)\right]$$

$$\times \sqrt{\frac{29{,}000(50)(0.855)}{0.510}} = 1890 \text{ kips} > 119 \text{ kips, ok}$$

In the web crippling check, the formula used is that for a location greater than $d/2$ from the chord end because $\bar{\beta} = 13.5 + 70/2 = 48.5$ in > 14.31/2 = 7.2 in. $\bar{\beta}$ is the position of the equivalent normal force. Additionally, the restraint provided by the beam end connection is sufficient to justify the check away from the end of the beam.

The checks for web yield and crippling could have been dismissed by inspection in this case, but were completed to illustrate the method. Another check that should be made when there is a couple acting on a gusset edge is to ensure that the transverse shear induced on the supporting member, in this case the top chord W14 × 82, can be sustained. In this case, the induced transverse shear is V_s = 16.2 kips. The shear capacity of the W14 × 82 is 0.510 × 14.3 × 1.0 × 0.6 × 50 = 219 kips > 16.2 kips, ok. Now consider for contrast, the couple of 15,860 kips-in shown in Fig. 2.21. For this couple, V_s = 15860/35 = 453 kips > 219 kips, so a ¾ in. doubler plate of GR50 steel is required as shown in Fig. 2.21.

2. *Gusset to truss vertical:*

a. *Weld:*

$$f_v = \frac{250}{2.21} = 5.95 \text{ kips/in}$$

$$f_a = \frac{87}{2 \times 21} = 2.07 \text{ kips/in}$$

$$f_R = \sqrt{5.95^2 + 2.07^2} = 6.30 \text{ kips/in}$$

Fillet weld size required $= \frac{6.30}{1.392} = 4.5\ 16^{\text{ths}}$

Because of the flexibility of the end plate and truss vertical flange, there is no need to size the weld to provide ductility. Therefore, use a 5/16-fillet weld.

b. *Bolts and end plate:* The bolts are A325SC-B-X, 1″ϕ in standard holes designed to the serviceability level, as is the default in the specification. The end plate is 9 in wide and the gage of the bolts is 5½ in. Thus, using the prying action formulation notation of the AISC 15th Manual (2016). The slip resistance per bolt can be calculated as

$$\phi R_n = \phi \mu D_u h_{sc} T_b = 1.0(0.50)(1.13)(1.0)(51) = 28.8 \text{ kips}$$

The tensile strength per bolt, excluding prying, from Table 7-2 is 53.0 kips.

$$b = \frac{5.5 - 0.75}{2} = 2.375$$

$$a = \frac{9 - 5.5}{2} = 1.75 < 1.25 \times 2.375 \text{ ok}$$

$$b' = 2.375 - 0.5 = 1.875$$

$$a' = 1.75 + 0.5 = 2.25$$

$$\rho = \frac{1.875}{2.25} = 0.833$$

$$\delta = 1 - \frac{1.0625}{3} = 0.646$$

Shear per bolt = V = 250/14 = 17.9 < 28.8 kips, ok. Tension per bolt = T = 87/14 = 6.21 kips.

The tension force will reduce the clamping force and therefore the slip resistance. The slip strength considering the applied tension is

$$k_{sc} = 1 - \frac{T_u}{D_u T_b n_b} = 1 - \frac{6.21}{1.13 \times 51} = 0.892$$

$$\varphi V_v = 0.892 \times 28.8 = 25.7 \text{ kips/bolt} > 17.9 \text{ kips/bolt}$$

While applied tension affects slip-critical connection shear strength, as shown above, applied shear does not affect the tensile strength of the bolt in quite the same manner. The reason for this lies in the physical behavior of slip-critical connections. Connection shear is carried by the faying surface through friction until slip occurs. Thus, the bolt itself "sees" no shear until the connection slips, and its tensile strength is consequently unaffected until slip. Once slip occurs, the connection will behave as a bearing connection and should be checked in this manner.

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\varphi F_{nv}} f_{rv} = 1.3 \times 90 - \frac{90}{0.75 \times 68}\left(\frac{17.9}{\pi \times 0.5^2}\right) = 76.8 \text{ ksi}$$

$$B = 0.75 \times 76.8 \times \pi \times 0.5^2 = 45.2 > 6.21 \text{ kips, ok}$$

Try ½ plate

$$t_c = \sqrt{\frac{4.44 \times 45.2 \times 1.875}{3 \times 58}} = 1.47$$

$$\alpha' = \frac{1}{0.646 \times 1.833}\left[\left(\frac{1.47}{0.5}\right)^2 - 1\right] = 6.47$$

Use α′ = 1

$$T_d = 45.2\left(\frac{0.5}{1.47}\right)^2 \times 1.833 = 6.57 \text{ kips} > 6.21 \text{ kips ok}$$

Use the ½-in plate for the end plate.

c. *Truss vertical flange:* The flange thickness of the W14 × 61 is 0.645 in which exceeds the end plate thickness as well as being Grade 50 steel. The truss vertical flange is therefore, ok by inspection, but a calculation will be performed to demonstrate how the flange can be checked. A formula (Mann and Morris, 1979) for an effective bolt pitch can be derived from yield-line analysis as

$$p_{\text{eff}} = \frac{p(n-1) + \pi\bar{b} + 2\bar{a}}{n}$$

where the terms are as previously defined in Fig. 2.11. For the present case

$$\bar{b} = \frac{5.5 - 0.375}{2} = 2.5625$$

$$\bar{a} = \frac{10 - 5.5}{2} = 2.25$$

$$n = 7$$

$$p = 3$$

$$p_{\text{eff}} = \frac{3 \times (7-1) + \pi \times 2.5625 + 2 \times 2.25}{7} = 4.36$$

Once p_{eff} is determined, the prying action theory of the AISC Manual is applied.

$$b = \bar{b} = 2.5625$$
$$b' = 2.5625 - 0.5 = 2.0625$$

a = smaller of $\bar{a}$ and a for the end plate = 1.75 < 1.25 × 2.5625 ok

$$a' = 1.75 + 0.5 = 2.25$$

$$\rho = \frac{b'}{a'} = 0.917$$

$$\delta = 1 - 1.0625/4.36 = 0.756$$

$$t_c = \sqrt{\frac{4.44 \times 45.2 \times 2.0625}{4.36 \times 65}} = 1.21 \text{ in}$$

$$\alpha' = \frac{1}{0.756 \times 1.917}\left[\left(\frac{1.21}{0.645}\right)^2 - 1\right] = 1.74, \text{ therefore use } 1.0$$

$$T_d = 45.2\left(\frac{0.645}{1.21}\right)^2 \times (1 + 1 \times 0.756)$$

$$= 22.6 \text{ kips} > 6.21 \text{ kips, ok}$$

3. *Truss vertical-to-top chord connection:*

The forces on this connection, from Figs. 2.20 and 2.24 are

$$\text{Vertical} = Q = 298 - 920 \times \cos(36.7) \times \tan(17.7) = 63 \text{ kips}$$

Horizontal = 87 kips

Converting these into normal and tangential components

$$T_{BC} = 87 \cos\gamma - 63 \sin\gamma = 64 \text{ kips}$$
$$N_{BC} = 87 \sin\gamma + 63 \cos\gamma = 86 \text{ kips (compression)}$$

a. *Bolts:* Since the normal force is always compression, the bolts see only the tangential or shear force; thus, the number of bolts required is

$$\frac{64}{28.8} = 2.2 \text{ use 4 bolts}$$

b. *Weld:* Use a profile fillet weld of the cap plate to the truss vertical, but only the weld to the web of the vertical is effective because there are no stiffeners between the flanges of the top chord. Thus, the effective length of weld is

$$(13.89 - 2 \times 0.645)/\cos\gamma = 13.23 \text{ in}$$

$$f_v = \frac{64}{2 \times 13.23} = 2.42 \text{ kips/in}$$

$$f_a = \frac{86}{2 \times 13.23} = 3.25 \text{ kips/in}$$

$$f_R = \sqrt{2.42^2 + 3.25^2} = 4.05 \text{ kips/in}$$

The weld size required is $\frac{4.05}{1.392} = 2.91$

Use ¼ (AISC minmum size).

Check the W14 × 61 web to support required 2.91 16ths FW. For welds of size *W* on both sides of a web of thickness t_w

$$1.0 \times 0.6 \times F_y t_w \geq 0.75 \times 0.60 \times 70 \times 0.7071 \times \mathrm{W} \times 2$$

or

$$t_w \geq 1.48 \text{ W for grade 50 steel}$$

Thus for W = 2.91/16 = 0.182

$$t_{w_{min}} = 1.48 \times 0.182 = 0.269 \text{ in}$$

Since the web thickness of a W14 × 61 is 0.375, the web can support the welds.

c. *Cap plate:*

The cap plate thickness will be governed by bearing. The bearing design strength per bolt is

$$\phi r_p = 0.75 \times 2.4 \times 58 \times t_p \times 1$$

The load per bolt is 64/4 = 16.0 kips. The required cap plate thickness is thus

$$t_p = \frac{16.0}{0.75 \times 2.4 \times 58 \times 1} = 0.153$$

Use a ½-in cap plate.

This completes the calculations required to produce the connection of Fig. 2.24.

2.2.3 Hanger Connections

The most interesting of the genre is the type that involves prying action, sometimes of both the connection fitting and the supporting member. Figure 2.25 shows a typical example. The calculations to determine the capacity of this connection are as follows: The connection can be broken into three main parts, that is, the angles, the piece W16 × 57, and the supporting member, the W18 × 50. The three main parts are joined by two additional parts, the bolts of the angles to the piece W16 and the bolts from the piece W16 to the W18. The load path in this connection is unique. The load P passes from the angles through the bolts into the piece W16, thence through bolts again into the supporting W18. The latter bolt group is arranged to straddle the brace line of action. These bolts then see only direct tension and shear, and no additional tension due to moment. Statics is sufficient to establish this. Consider now the determination of the capacity of this connection.

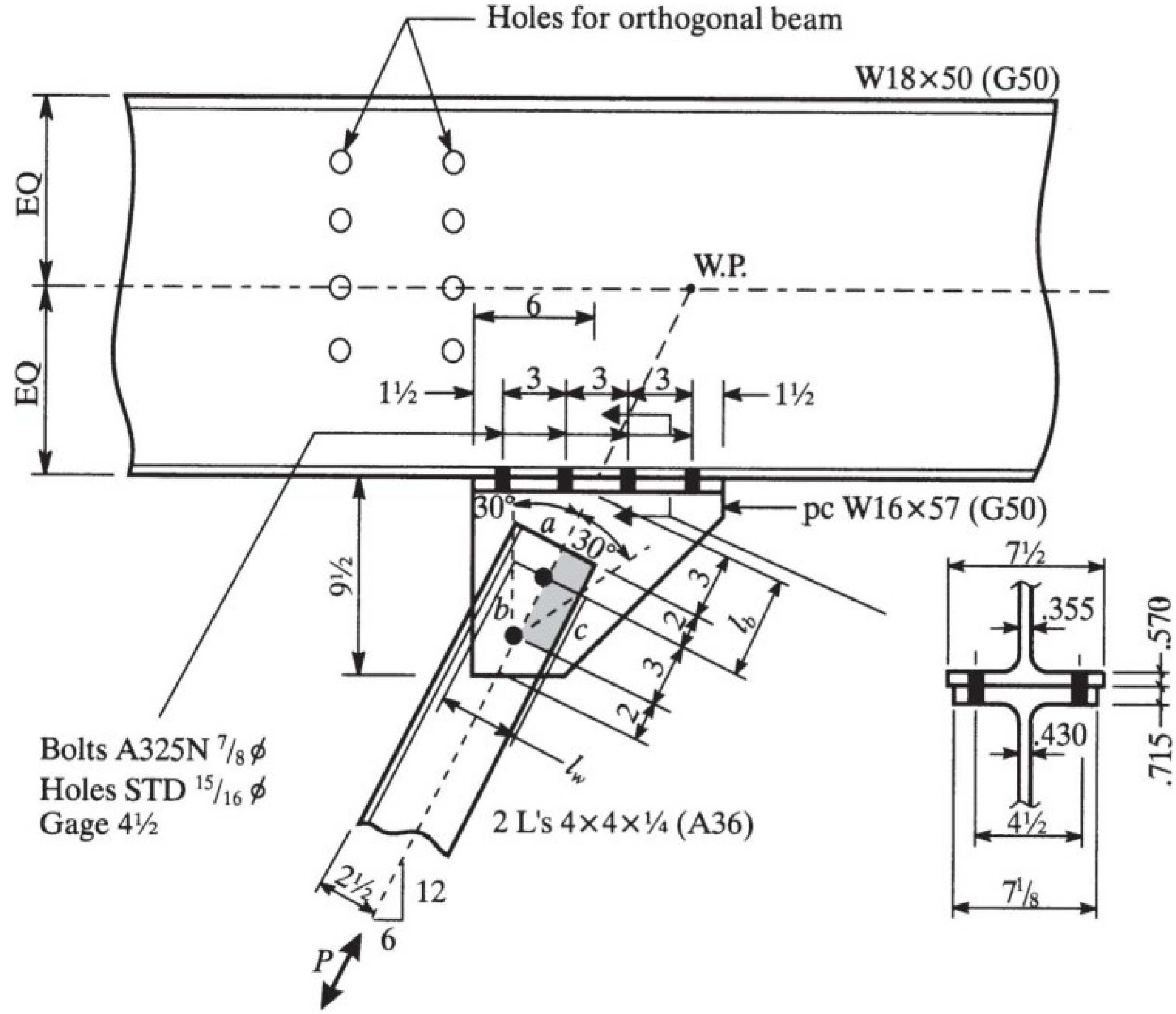


FIGURE 2.25 Typical bolted hanger connection.

1. *Angles:* The limit states for the angles are gross tension, net tension, block shear rupture, and bearing. The load can be compression as well as tension in this example. Compression will affect the angle design, but tension will control the above limit states.

   a. *Gross tension:* The gross area A_{gt} is 1.94 × 2 = 3.88 in^2. The capacity (design strength) is

$$\phi R_{gt} = 0.9 \times 36 \times 3.88 = 126 \text{ kips}$$

   b. *Net tension:* The net tension area is $A_{nt} = 3.88 - 0.25 \times 1.0 \times 2 = 3.38$ in^2. The effective net tension area A_e is less than the net area because of shear lag since only one of the two angle legs is connected. From the AISC Specification (2016) Section D3.3

$$U = \max(0.60, 1 - 1.09/3) = 0.637$$
$$A_e = U \times A_{nt} = 0.637 \times 3.38 = 2.15$$

The net tension capacity is

$$\phi R_{nt} = 0.75 \times 58 \times 2.15 = 93.5 \text{ kips}$$

c. *Block shear rupture:* This failure mode involves the tearing out of the cross-hatched block in Fig. 2.25. The failure is by yield on the longitudinal line through the bolts (line ab) and a simultaneous fracture failure on the perpendicular line from the bolts longitudinal line to the angle toe (line bc).

Because yield on the longitudinal section may sometimes exceed fracture on this section, the AISC Specification J4.3 limits the strength to the lesser of the two. Thus, the block shear limit state is

$$\phi R_{bs} = 0.75[U_{bs}F_uA_{nt} + \min\{0.6F_yA_{gv}, 0.6F_uA_{nv}\}]$$

where the terms will be defined in the following paragraphs.

For line ab, the gross shear area is

$$A_{gv} = 5 \times 0.25 \times 2 = 2.5 \text{ in}^2$$

and the net shear area is

$$A_{nv} = 2.5 - (1.5 \times 0.25 \times 1.0)2 = 1.75 \text{ in}^2$$

For line bc, the gross tension area is

$$A_{gt} = 1.5 \times 0.25 \times 2 = 0.75 \text{ in}^2$$

and the net tension area is

$$A_{nt} = 0.75 - 0.5 \times 1.0 \times 0.25 \times 2 = 0.5 \text{ in}^2$$

The term U_{bs} accounts for the fact that for highly eccentric connections, the tension force distribution on section bc will not be uniform. In this case, U_{bs} is taken as 0.5. In the present case, the force distribution is essentially uniform because the angle gage line and the angle gravity axis are close to each other. Thus $U_{bs} = 1.0$, and the block shear strength is

$$\phi R_{bs} = 0.75[1.0 \times 58 \times 0.5 + \min\{0.6 \times 36 \times 2.5, 0.6 \times 58 \times 1.75\}] = 62.2 \text{ kips}.$$

d. *Shear/bearing/tearout on bolts and parts:*

Bearing, tearout, and bolt shear are inextricably tied to each bolt. Therefore, it is no longer possible to check bolt shear for the bolt group as a whole, and bearing/tearout for each part separately, and then to take the minimum of these limit states as the controlling limit state. The procedure is as follows for each bolt. For the upper bolt, the limit states are

1. bolt shear $\phi R_v = 0.75 \times 54 \times \pi/4 \times 0.875^2 \times 2 = 48.7$ kips
2. bearing on angles $\phi R_p = 0.75 \times 2.4 \times 0.875 \times 2 \times 0.25 \times 5 = 45.7$ kips
3. bearing on W16 × 57 $\phi R_p = 0.75 \times 2.4 \times 0.875 \times 0.43 \times 65 = 44.0$ kips
4. tearout on angles $\phi R_{to} = 0.75 \times 1.2(2 - 0.5 \times 0.9375) \times 2 \times 0.25 \times 58 = 40.0$ kips
5. tearout on W16 × 57 $\phi R_{to} = 0.7\ 5 \times 1.2(3 - 0.9375) \times 0.430 \times 65 = 51.9$ kips

The shear/bearing/tearout of the upper bolt is thus 40.0 kips.

For the lower bolt, the limit states are

1. bolt shear $\phi R_v = 48.7$ kips
2. bearing on the angles $\phi R_p = 45.7$ kips
3. bearing on the W16 × 57 $\phi R_p = 44.0$ kips
4. tearout on the angles $\phi R_{to} = 0.75 \times 1.2(3 - 0.9375) \times 2 \times 0.25 \times 58 = 53.8$ kips
5. tearout on the W15 × 57 $\phi R_{to} = 0.75 \times 1.2(2 - 0.5 \times 0.9375) \times 0.430 \times 65 = 38.5$ kips

The shear/bearing/tearout strength of the lower bolt is thus 38.5 kips, and the capacity of the connection in these limit states is $R_{vp} = 40.0 \times 1 + 38.5 \times 1 = 78.5$ kips.

2. *Bolts—angles to piece W16:* The limit state for the bolts is shear. The shear capacity of one bolt is $\phi r_v = 0.75 \times 54 \times \pi/4 \times 0.875^2 = 24.4$ kips.

In this case, the bolts are in double shear and the double shear value per bolt is 24.4 × 2 = 48.8 kips/bolt. Note that because of bearing limitations, this value cannot be achieved. The bolt shear strength is limited by the bearing strength of the parts; thus the bolt shear strength is equal to the bearing strength, so

$$\phi R_y = \phi R_p = 78.5 \text{ kips}$$

3. *Piece W16 × 57:* The limit states for this part of the connection are Whitmore section yield and buckling, bearing, and prying action in conjunction with the W16 flange to W18 flange bolts. Because there is only one line of bolts, block shear is not a limit state. Bearing has already been considered with the angle checks.

a. *Whitmore section*: This is the section denoted by l_w on Fig. 2.25. It is formed by 30° lines from the bolt furthest away from the end of the brace to the intersection of these lines with a line through and perpendicular to the bolt nearest to the end of the brace.

Whitmore (1952) determined that this 30° spread gave an accurate estimate of the stress in gusset plates at the end of the brace. The length of the Whitmore section l_w = 3(tan 30°)2 = 3.46 in.

(1) Whitmore yield:

$$\phi R_{wy} = 0.9 \times 50 \times 3.46 \times 0.430 = 67.0 \text{ kips}$$

where 0.430 is the web thickness of a W16 × 57.

(2) Whitmore buckling:

Tests (Gross, 1990; Dowswell, 2006) have shown that the Whitmore section can be used as a conservative estimate for gusset buckling. In the present case, the web of the W16 × 57 is a gusset. If the load P is a compression, it is possible for the gusset to buckle laterally in a sidesway mode. For this mode of buckling, the K factor is 1.2. The buckling length is l_b = 5 in in Fig. 2.25. Thus the slenderness ratio is

$$\frac{Kl}{r} = \frac{1.2 \times 5 \times \sqrt{12}}{0.430} = 48.3$$

Since $Kl/r > 25$, Section J4.4 on strength of elements in compression does not apply; the column buckling equations of Chapter E apply. Thus, from Section E3,

$$F_e = \frac{\pi^2 \times 29{,}000}{(28.3)^2} = 123$$

$$F_{cr} = \left[0.658\left(\frac{50}{123}\right)\right] 50 = 42.2 \text{ ksi}$$

$$\phi F_{cr} = 0.9 \times 42.2 = 38.0 \text{ ksi}$$

$$\text{and } \phi R_{wb} = 38.0 \times 3.46 \times 0.430 = 56.5 \text{ kips}$$

b. *Bearing:* This has been considered with the angles, above.

c. *Prying action:* Prying action explicitly refers to the extra tensile force in bolts that connect flexible plates or flanges subjected to loads normal to the flanges. For this reason, prying action involves not only the bolts but the flange thickness, bolt pitch and gage, and in general, the geometry of the entire connection.

The AISC LRFD Manual presents a method to calculate the effects of prying. This method was originally developed by Struik and deBack (1969) and presented in the book (Kulak et al., 1987). The form used in the AISC LRFD Manual was developed by Thornton (1985), for ease of calculation and to provide optimum results, that is, maximum capacity for a given connection (analysis) and minimum required thickness for a given load (design). Thornton (1992, 1997) has shown that this method gives a very conservative estimate of ultimate load and shows that very close estimates of ultimate load can be obtained by using the flange ultimate strength, F_u, in place of yield strength, F_y, in the prying action formulas. More recently, Swanson (2002) has confirmed Thornton's (1992, 1997) results with modern materials. For this reason, the AISC Manual now uses F_u in place of F_y in the prying action formulas. Note that the resistance factor, f, used with the F_u is 0.90, because the flange failure mode is yielding with strain hardening rather than fracture.

From the foregoing calculations, the capacity (design strength) of this connection is 56.5 kips. Let us take this as the design load (required strength) and proceed to the prying calculations. The vertical component of 56.5 is 50.5 kips and the horizontal component is 25.3 kips. Thus, the shear per bolt is $V = 25.3/8 = 3.16$ kips and the tension per bolt is $T = 50.5/8 = 6.31$ kips. Since 3.16 < 24.4, the bolts are ok for shear. Note that the bolts also need to be checked for bearing as was done for the angles. In this case, bearing is seen to be "ok by inspection." The interaction equation for A325 *N* bolts is

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \leq F_{nt}$$

were F_{nt} = bolt nominal tensile strength = 90 ksi
F_{nv} = bolt nominal shear strength = 54 ksi
$\phi = 0.75$
f_v = the required shear strength per bolt

With $V = 3.16$ kips/bolt, $f_v = 3.16/0.6013 = 5.26$ ksi, and

$$F'_{nt} = 1.3 \times 90 - \frac{90}{0.75 \times 54} \times 5.26 = 105 \text{ ksi, use } F'_{nt} = 90 \text{ ksi.}$$

Now, the design tensile strength per bolt is

$\phi r'_t = 0.75 \times 90 \times 0.6013 = 40.6$ kips is greater than the required strength (or load) per bolt T = 6.31 kips, the bolts are ok.

Now, to check prying of the W16 piece, following the notation of the AISC Manual,

$$b = \frac{4.5 - 0.430}{2} = 2.035$$

$$a = \frac{7.125 - 4.5}{2} = 1.3125$$

Check that $a < 1.25b = 1.25 \times 2.035 = 2.544$. Since $a = 1.3125 < 2.544$, use a = 1.3125. If $a > 1.25b$, $a = 1.25b$ would be used.

$$b' = 2.035 - 0.875/2 = 1.598$$

$$a' = 1.3125 + 0.875/2 = 1.75$$

$$\rho = \frac{b'}{a'} = 0.91$$

$$p = 3$$

$$\delta = 1 - d'/p = 1 - 0.9375/3 = 0.6875$$

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

$$t_c = \sqrt{\frac{4.44(\phi r_{t'})b'}{pF_u}} = \sqrt{\frac{4.44 \times 40.6 \times 1.598}{3 \times 65}} = 1.215$$

$$\alpha' = \frac{1}{0.6875 \times 1.91}\left[\left(\frac{1.215}{0.715}\right)^2 - 1\right] = 1.44$$

Since $\alpha' > 1$, use $\alpha' = 1$ in subsequent calculations. $\alpha' = 1.44$ means that the bending of the W16 × 57 flange will be the controlling limit state. The bolts will not be critical, that is, the bolts will not limit the prying strength. The design tensile strength T_d per bolt including the flange strength is

$$T_d = \phi r_t'\left(\frac{t}{t_c}\right)^2(1+\delta) = 40.6\left(\frac{0.715}{1.215}\right)^2 1.6875$$

$$= 23.7 \text{ kips} > 5.96 \text{ kips, ok}$$

The subscript *d* denotes “design” strength.

In addition to the prying check on the piece W16 × 57, a check should also be made on the flange of the W18 × 50 beam. A method for doing this was presented in Fig. 2.11. Thus,

$$\bar{b} = \frac{4.5 - 0.355}{2} = 2.073$$

$$\bar{a} = \frac{7.5 - 4.5}{2} = 1.50$$

$$n = 4$$

$$p = 3$$

$$p_{\text{eff}} = \frac{3(4-1) + \pi \times 2.073 + 2 \times 1.50}{4} = 4.63$$

Now, using the prying formulation from the AISC Manual,

$$b = \bar{b} = 2.073$$
$$a = 1.3125$$

Note that the prying lever arm is controlled by the narrower of the two flanges.

$$b' = 2.073 - 0.875/2 = 1.636$$
$$a' = 1.3125 + 0.875/2 = 1.75$$
$$\rho = 0.93$$
$$p = p_{\text{eff}} = 4.63$$
$$\delta = 1 - 0.9375/4.63 = 0.798$$
$$t_c = \sqrt{\frac{4.44 \times 40.6 \times 1.636}{4.63 \times 50}} = 0.990$$
$$\alpha' = \frac{1}{0.798 \times 1.93}\left[\left(\frac{0.990}{0.570}\right)^2 - 1\right] = 1.31$$

Use $\alpha' = 1$

$$T_d = 40.6\left(\frac{0.570}{0.990}\right)^2 1.798 = 24.2^k > 5.96^k \quad \text{ok}$$

Additional checks on the W18 × 50 beam are for web yielding. Since $5k = 5 \times 1.25 = 6.25 > p = 3$, the web tributary to each bolt at the k distance exceeds the bolt spacing and thus $N = 9$.

$\phi R_{wy} = 1.0 \times (9 + 5 \times 1.25) \times 50 \times 0.355 = 271$ kips > 50.5 kips, ok, and for web crippling, web crippling occurs when the load is compression, thus $N = 12$, the length of the piece W16.

$$\phi R_{wcp} = 0.75 \times 0.80 \times 0.355^2\left[1 + 3\left(\frac{12}{18.0}\right)\left(\frac{0.355}{0.570}\right)^{1.5}\right]$$
$$\times \sqrt{\frac{29{,}000 \times 50 \times 0.570}{0.355}} = 229 \text{ kips} > 50.5 \text{ kips, ok}$$

This completes the design calculations for this connection. A load path has been provided through every element of the connection. For this type of connection, the beam designer should make sure that the bottom flange is stabilized if P can be compressive. A transverse beam framing nearby as shown in Fig. 2.25 by the W18 × 50 web hole pattern, or a bottom flange stay (kicker), will provide stability.

2.2.4 Column Base Plates

The geometry of a column base plate is shown in Fig. 2.26. The area of the base plate is $A_1 = B \times N$. The area of the pier that is concentric with A_1 is A_2. If the pier is not concentric with the base plate, only the portion that is concentric can be used for A_2. The design strength of the concrete in bearing is

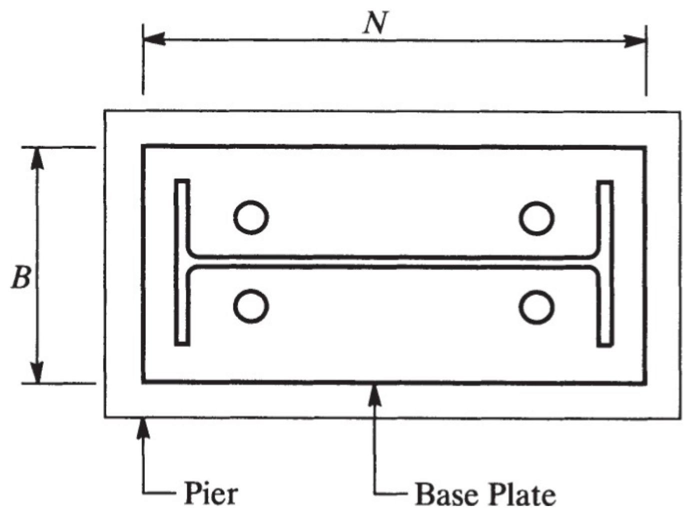


FIGURE 2.26 Column base plate.

$$\phi_c F_p = 0.6 \times 0.85 f'_c \sqrt{\frac{A_2}{A_1}}$$

where f'_c is the concrete compressive strength in ksi and

$$1 \le \sqrt{\frac{A_2}{A_1}} \le 2$$

The required bearing strength is

$$f_p = \frac{P}{A_1}$$

where P is the column load (factored) in kips. In terms of these variables, the required base plate thickness is

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

where $l = \max\{m, n, \lambda n'\}$

ϕF_y = base plate design strength = $0.9\ F_y$

$$m = \frac{N - 0.95d}{2}$$

$$n = \frac{B - 0.8b_f}{2}$$

$$n' = \frac{\sqrt{db_f}}{4}$$

$$\lambda = \frac{2\sqrt{x}}{1 + \sqrt{1 - x}} \le 1$$

$$x = \frac{4db_f}{(d + b_f)^2} \frac{f_p}{\phi_c F_p}$$

d = depth of column

b_f = flange width of column

For simplicity, λ can always be conservatively taken as unity. The formulation given here was developed by Thornton (1990a, 1990b) based on previous work by Murray (1983), Fling (1970), and Stockwell (1975). It is the method given in the AISC Manual (2016).

Example The column of Fig. 2.26 is a W24 × 84 carrying 600 kips. The concrete has $f'_c = 4.0$ ksi. Try a base plate of A36 steel, 4 in bigger than the column in both directions. Since d = 24⅛ and b_f = 9, N = 24⅛ + 4 = 28⅛, b = 9 + 4 = 13. Try a plate 28 × 13. Assume that 2 in of grout will be used, so the minimum pier size is 32 × 17. Thus A_1 = 28 × 13 = 364 in^2, A_2 = 32 × 17 = 544 in^2, $\sqrt{A_2/A_1} = 1.22 < 2$ (ok), and

$$\phi_c F_p = 0.6 \times 0.85 \times 4 \times 1.22 = 2.49 \text{ ksi}$$

$$f_p = \frac{600}{364} = 1.65 \text{ ksi} < 2.49 \text{ ksi, ok}$$

$$m = \frac{28 - 0.95 \times 24.125}{2} = 2.54$$

$$n = \frac{13 - 0.8 \times 9}{2} = 2.90$$

$$n' = \frac{\sqrt{24.125 \times 9}}{4} = 3.68$$

$$x = \frac{4 \times 24.125 \times 9.0}{(24.125 + 9.0)^2} \frac{1.65}{2.49} = 0.52$$

$$\lambda = \frac{2\sqrt{0.52}}{1 + \sqrt{1 - 0.52}} = 0.85$$

$$l = \max\{2.54, 2.90, 0.85 \times 3.68\} = 3.13$$

$$t_p = 3.13\sqrt{\frac{2 \times 1.65}{0.9 \times 36}} = 0.99 \text{ in}$$

Use a plate 1 × 13 × 28 of A36 steel. If the conservative assumption of $\lambda = 1$ were used, $t_p = 1.17$ in, which indicates a 1¼-in-thick base plate.

Erection Considerations. In addition to designing a base plate for the column compression load, loads on base plates and anchor rods during erection should be considered. The latest OSHA requirements postulate a 300 lb. load 18 in off the column flange in the strong axis direction, and the same load 18 in off the flange tips in the weak axis direction. Note these loads would be applied sequentially. A common design load for erection, which is much more stringent than the OSHA load, is a 1-kip working load, applied at the top of the column in any horizontal direction. If the column is, say, 40 ft high, this 1-kip force at a lever arm of 40 ft will cause a significant couple at the base plate and anchor bolts. The base plate, anchor bolts, and column-to-base plate weld should be checked for this construction load condition. The paper by Murray (1983) gives some yield-line methods that can be used for doing this. Figure 2.26 shows four anchor rods. This is an OSHA erection requirement for all columns except minor posts.

2.2.5 Splices—Columns and Truss Chords

Section J1.4 of the AISC Specification (2016) says that finished-to-bear compression splices in columns need be designed only to hold the parts "securely in place." For this reason, the AISC provides a series of "standard" column splices in the AISC *Manual of Steel Construction*. These splices are nominal in the sense that they are designed for no particular loads. Section J1.4 also requires that splices in trusses be designed for at least 50% of the design load (required compression strength), or for the moment and shear resulting from a transverse load equal to 2% of the required compressive strength of the member, whichever is less severe. The difference between columns and "other compression members," such as compression chords of trusses, is that for columns, splices are usually near lateral support points, such as floors, whereas trusses can have their splices at mid-panel points where there is no lateral support. Either the 50% requirement or the 2% requirement can be used to address this situation.

Column Splices. Figure 2.27 shows a standard AISC column splice for a W14 × 99 to a W14 × 109. If the column load remains compression, the strong-axis column shear can be carried by friction. The coefficient of static friction of steel to steel is on the order of 0.5 to 0.7, so quite high shears can be carried by friction. Suppose the compression load on this column is 700 kips. How much major axis bending moment can this splice carry? Even though these splices are nominal, they can carry quite significant bending moment. The flange area of the W14 × 99 is $A_f = 0.780 \times 14.565 = 11.4\ \text{in}^2$. Thus, the compression load per flange is $700 \times 11.4/29.1 = 274$ kips. In order for a bending moment to cause a tension in the column flange, this load of 274 kips must first be unloaded. Assuming that the flange force acts at the flange centroid, the moment in the column can be represented as:

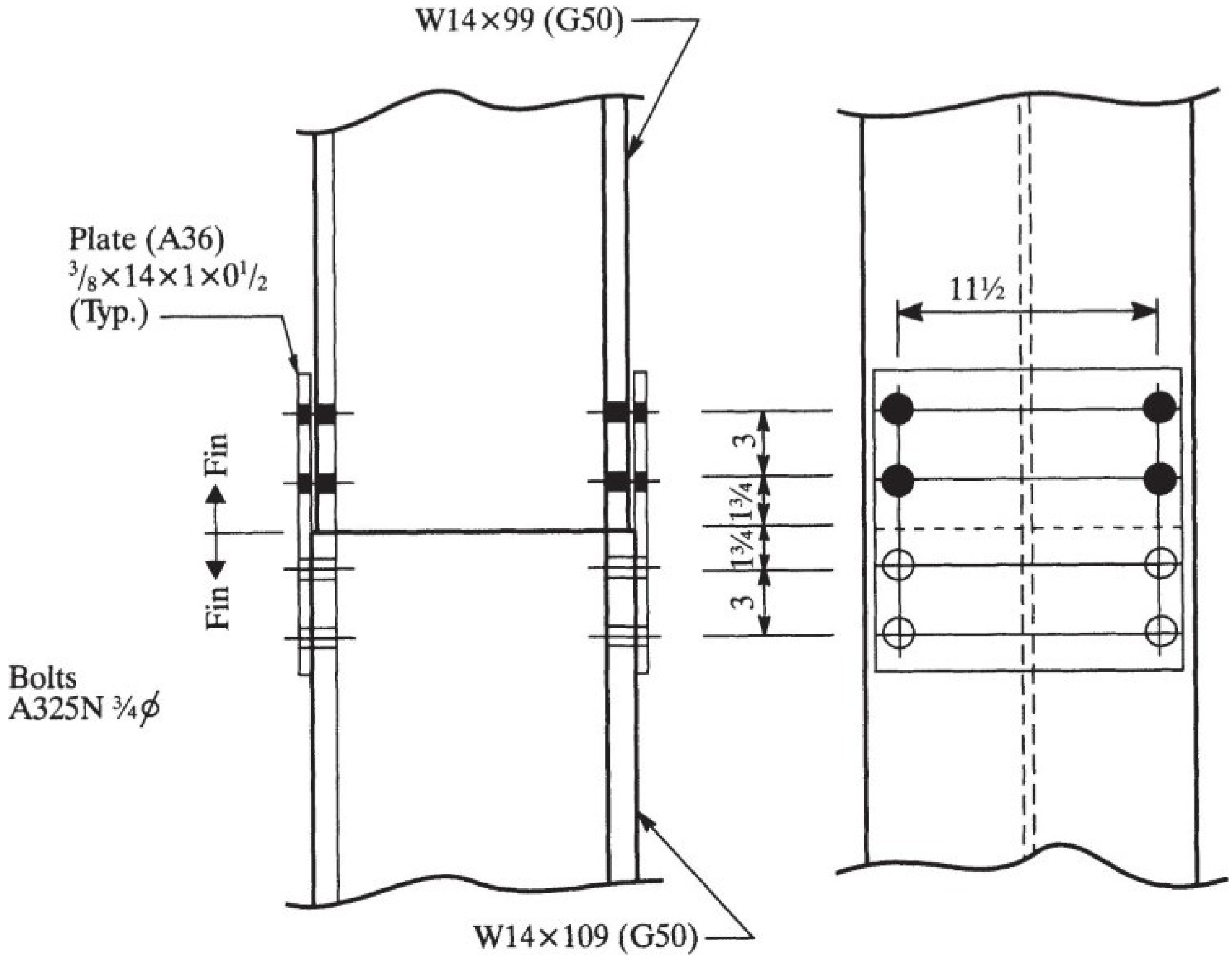


FIGURE 2.27 An AISC standard column splice.

$$M = T(d - t_f) = T(14.16 - 0.780) = 13.38T$$

If T = 274 kips, one flange will be unloaded, and M = 13.38 × 274 = 3666 kips-in = 306 kips-ft. The design strength in bending for this column (assuming sufficient lateral support) is ϕM_p = 647 kips-ft. Thus, because of the compression load, the nominal AISC splice, while still seeing no load, can carry almost 50% of the column's bending capacity.

The splice plates and bolts will allow additional moment to be carried. It can be shown that the controlling limit state for the splice material is bolt shear. For one bolt ϕr_v = 17.9 kips. Thus for 4 bolts ϕR_v = 17.9 × 4 = 71.6 kips. The splice forces are assumed to act at the faying surface of the deeper member. Thus the moment capacity of the splice plates and bolts is M_s = 71.6 × 14.32 = 1030 kips-in = 85.8 kips-ft. The total moment capacity of this splice with zero compression is thus 85.8 kips-ft, and with 700 kips compression, it is 306 + 85.8 = 392 kips-ft. The role of compression in providing moment capability is often overlooked in column splice design.

Erection Stability. As discussed earlier for base plates, the stability of columns during erection must be a consideration for splice design also. The usual nominal erection load for columns is a 1-kip horizontal force at the column top in any direction. In LRFD format, the 1-kip working load is converted to a factored load by multiplying by a load factor of 1.5. This load of 1 × 1.5 = 1.5 kips will require connections that will be similar to those obtained in allowable strength design (ASD) with a working load of 1 kip. It has been established that for major axis bending, the splice is good for 85.8 kips-ft. This means that the 1.5 kip load can be applied at the top of a column 85.8/1.5 = 57.2 ft tall. Most columns will be shorter than 57.2 ft, but if not, a more robust splice should be considered.

Minor Axis Stability. If the 1.5-kip erection load is applied in the minor or weak axis direction, the forces at the splice will be as shown in Fig. 2.28. The upper shaft will tend to pivot about point O. Taking moments about point O,

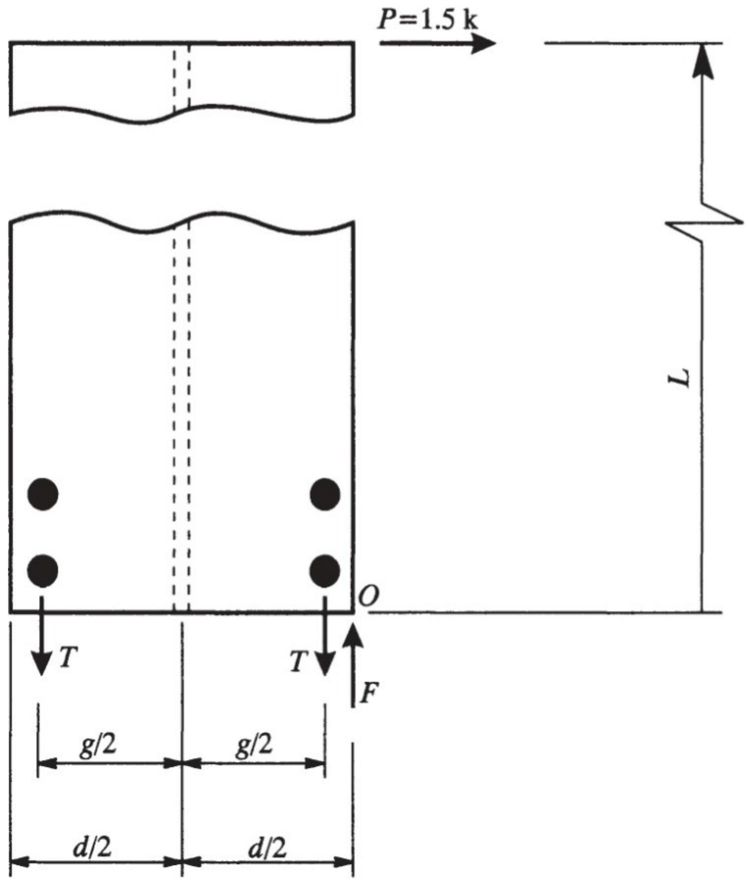


FIGURE 2.28 Weak-axis stability forces for column splice.

$$PL = T\left(\frac{d}{2}+\frac{g}{2}\right)+T\left(\frac{d}{2}-\frac{g}{2}\right)=Td$$

Thus the erection load P that can be carried by the splice is

$$P = Td/L$$

Note that this erection load capacity (design strength) is independent of the gage g. This is why the AISC splices carry the note, “Gages shown can be modified if necessary to accommodate fittings elsewhere on the column.” The standard column gages are 5½ and 7½ in for beams framing to column flanges. Errors can be avoided by making all column gages the same. The gages used for the column splice can also be 5½ or 7½ in without affecting erection stability.

If the upper column of Fig. 2.27 is 40 ft long and T is the shear strength of four (two per splice plate) bolts,

$$P = \frac{4 \times 17.9 \times 14.565}{40 \times 12} = 2.17 \text{ kip}$$

Since 2.17 > 1.5, this splice is satisfactory for a 40-ft-long column. If it were not, larger or stronger bolts could be used.

2.2.5.1 Column Splices for Biaxial Bending. The simplest method for designing this type of splice is to establish a flange force (required strength) that is statically equivalent to the applied moments and then to design the bolts, welds, plates, and fillers (if required) for this force.

Major Axis Bending. If M_x is the major axis applied moment and d is the depth of the deeper of the two columns, the flange force (or required strength) is

$$F_{fx} = \frac{M_x}{d}$$

Minor Axis Bending. The force distribution is similar to that shown in Fig. 2.28 for erection stability. The force F in the case of actual (factored) design loads can be quite large and will need to be distributed over some finite bearing area as shown in Fig. 2.29. In Fig. 2.29, the bearing area is $2\varepsilon t$, where t is the thickness of the thinner flange, ε is the position of the force F from the toe of the flange of the smaller column, and T is the force per gage line of bolts. The quantities T and F are for each of the two flanges. If M_y is the weak axis applied moment, $M_f = M_y/2$ is the weak axis applied moment per flange. Taking moments about O gives (per flange)

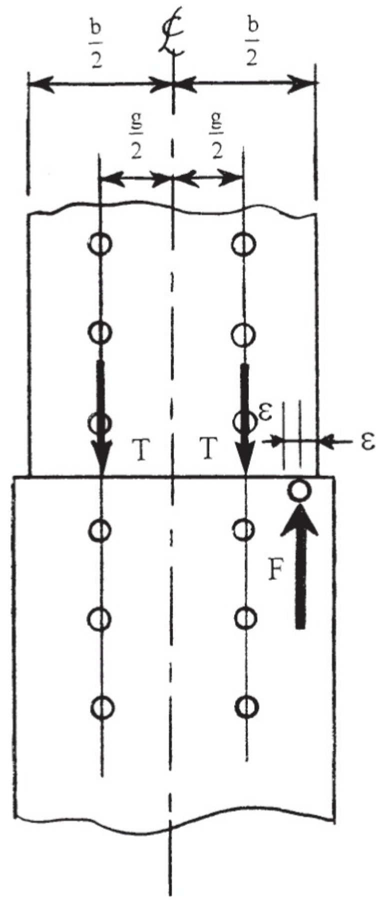


FIGURE 2.29 Force distribution for minor axis bending.

$$M_f = T\left(\frac{b}{2} - \frac{g}{2} - \varepsilon\right) + T\left(\frac{b}{2} + \frac{g}{2} - \varepsilon\right) = T(b - 2\varepsilon)$$

The bearing area is determined by requiring that the bearing stress reaches its design strength at the load F. Thus, $0.75\,(1.8F_y)\,(2\varepsilon)\,t = F$, and since from vertical equilibrium $F = 2T$, and

$$0.75(1.8F_y)\, t\, \varepsilon = T$$

Thus $M_f = 0.75(1.8F_y)\, t\varepsilon(b\ 2\ 2\varepsilon)$

and solving for ε

$$\varepsilon = \frac{1}{4}b - \frac{1}{2}\sqrt{\left(\frac{b}{2}\right)^2 - \frac{40}{27}\left(\frac{M_f}{F_y}\right)} = \frac{1}{4}b\left[1 - \sqrt{1 - \frac{8}{3}\frac{M_f}{\phi M_{py}}}\right]$$

where $M_{py} = F_y Z_y = \frac{1}{2}F_y t b^2$

This expression for ε is valid as long as

$$M_f \le \frac{27}{40}\left(\frac{F_y t b^2}{4}\right) = \frac{3}{8}\phi M_{py}$$

When $M_f > 3/8\ \phi M_{py}$, the tension T on the bolts on the bearing side vanishes and Fig. 2.30 applies. In this case, $F = T = 0.75\ (1.8\ F_y)\ t\ (2\varepsilon)$,

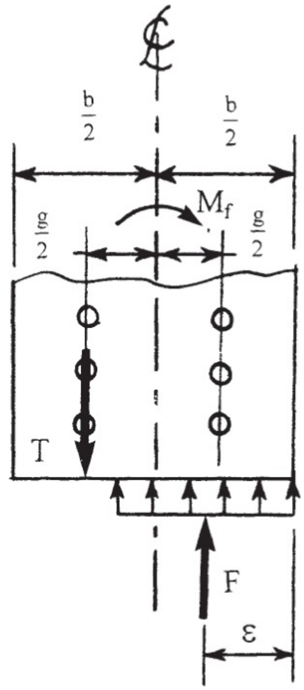


FIGURE 2.30 Splice force distribution when bolts on bearing side are ineffective.

$$M_f = T\left(\frac{b+g}{2} - \varepsilon\right)$$

and

$$\varepsilon = \frac{1}{4}(b+g) - \frac{1}{2}\sqrt{\left(\frac{b+g}{2}\right)^2 - \frac{40}{27}\left(\frac{M_f}{F_y t}\right)} = \frac{1}{4}b\gamma\left[1 - \sqrt{1 - \frac{8}{3}\frac{M_f}{\phi M_{py}}\left(\frac{1}{\gamma}\right)^2}\right]$$

where $\gamma = 1 + g/b$.

This expression for ε is valid as long as

$$M_f \leq \frac{27}{40}\frac{F_y t(b+g)^2}{4} = \frac{3}{8}\gamma^2 \phi M_{py}$$

but T need never exceed Mf/g. The flange force in every case is $F_{fy} = 2T$.

Example Design a bolted splice for a W14 × 99 upper shaft to a W14 × 193 lower shaft. Design the splice for 15% of the axial capacity of the smaller member plus 20% of the smaller member's bending capacity about either the major or the minor axis, whichever produces the greater flange force F_f. The columns are ASTM A992, the splice plates are ASTM A36, and the bolts are ASTM A490 1-in-diameter X type. The holes are standard 1⅛-in diameter. The gage is 7½ in.

The completed splice is shown in Fig. 2.31. The flange force due to tension is

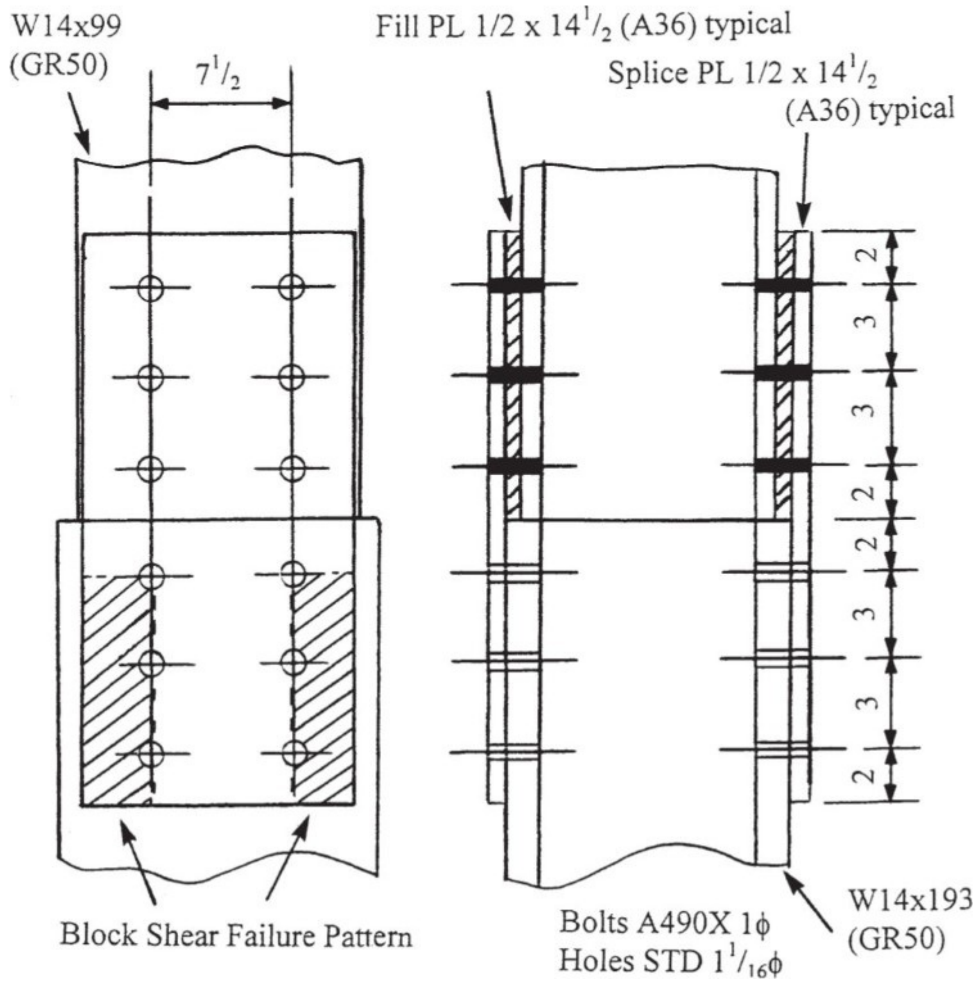


FIGURE 2.31 Bolted column splice for biaxial bending.

$$F_{f_t} = 0.15 \times \phi \frac{F_y}{2} A_g = \frac{0.15 \times 0.9 \times 50 \times 29.1}{2} = 98.2 \text{ kips}$$

The flange force due to major axis bending is

$$F_{f_x} = \frac{0.20 \phi M_{px}}{d} = \frac{0.20 \times 647 \times 12}{14.16} = 110 \text{ kips}$$

The flange force due to minor axis bending is calculated as follows:

$$M_f = \frac{0.20 \phi M_{py}}{2} = \frac{0.20 \times 0.9 \times 50 \times 836}{2} = 376 \text{ kips}$$

Check that $M_f = 376 \leq 3/8 \times 0.9 \times 50 \times 83.6 = 1410$ kips-in, ok.

Calculate

$$\varepsilon = \frac{1}{4} \times 14.565 \left[1 - \sqrt{1 - \frac{376}{1410}} \right] = 0.523 \text{ in}$$

Thus, $T = 0.75(1.8 \times 50) \times 0.780 \times 0.523 = 27.5$ kips and $F_{fy} = 2 \times 27.5 = 55.0$ kips.

The flange force for design of the splice is thus

$$f_f = F_{f_t} + \max\{F_{f_x}, F_{f_y}\} = 98.2 + \max\{110, 55.0\} = 208 \text{ kips}$$

Suppose that $M_f > 3/8\phi\, M_{py}$. Let $M_f = 1500$ kips-in, say, $\gamma = 1 + 7.5/14.565 = 1.515$ and check $M_f = 1500$ kips in $< 3/8\, \gamma\, \phi\, M_{py} = (1.515)^2 \times 1410 = 3236$ kips in, so proceeding

$$\varepsilon = \frac{1}{4} \times 14.565 \times 1.515 \left[1 - \sqrt{1 - \frac{1500}{3236}}\right] = 1.476 \text{ in}$$

$T = 0.75 \times (1.8 \times 50) \times 0.780 \times 1.476 = 77.5$ kips and $F_{fy} = 2\,T = 155$ kips, which is still less than the maximum possible value of $F_{fy} = 1500/7.5 \times 2 = 400$ kips

Returning to the splice design example, the splice will be designed for a load of 208 kips. Since the columns are of different depths, fill plates will be needed. The theoretical fill thickness is (15½ – 14⅛)/2 = 11/16 in, but for ease of erection, the AISC suggests subtracting either ⅛ or 3/16 in, whichever results in ⅛-in multiples of fill thickness. Thus, use actual fills 11/16 – 3/16 = ½ in thick. Since this splice is a bearing splice, either the fills must be developed, or the shear strength of the bolts must be reduced. It is usually more economical to do the latter in accordance with AISC Specification Section J5. Using Section J5, the bolt shear design strength is

$$\phi r_v = 49.5\,[1 - 0.4(0.5 - 0.25)] = 44.6 \text{ kips}$$

The number of bolts required is 208/44.6 = 4.66 or 6 bolts. The choice of six bolts here may have to be adjusted for bearing/tearout as will be seen later. Next, the splice plates are designed. These plates will be approximately as wide as the narrower column flange. Since the W14 × 99 has a flange width of 14⅝ in., use a plate 14½ in. wide. The following limit states are checked:

1. Gross area: The required plate thickness based on gross area is $t_p = 208/(0.9 \times 36 \times 14.5) = 0.44$ in. Use a ½-in plate so far.
2. Net area: The net area is $A_n = (14.5 - 2 \times 1.19) \times 0.5 = 6.06$ in^2. The design strength in gross tension is $\phi R_n = 0.75 \times 58 \times 6.06 = 264$ kips > 208 kips, ok.
3. Block shear rupture: Since $b - g < g$, the failure will occur as shown in Fig. 2.31 on the outer parts of the splice plate.

$$A_{gv} = 8 \times 0.5 \times 2 = 8.0 \text{ in}^2$$
$$A_{gt} = (14.5 - 7.5) \times 0.5 = 3.5 \text{ in}^2$$
$$A_{nv} = 8.0 - 2.5 \times 1.19 \times 0.5 \times 2 = 5.03 \text{ in}^2$$
$$A_{nt} = 3.5 - 1 \times 1.19 \times 0.5 = 2.91 \text{ in}^2$$
$$F_u A_{nt} = 58 \times 2.91 = 169 \text{ kips}$$
$$0.6\, F_y A_{gv} = 0.6 \times 36 \times 8.0 = 173 \text{ kips}$$
$$0.6\, F_u A_{nv} = 0.6 \times 58 \times 5.03 = 175 \text{ kips}$$
$$U_{bs} = 1.0 \text{ (uniform tension)}$$
$$\phi R_{bs} = 0.75\, [1.0 \times 169 + \min\{173, 175\}] = 257 \text{ kips} > 208 \text{ kips, ok}$$

4. Bearing/tearout: Although we have initially determined that six bolts are required, the following bearing/tearout check may require an adjustment in this number:

a. Bolt shear:

$\phi R_v = 44.6$ kips

b. Bearing on splice plate:

$\phi R_p = 0.75 \times 2.4 \times 1.0 \times 0.5 \times 58 = 52.2$ kips

c. Bearing on W14 × 99 flange:

$\phi R_p = 0.75 \times 2.4 \times 1.0 \times .0.780 \times 65 = 91.3$ kips

d. Tearout on splice plate:

$\phi R_{to} = 0.75 \times 1.2 \times (2 - 0.5 \times 1.125) \times 0.5 \times 58 = 37.5$ kips

e. Tearout on W14 × 99 flange:

$\phi R_{to} = 0.75 \times 1.2(2 - 0.5 \times 1.125) \times 0.780 \times 65 = 65.6$ kips

Two more tearout limit states are related to the spacing of the bolts, but these are obviously not critical.

The bearing/tearout limit state is

$$\phi R_{\text{pto}} = 4 \times 44.6 + 2 \times 37.5 = 253 \text{ kips} > 208 \text{ kips, ok}$$

5. Whitmore section:

$$l_w = (6 \tan 30)2 + 7.5 = 14.43 \text{ in}$$
$$\phi R_n = 0.9 \times 36 \times 14.43 \times 0.5 = 234 \text{ kips} > 208 \text{ kips}$$

Note that if $l_w > 14.5$ in, 14.5 in would have been used in the calculation of design strength.

In addition to the checks for the bolts and splice plates, the column sections should also be checked for bearing and block shear rupture. These are not necessary in this case because $t_f = 0.780 > t_p = 0.50$, the edge distances for the column are the same as for the plates, and the column material is stronger than the plate material.

2.2.5.2 Splices in Truss Chords. These splices must be designed for 50% of the chord load as an axial force, or 2% of the chord load as a transverse force, as discussed in Sec. 5.5.3, even if the load is compression and the members are finished to bear. As discussed earlier, these splices may be positioned in the center of a truss panel and, therefore, must provide some degree of continuity to resist bending. For the tension chord, the splice must be designed to carry the full tensile load.

Example Design the tension chord splice shown in Fig. 2.32. The load is 800 kips (factored). The bolts are A325X, ⅞ in in diameter, ϕr_V = 30.7 kips. The load at this location is controlled by the W14 × 90, so the loads should be apportioned to flanges and web based on this member. Thus, the flange load is

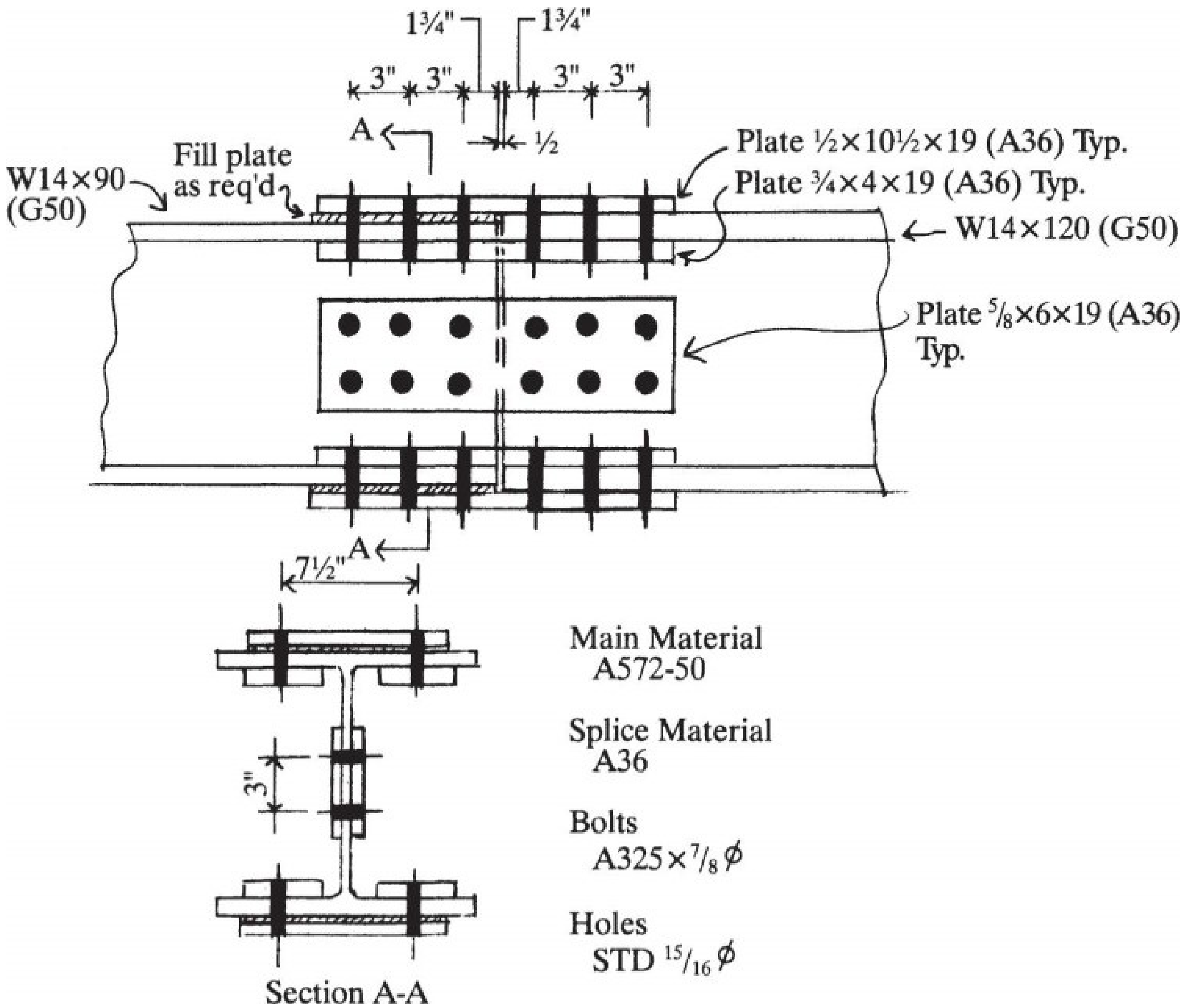


FIGURE 2.32 Truss chord tension splice.

$$P_f = \frac{0.710 \times 14.520}{26.5} \times 800 = 311 \text{ kips}$$

and the web load is

$$P_w = 800 - 2 \times 311 = 178 \text{ kips}$$

The load path is such that the flange load P_f passes from the W14 × 90 (say) through the bolts into the flange plates and into the W14 × 120 flanges through a second set of bolts. The web load path is similar.

A. Flange connection.

1. Member limit states:

a. *Bolts*: Although not a member limit state, a bolt pattern is required to check the chords. The number of bolts in double shear is 311/(2 × 30.7) = 5.07. Try 6 bolts in 2 rows of 3 as shown in Fig. 2.34. This may need to be adjusted because of bearing/tearout.

b. *Chord net section:* Check to see if the holes in the W14 × 90 reduce its capacity below 800 kips. Assume that there will be two web holes in alignment with the flange holes.

$$A_{\text{net}} = 26.5 + 4 \times 1 \times 0.710 - 2 \times 1 \times .440 = 22.8 \text{ in}^2$$
$$\phi R_{\text{net}} = 22.8 \times 0.75 \times 65 = 1111 \text{ kips} > 800 \text{ kips, ok}$$

c. *Bearing/tearout:* This will be checked after the splice plates are designed.

d. *Block shear fracture:*

$$A_{nv} = (7.75 - 2.5 \times 1.0)0.710 \times 2 = 7.46 \text{ in}^2$$
$$A_{nt} = \left(\frac{(14.520 - 7.5)}{2} - 0.5 \times 1\right)0.710 \times 2 = 4.27 \text{ in}^2$$
$$A_{gv} = 7.75 \times 0.710 \times 2 = 11.0 \text{ in}^2$$
$$A_{gt} = 3.51 \times 0.710 \times 2 = 4.98 \text{ in}^2$$
$$F_u A_{nt} = 65 \times 4.27 = 278 \text{ kips}$$
$$0.6F_y A_{gv} = 0.6 \times 50 \times 11.00 = 330 \text{ kips}$$
$$0.6F_u A_{nv} = 0.6 \times 65 \times 7.46 = 291 \text{ kips}$$
$$U_{bs} = 1.0$$
$$\phi R_{bs} = 0.75[278 + \min\{330, 291\}] = 427 \text{ kips} > 311 \text{ kips, ok}$$

2. Flange plates: Since the bolts are assumed to be in double shear, the load path is such that one half of the flange load goes into the outer plate, and one half goes into the inner plates.

a. *Outer plate:*

(1) Gross and net area: Since the bolt gage is 7½ in, try a plate 10½ in wide. The gross area in tension required is

$$A_{gt} = \frac{311/2}{0.9 \times 36} = 4.8 \text{ in}^2$$

and the thickness required is 4.8/10.5 = 0.46 in. Try a plate ½ × 10½

$$A_{gt} = 0.5 \times 10.5 = 5.25 \text{ in}^2$$
$$A_{nt} = (10.5 - 2 \times 1) \times 0.5 = 4.25 \text{ in}^2$$
$$0.85\ A_{gt} = 0.85 \times 5.25 = 4.46 \text{ in}^2$$

Since 0.85 $A_{gt} > A_{nt}$, use A_{nt} = 4.25 in^2 as the effective net tension area.

$$\phi R_{nt} = 0.75 \times 58 \times 4.25 = 185 \text{ kips} > 311/2 = 156 \text{ kips, ok}$$

Use a plate ½ × 10½ for the outer flange splice plate for the following limit state checks:

(2) Block shear fracture:

$$A_{gv} = 7.5 \times 0.5 \times 2 = 7.5 \text{ in}^2$$
$$A_{gt} = 1.5 \times 0.5 \times 2 = 1.5 \text{ in}^2$$
$$A_{nv} = (7.5 - 2.5 \times 1)0.5 \times 2 = 5.0 \text{ in}^2$$
$$A_{nt} = (1.5 - .5 \times 1)0.5 \times 2 = 1.0 \text{ in}^2$$
$$F_u A_{nt} = 58 \times 1.0 = 58.0 \text{ kips}$$
$$0.6 F_y A_{gy} = 0.6 \times 36 \times 7.5 = 162 \text{ kips}$$
$$0.6 F_u A_{nv} = 0.6 \times 58 \times 5.0 = 174 \text{ kips}$$
$$\phi R_{bs} = 0.75[58 + \min\{162, 174\}] = 165 \text{ kips} > 156 \text{ kips, ok}$$

(3) Bearing:

$$\phi R_p = 0.75 \times 2.4 \times 58 \times 0.50 \times .875 \times 6 = 274 \text{ kips} > 156 \text{ kips, ok}$$

Thus, the plate ½ × 10½ (A36) outer splice plate is ok, but bearing/tearout still needs to be checked.

b. *Inner plates:*

(1) Gross and net area: The load to each plate is 156/2 = 78 kips. The gross area in tension required is

$$A_{gt} = \frac{78}{0.9 \times 36} = 2.41 \text{ in}^2$$

Try a plate 4 in wide. Then the required thickness is 2.41/4 = 0.6 in.

Try a plate ¾ × 4 (A36).

$$A_{gt} = 0.75 \times 4 = 3 \text{ in}^2$$
$$A_{nt} = (4 - 1.0)0.75 = 2.25 \text{ in}^2$$
$$0.85A_{gt} = 0.85 \times 3 = 2.55 \text{ in}^2$$
$$\phi R_{nt} = 0.75 \times 58 \times 2.25 = 97.9 \text{ kips} > 78 \text{ kips, ok}$$

(2) Block shear fracture: Since there is only one line of bolts, this limit state is not possible. The plate will fail in net tension.

The ¾ × 4 (A36) inner splice plates are so far ok, now check bearing/tearout.

3. *Bearing/tearout*: Now that the bolts, the outer plate, and the inner plates have been chosen, bearing/tearout can be checked for the connection as a whole.

a. *Bolt shear:*

$$\phi r_v = 61.3 \text{ kips (double shear)}$$
$$\phi r_v = 30.7 \text{ kips (single shear)}$$

b. *Bearing on W14 × 99 flange:*

$$\phi r_p = 0.75 \times 2.4 \times 0.875 \times 0.710 \times 65 = 72.7 \text{ kips}$$

c. *Bearing on outer plate:*

$$\phi r_p = 0.75 \times 2.4 \times 0.875 \times 0.5 \times 58 = 45.7 \text{ kips}$$

d. *Bearing on inner plate:*

$$\phi r_p = 0.75 \times 2.4 \times 0.875 \times 0.75 \times 58 = 68.5 \text{ kips}$$

e. *Tearout on W14 × 99 flange; $L_c = 1.75 - 0.5 \times 0.9375 = 1.281$ in:*

$$\phi r_{to} = 0.75 \times 1.2 \times 1.281 \times 0.710 \times 65 = 53.2 \text{ kips}$$

f. *Tearout on outer plate; $L_c = 1.5 - 0.5 \times 0.9375 = 1.031$ in:*

$$\phi r_{to} = 0.75 \times 1.2 \times 1.031 \times 0.5 \times 58 = 26.9 \text{ kips}$$

g. *Tearout on inner plates; $L_c = 1.031$ in:*

$$\phi r_{to} = 0.75 \times 1.2 \times 1.031 \times 0.75 \times 58 = 40.4 \text{ kips}$$

Tearout between bolts will not control in this case since 3 – 0.9375 = 2.0625 > 1.281 or 1.031.

From the above, the shear/bearing/tearout strength of the flange connection is

$$\phi R_{vpt} = 2 \times 30.7 \times 2 + 2 \times 26.9 \times 2 + 2 \times 30.7 \times 2 = 324 \text{ kips} > 311 \text{ kips, ok}$$

In the expression for ϕR_{vpt}, the first term is for the two bolts in the center, which are controlled by shear; the second term is for the outer two bolts controlled by outer plate edge distance; and the third term is for the two inner bolts again controlled by bolt shear.

This completes the calculation for the flange portion of the splice. The bolts, outer plate, and inner plates, as chosen above, are ok.

B. Web connection: The calculations for the web connection involve the same limit states as the flange connection, except for chord net section, which involves flanges and web.

1. Member limit states:

a. *Bolts:* A bolt pattern is required to check the web.

$$\text{Number required} = \frac{178}{2 \times 30.7} = 2.90$$

Try four bolts.

b. *Bearing/tearout:* This will be checked after the web splice plates are designed.

c. *Block shear fracture:* Assume the bolts have a 3-in pitch longitudinally.

$$\begin{aligned}
A_{nv} &= (4.75 - 1.5 \times 1) \times 0.440 \times 2 = 2.86 \text{ in}^2 \\
A_{nt} &= (3 - 1 \times 1) \times 0.440 = 0.88 \\
A_{gv} &= 4.75 \times 0.440 \times 2 = 4.18 \text{ in}^2 \\
A_{gt} &= 3 \times 0.440 = 1.32 \text{ in}^2 \\
F_u A_{nt} &= 65 \times 0.88 = 57.2 \text{ kips} \\
0.6 F_u A_{nv} &= 0.6 \times 65 \times 2.86 = 112 \text{ kips} \\
0.6 F_y A_{gv} &= 0.6 \times 50 \times 4.18 = 125 \text{ kips} \\
U_{bs} &= 1.0 \\
\phi R_{bs} &= 0.75[57.2 + \min\{115,125\}] = 127 \text{ kips} < 178 \text{ kips, no good}
\end{aligned}$$

Since the block shear limit state fails, the bolts can be spaced out to increase the capacity. Increase the bolt pitch from the 3 in assumed above to 6 in. Then

$$A_{nv} = (7.75 - 1.5 \times 1) \times 0.440 \times 2 = 5.50 \text{ in}^2$$
$$A_{nt} = 0.88 \text{ in}^2$$
$$A_{gv} = 7.75 \times 0.440 \times 2 = 6.82 \text{ in}^2$$
$$A_{gt} = 1.32 \text{ in}^2$$
$$U_{bs} = 1.0$$
$$F_u A_{nt} = 65 \times 0.88 = 57.2 \text{ kips}$$
$$0.6 F_u A_{nv} = 0.6 \times 65 \times 5.50 = 214 \text{ kips}$$
$$0.6 F_y A_{gv} = 0.6 \times 50 \times 6.82 = 205 \text{ kips}$$
$$\phi R_{bs} = 0.75[57.2 + \min\{214, 205\}] = 197 \text{ kips} > 178 \text{ kips, ok}$$

The web bolt pattern shown in Fig. 2.32 is the final design. At this point, there are four bolts in the web at 6-in pitch, but the six bolts shown will be required.

2. Web plates: Try two plates, one each side of web, 6 in wide and ½ in thick.

a. *Gross area:*

$$\phi R_{gt} = 0.9 \times 36 \times 0.5 \times 6 \times 2 = 194 \text{ kips} > 178 \text{ kips, ok}$$

b. *Net area:*

$$A_{nt} = (6 - 2 \times 1) \times 0.5 \times 2 = 4.0 \text{ in}^2$$
$$.85 A_{gt} = 0.85 \times 0.5 \times 6 \times 2 = 5.1 \text{ in}^2$$
$$\phi R_{nt} = 0.75 \times 58 \times 4.0 = 174 \text{ kips} < 178 \text{ kips, no good}$$

Increase web plates to ⅝ in thick. Net area will be ok by inspection.

c. *Block shear rupture:* This is checked as shown in previous calculations. It is not critical here.

3. *Bearing/tearout:* The bolt pattern and plates are now known, so this combined limit state can be checked.

a. *Bolt shear:*

$$\phi r_v = 61.3 \text{ kips (double shear)}$$
$$\phi r_v = 30.7 \text{ kips (single shear)}$$

b. *Bearing on W14 × 99 web:*

$$\phi r_p = 0.75 \times 2.4 \times 0.875 \times 0.440 \times 65 = 45.0 \text{ kips}$$

c. *Bearing on splice plates:*

$$\phi r_p = 0.75 \times 2.4 \times 0.875 \times 0.625 \times 2 \times 58 = 114 \text{ kips}$$

d. *Tearout on W14 × 99 web:*

$$L_c = 1.75 - 0.5 \times .9375 = 1.281$$
$$\phi r_{to} = 0.75 \times 1.2 \times 1.281 \times 0.440 \times 65 = 33.0 \text{ kips}$$

e. *Tearout on splice plates:*

$$\phi r_{to} = 0.75 \times 1.2 \times 1.281 \times 0.625 \times 2 \times 58 = 83.6 \text{ kips}$$

Tearout between bolts will not control in this case.

From the above, the shear/bearing/tearout strength of the connection is

$$\phi R_{\text{vpt}} = 2 \times 33.0 + 2 \times 45.0 = 156 \text{ kips} < 178 \text{ kips, no good}$$

Add two bolts in the web. The 6-in pitch become 3-in pitch as shown in Fig. 2.32. The shear/bearing/tearout capacities per bolt given above do not change. Tearout between bolts is still not critical. Thus

$$\phi R_{\text{vpt}} = 2 \times 33.0 + 4 \times 45.0 = 246 \text{ kips} > 178 \text{ kips, ok}$$

Note that, in this case, none of the bolts was able to achieve its double shear value.

4. Additional checks because of change in web bolts pattern:

a. *Block shear fracture:*

$$A_{nv} = (7.75 - 2.5 \times 1.0)\ 0.440 \times 2 = 4.62 \text{ in}^2$$
$$A_{gv} = 7.75 \times 0.440 \times 2 = 6.82 \text{ in}^2$$
$$A_{nt} = (3 - 1 \times 1.0) \times 0.440 = 0.88 \text{ in}^2$$
$$F_u A_{nt} = 65 \times 0.88 = 57.2 \text{ kips}$$
$$0.6 F_u A_{nv} = 0.6 \times 65 \times 4.62 = 180 \text{ kips}$$
$$0.6 F_y A_{gv} = 0.6 \times 50 \times 6.82 = 205 \text{ kips}$$
$$U_{bs} = 1.0$$
$$\phi R_{bs} = 0.75\ [57.2 + \min\{180, 205\}] = 178 \text{ kips} = 178 \text{ kips, ok}$$

No other design check must be done. The final design is shown in Fig. 2.32.

If this were a non-bearing compression splice, the splice plates would be checked for buckling. The following paragraph shows the method, which is not required for a tension splice.

5. Buckling: The plates at the web splice line of 4 in length can be checked against a load of 178/2 = 89 kips/plate. The slenderness ratio is

$$\frac{Kl}{r} = 0.65 \times 4.0 \times \sqrt{\frac{12}{0.625}} = 14.4$$

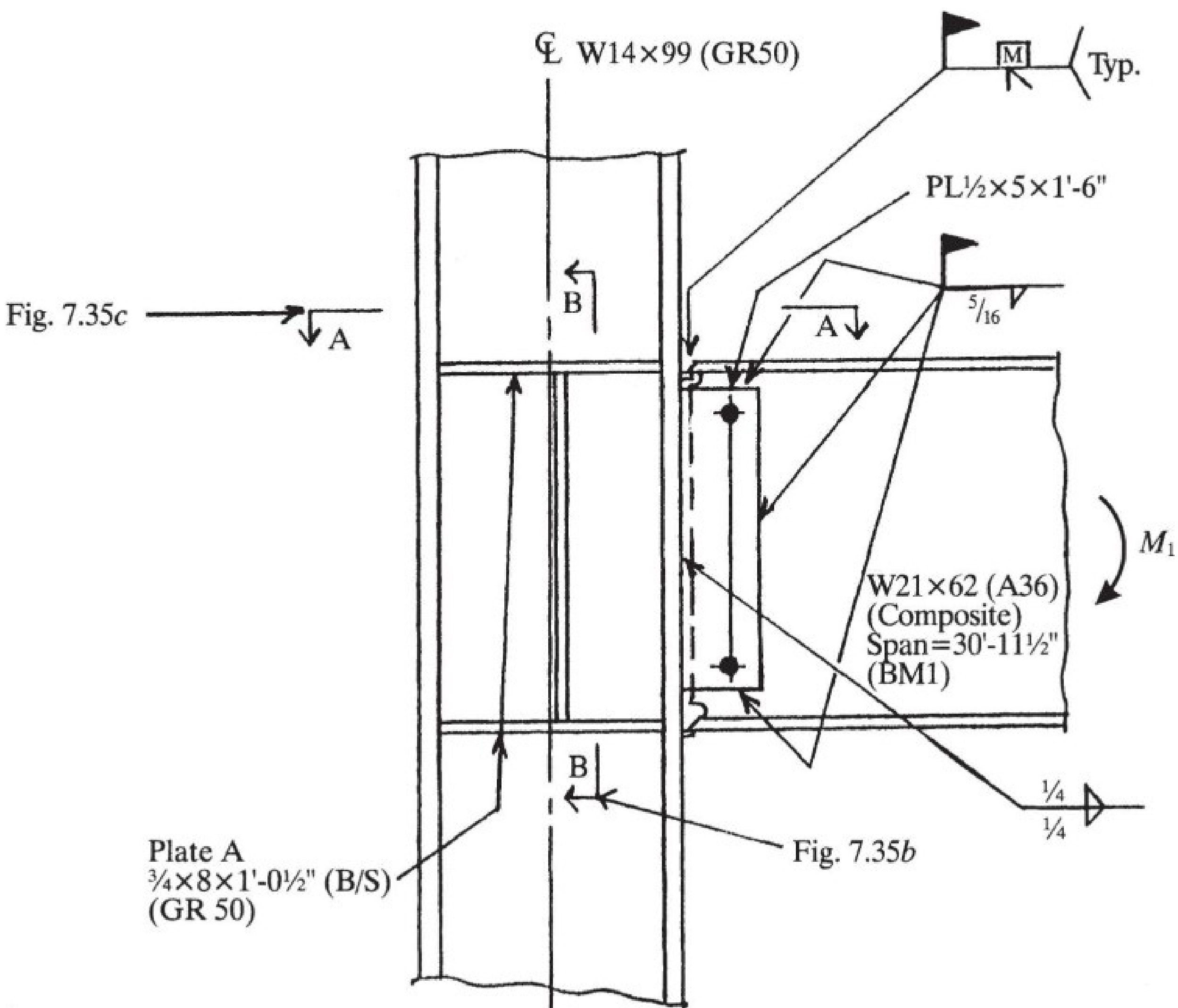
Since this is less than 25, AISC Specification Section J4.4 allows the plate to be checked for yield rather than buckling. This has already been done.

This limit state is checked for the flange plates also.

2.3 MOMENT CONNECTIONS

2.3.1 Introduction

The most commonly used moment connection is the field-welded moment connection as shown in Fig. 2.33*a*. This connection is in common use in all regions of the United States, where the seismic design category (SDC) is A, B, or C, and the response modification factor R is three or less (AISC Seismic Provisions, 2005).



$M_1 = \phi M_p = 389$ k-ft (Full Moment Capacity)

(a)

FIGURE 2.33*a* Field-welded moment connection.

2.3.2 Example: Three-Way Moment Connection

The moment connection of Fig. 2.33*a* is a three-way moment connection. Additional views are shown in Figs. 2.33*b* and 2.33*c*. If the strong axis connection requires stiffeners, there will be an interaction between the flange forces of the strong and weak axis beams. If the primary function of these moment connections is to resist lateral maximum load from wind or seismic sources, the interaction can generally be ignored because the maximum lateral loads will act in only one direction at any one time. If the moment connections are primarily used to carry gravity loads, such as would be the case when stiff floors with small deflections and high natural frequencies are desired, there will be interaction between the weak and strong beam flange forces. The calculations here will be for both a wind or a seismic condition in a region of low to moderate seismicity (SDC A, B, or C, and R = 3), and gravity condition. Thus, interaction will be included.

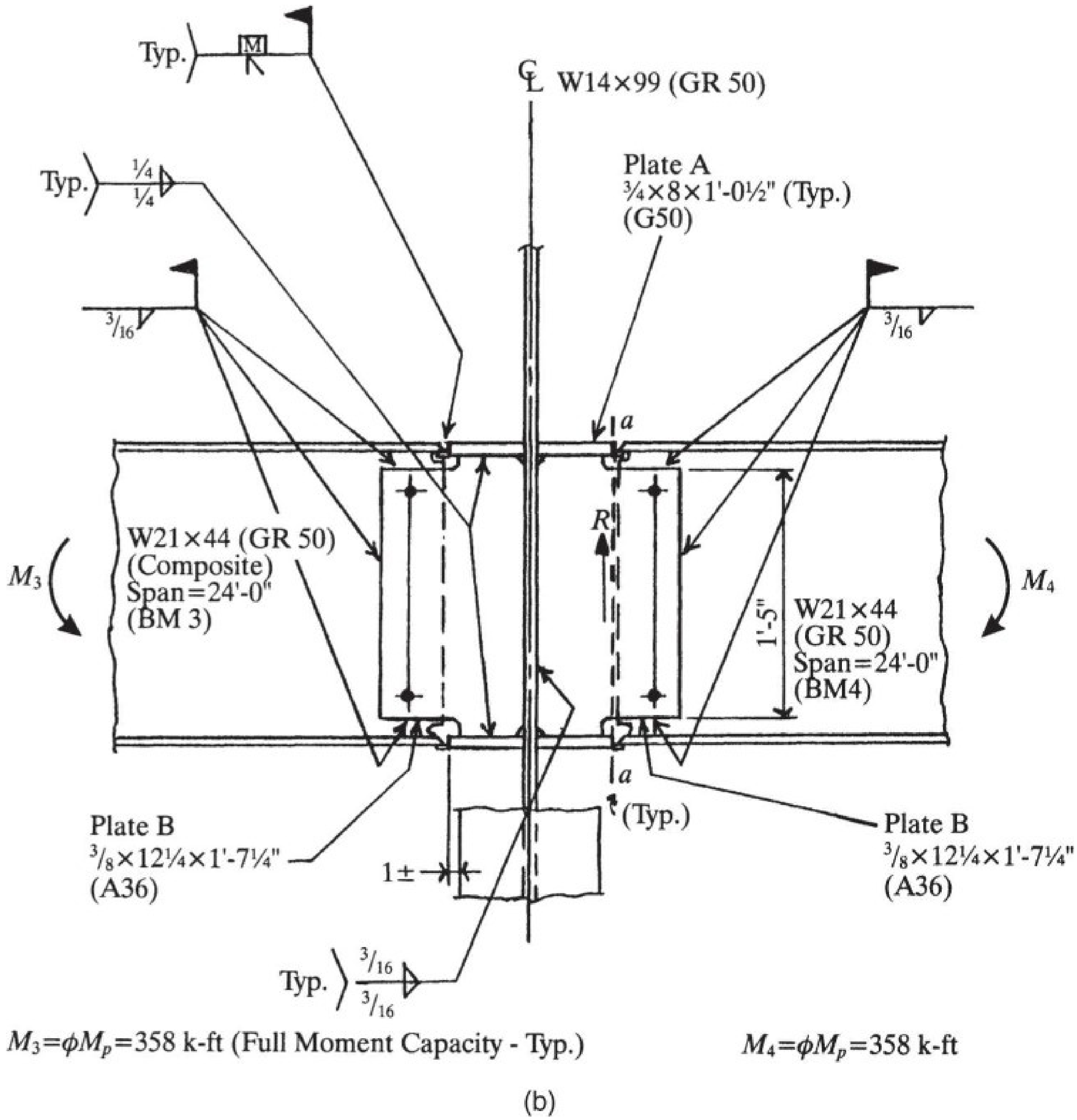


FIGURE 2.33*b* Section B-B of Fig. 2.33*a*.

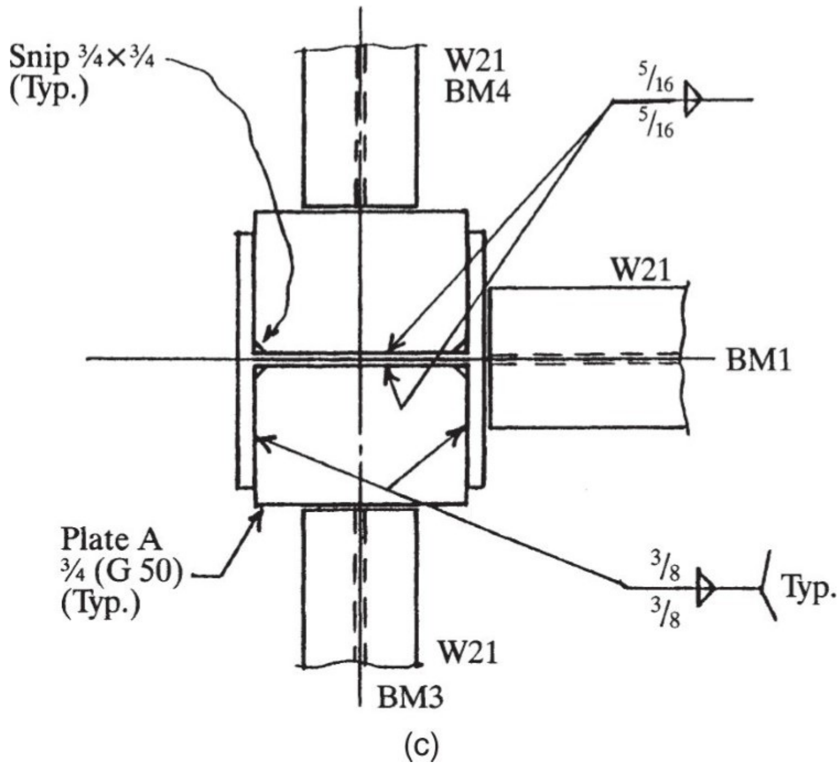


FIGURE 2.33*c* Section A-A of Fig. 2.33*a*.

The load path through this connection that is usually assumed is that the moment is carried entirely by the flanges, and the shear entirely by the web. This load path has been verified by testing (Huang et al., 1973) and will be the approach used here. Proceeding to the connection design, the strong axis beam, beam no. 1, will be designed first.

Beam No. 1 W21 × 62 (A36) Composite. The flange connection is a full penetration (referred to as a CJP weld in AWS D1.1) weld, so no design is required. The column must be checked for stiffeners and doublers.

Stiffeners. The connection is to be designed for a given moment of ϕM_b = 389 kips-ft. The given beam moment of ϕM_b = 389 kips-ft can only be achieved if the column is strong enough to support it. The full plastic moment capacity of the column is

$$\phi M_p = 0.9 \times \frac{50}{12} \times 173 = 649 \text{ kips-ft}$$

Thus, since $\phi M_b = 389 < 2 \times 649 = 1300$, the column can support the specified beam moment.

Thus, the flange force F_f is

$$F_f = \frac{\phi M_b}{d - t_f} = \frac{389 \times 12}{(20.99 - 0.615)} = 229 \text{ kips}$$

From Table 4-1 of the AISC 15th Edition *Manual of Steel Construction*.

Web yielding: $P_{wy} = P_{wo} + t_b\, P_{wi} = 167 + 0.615 \times 24.3 = 182$ kips < 229 kips, thus stiffeners are required at both flanges.

Web buckling: P_{wb} = 260 kips > 229 kips – no stiffener required at compression flange.

Flange bending: P_{fb} = 171 kips < 229 kips – stiffener required at tension flange.

From the preceding three checks (limit states), a stiffener is required at both flanges. For the tension flange, the total stiffener force is 229 – 171 = 58 kips and for the compression flange, the stiffener force is 229 – 189 = 40 kips. But the loads may reverse, so use the larger of 58 and 40 as the stiffener force for both flanges. Then, the force in each stiffener is 58/2 = 29 kips, both top and bottom.

Determination of Stiffener Size. The minimum stiffener width w_s is

$$\frac{b_{fb}}{3}-\frac{t_{wc}}{2}=\frac{8.24}{3}-\frac{0.485}{2}=2.5\text{ in}$$

Use a stiffener 6½ in wide to match column.

The minimum stiffener thickness t_s is

$$\frac{t_{fb}}{2}=\frac{0.615}{2}=0.31\text{ in}$$

Use a stiffener at least ⅜ in thick.

The minimum stiffener length l_s is

$$\frac{d_c}{2}-t_{fc}=\frac{14.2}{2}-0.78=6.3\text{ in}$$

The minimum length is for a “half depth” stiffener, which is not possible in this example because of the weak axis connections. Therefore, use a full-depth stiffener of 12½ in length.

A final stiffener size check is a plate buckling check that requires that

$$t_s \geq \frac{Ws}{15}=\frac{6.5}{15}=0.433\text{ in}$$

Therefore, the minimum stiffener thickness is ½ in. The final stiffener size for the strong axis beam is ½ × 6½ × 12½. The contact area of this stiffener against the inside of the column flange is 6.5 – 0.75 = 5.75 due to the snip to clear the column web to flange fillet. The stiffener design strength is thus 0.9 × 36 × 5.75 × 0.5 = 93.2 kips > 29 kips, ok.

Welds of Stiffeners to Column Flange and Web. Putting aside for the moment that the weak axis moment connections still need to be considered and will affect both the strong axis connection stiffeners and welds, the welds for the ½ × 6½ × 12½ strong axis stiffener are designed as follows. For the weld to the inside of the flange, the force to be developed by the weld to the connected portion is 29 kips. Thus, the 5¾ contact, which is the connected portion, is designed for 29 kips. The weld to the flange is thus

$$D_f = \frac{29}{2 \times 5.75 \times 1.392 \times 1.5} = 1.21$$

An AISC minimum fillet weld is indicated. The factor 1.5 in the denominator above comes from the AISC Specification, Section J2.4, for transversely loaded fillets. The weld to the web has a length 12.5 – 0.75 – 0.75 = 11.0, and is designed to transfer the unbalanced force in the stiffener to the web. The unbalanced force in the stiffener is 29 kips in this case. Since the weld at the web and the weld at the flange do not share load in this case, both the longitudinally and transversely loaded welds can develop their full strength. Thus,

$$D_w = \frac{29}{2 \times 11.0 \times 1.392} = 0.95$$

An AISC minimum fillet is indicated.

2.3.2.1 Doublers. The beam flange force (required strength) delivered to the column is F_f = 229 kips. The design shear strength of the column $\phi V_v = 0.9 \times 0.6 \times 50 \times 0.485 \times 14.16 = 185$ kips < 229 kips, so a doubler appears to be required. However, if the moment that is causing doublers is ϕM_b = 389 kips-ft, then from Fig. 2.34, the column story shear is

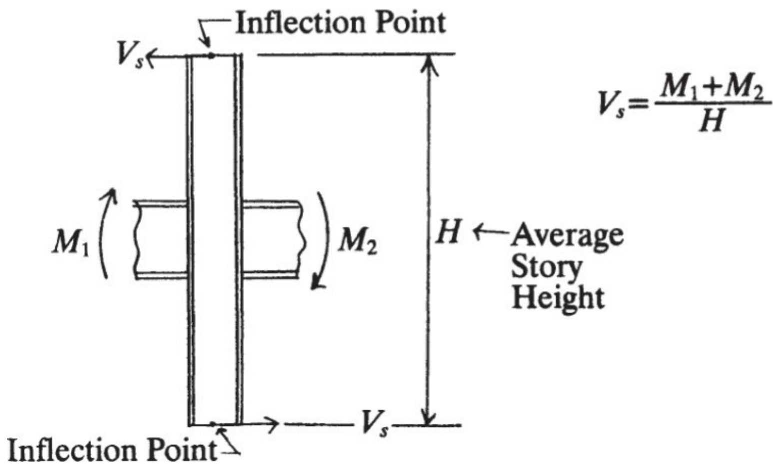


FIGURE 2.34 Relationship between column story shear and the moments which induce it.

$$V_s = \frac{\phi M_b}{H}$$

where H is the story height. If H = 13 ft,

$$V_s = \frac{389}{13} = 30 \text{ kips}$$

and the shear delivered to the column web is $F_f - V_s = 229 - 30 = 199$ kips. Since 199 kips > 185 kips, a doubler (or doublers) is still indicated. If some panel zone deformation is acceptable, the AISC Specification Section J10.6, Formula J10-11 or J10-12, contains an extra term which increases the panel zone strength. The term is

$$\frac{3b_{fc}t_{fc}^2}{d_b d_c t_{wc}} = \frac{3 \times 14.6 \times 0.780^2}{21.0 \times 14.2 \times 0.485} = 0.184$$

and if the column load is less than $0.75P_y = 0.75 \times A_c F_{yc} = 0.75 \times 29.1 \times 50 = 1091$ kips, which is the usual case,

$$\phi V_v = 185 \times 1.184 = 219 \text{ kips}$$

Since 219 kips > 199 kips, no doubler is required. In a high-rise building where the moment connections are used for drift control, the extra term can still be used, but an analysis that includes inelastic joint shear deformation should be considered.

Placement of Doubler Plates. If a doubler plate or plates is/are required in this example, the most inexpensive arrangement is to place the doubler plate against the column web between the stiffeners (the panel zone) and to attach the weak axis shear connection plates, plates B, to the face of the doubler. This is permissible provided that the doubler is capable of carrying the entire weak axis shear load $R = 163$ kips on one vertical cross section of the doubler plate. To see this, consider Fig. 2.35. The portion of the shear force induced in the doubler plate by the moment connection flange force F_f is H. For the doubler to be in equilibrium under the forces H, vertical shear forces $V = Hd/w$ must exist. The welds of the doubler at its four edges develop the shear strength of the doubler. Let the shear force R from the weak axis connection be applied to the face of the doubler at or near its horizontal center as shown in Fig. 2.35. If it is required that all of the shear R can be carried by one vertical section a-a of Fig. 2.35, that is, $1.0 \times 0.6 \times F_y t_d d \geq R$, where t_d is the doubler thickness and F_y is the yield strength of the doubler (and the column), then the free-body diagram of Fig. 2.35 is possible. In this figure, all of the shear force R is delivered to the side of the doubler where it is opposite in direction to the shear delivered by the moment connection, thereby avoiding over-stressing the other side where the two shears would add. Since the doubler and its welds are capable of carrying V or R alone, they are capable of carrying their difference. The same argument applies to the top and bottom edges of the doubler. Also, the same argument holds if the moment and/or weak axis shear reverse(s). The validity of this approach is based on the lower bound theorem of limit analysis.

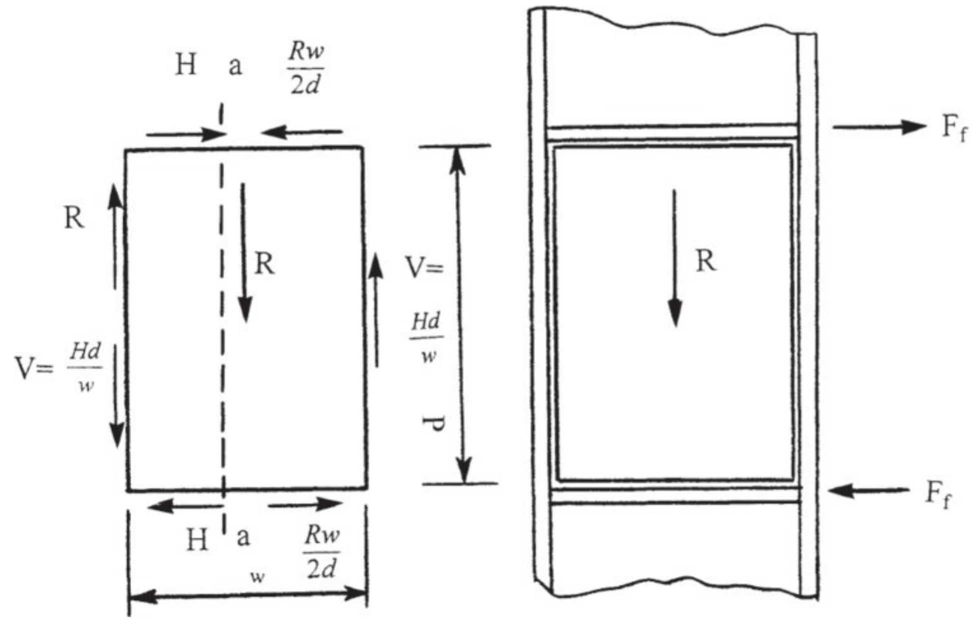


FIGURE 2.35 Equilibrium of doubler plate with weak axis shear load.

2.3.2.2 Associated Shear Connections—Beam 1. The specified shear for the web connection is R = 163 kips, which is the shear capacity of the W21 × 62 (A36) beam. The connection is a shear plate with two erection holes for erection bolts. The shear plate is shop welded to the column flange and field welded to the beam web. The limit states are plate gross shear, weld strength, and beam web strength.

Plate Gross Shear. Try a plate ½ × 18

$$\phi R_{gv} = 0.5 \times 18 \times 0.9 \times 0.6 \times 36 = 175 \text{ kips} > 163 \text{ kips, ok}$$

Plate net shear need not be checked here because it is not a valid limit state.

Weld-to-Column Flange. This weld sees shear only. Thus

$$D = \frac{163}{2 \times 18 \times 1.392} = 3.25; \text{ use } \tfrac{1}{4} \text{ FW}$$

Weld-to-Beam Web. This weld sees the shear plus a small couple. Using AISC 15th Edition Manual Table 8-8, l = 18, kl = 4.25, k = 0.24, x = 0.04, xl = 0.72, al = 4.28, a = 0.24, c = 2.71, and

$$D = \frac{163}{0.75 \times 2.71 \times 18} = 4.46$$

Thus a 5/16 fillet weld is satisfactory.

Beam Web. To support a 5/16 fillet weld on both sides of a plate, AISC LRFD Manual Table 10-2 shows that a 0.476-in web is required. For a 5/16 fillet on one side, a 0.238-in web is required. Since the W21 × 62 web is 0.400 in thick, it is ok.

Beam Nos. 3 and 4 W21 × 44 (G50) Composite. The flange connection is a full penetration weld, so again, no design is required. Section A-A of Fig. 2.33*a* shows the arrangement in plan. See Fig. 2.33*c*. The connection plates A are made ¼ in thicker than the W21 × 44 beam flange to accommodate under and over rolling and other minor misfits. Also, the plates are extended beyond the toes of the column flanges by ¾ to 1 in to improve ductility. The plates A should also be welded to the column web, even if not required to carry load, to provide improved ductility. A good discussion of this is contained in the AISC 15th Edition *Manual of Steel Construction*.

The flange force for the W21 × 44 is based on the full moment capacity as required in this example, so ϕM_p = 358 kips-ft. For gravity moments, the beam moments counteract each other, and the column bending strength is not an issue. For lateral moments, however, the beam moments add, and the column strength may limit the beam moments. The weak-axis column design strength is

$$\phi M_p = 0.9 \times \frac{50}{12} \times 83.6 = 314 \text{ kips-ft}$$

Therefore, for lateral loads, the beam plastic moment cannot be achieved because 2 × 358 > 2 × 314.

For lateral loads, the maximum beam moment is ϕM_b = 314 kips-ft.

In summary, for gravity loads, $\phi M_b = \phi M_p$ = 358 kips-ft and the flange force is

$$F_f = \frac{358 \times 12}{(20.7 - 0.45)} = 212 \text{ kips}$$

and for lateral loads, ϕM_b = 314 kips-ft and the flange force is

$$F_f = \frac{314 \times 12}{(20.7 - 0.450)} = 186 \text{ kips}$$

Figure 2.36 shows the distribution of forces on the plates A, including the forces from the strong axis connection. The weak axis gravity force of 212 kips is distributed one-fourth to each flange and one-half to the web. This is done to cover the case when full gravity loads are not present on each side. In this case, all of the 212 kips must be passed to the flanges. To see this, imagine that beam 4 is removed and the plate A for beam 4 remains as a back-up stiffener. One half of the 212 kips from beam 3 passes into the beam 3 near side column flanges, while the other half is passed through the column web to the back-up stiffener, and thence into the far side flanges, so that all of the load is passed to the flanges. This is the load path usually assumed for gravity loads, although others are possible.

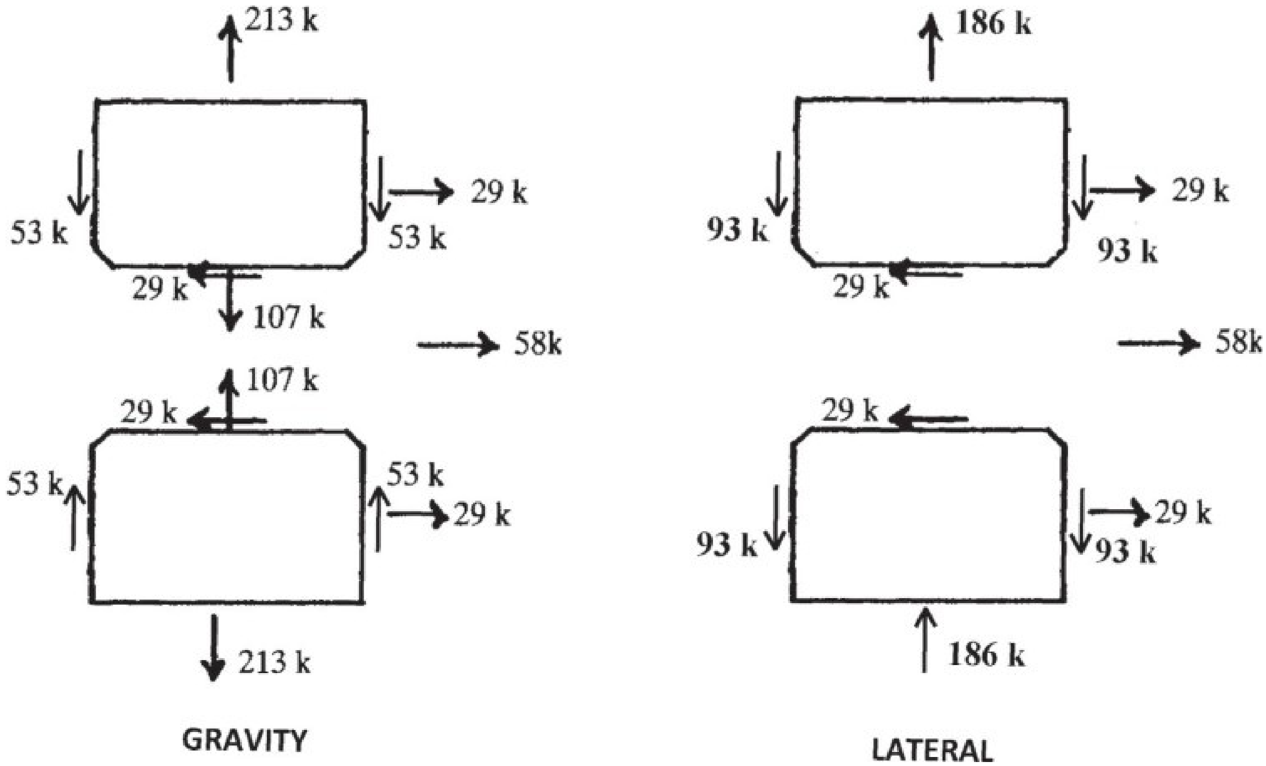


FIGURE 2.36 Distribution of forces on plates A.

The weak-axis lateral load is distributed one-half to each flange and none to the web. As in the unbalanced gravity load case, all load must be delivered to the flanges. Although no load goes to the web, the stiffener would still be welded to the web for ductility purposes.

Merging of Stiffeners from Strong and Weak Axis Beams. The strong axis beam, beam no. 1, required stiffeners ½ × 6½ × 12½. The weak axis beams no. 3 and no. 4 require plates A ¾ × 8 × 12½. These plates occupy the same space because the beams are all of the same depth. Therefore, the larger of the two plates is used, as shown in Fig. 2.33*c*.

Since the stiffeners are merged, the welds that were earlier determined for the strong axis beam must be revisited.

Weld to Web. From the worst case of Fig. 2.36,

$$D_w = \frac{\sqrt{29^2 + 107^2}}{2 \times 11.0 \times 1.392} = 3.59$$

Use a ¼ fillet weld or AISC minimum.

Weld to Flanges. From the worst case of Fig. 2.36,

$$D_f = \frac{\sqrt{29^2 + 93^2}}{2 \times 6.25 \times 1.392} = 5.60$$

This indicates a ⅜ fillet weld is required.

In the above weld size calculations, the worst case of gravity loads and lateral loads is used. If it is known that one or the other only exists, only that cases need be considered. When it is not known whether the loads are gravity or lateral, the worst case presumed here must be used.

Note also, that is the weld size calculations, the AISC Specification Section J2.4, which allows for increased strength of obliquely loaded fillet welds, is not used. The compatibility requirements associated with obliquely loaded fillets of different sizes in the same group are complex and are not considered here.

Stresses in Stiffeners (Plate A). The weak axis beams are G50 steel and are butt welded to plates A. Therefore, plates A should also be G50 steel. Previous calculations involving this plate assumed it was A36, but changing to G50 will not change the final results in this case because the stiffener contact force is limited by the beam no. 1 delivered force rather than the stiffener strength.

The stiffener stresses for the flange welds are, from Fig. 2.36 (worst case),

$$f_v = \frac{93}{0.75 \times 6.25} = 19.8\text{ ksi} < 1.0 \times 0.6 \times 50 = 30\text{ ksi, ok}$$

$$f_a = \frac{29}{0.75 \times 6.25} = 6.19\text{ ksi} < 0.9 \times 50 = 45\text{ ksi, ok}$$

and for the web welds

$$f_v = \frac{29}{0.75 \times 11} = 3.5\text{ ksi} < 30\text{ ksi, ok}$$

$$f_a = \frac{107}{0.75 \times 11} = 13.0\text{ ksi} < 45\text{ ksi, ok}$$

2.3.2.3 Associated Shear Connections—Beams 3 and 4. The specified shear for these beams is R = 107 kips.

Weld-to-Beam Web. As with the strong axis beam web connection, this is a field-welded connection with bolts used for erection only. The design load (required strength) is R = 107 kips. The beam web shear R is essentially constant in the area of the connection and is assumed to act at the edge of plate A (Section a-a of Fig. 2.33*b*). This being the case, there will be a small eccentricity on the C-shaped field weld. Following AISC 15th Edition Manual Table 8-8, l = 17, kl = 4, k = 0.24, x = 0.04, xl = 0.68, al = 4.25 – 0.68 = 3.57, and a = 0.21. From Table 8-8 by interpolation, c = 2.80, and the weld size required is

$$D = \frac{107}{0.75 \times 2.80 \times 17} = 3.00$$

which indicates that a 3/16 fillet weld is required.

Plate B (Shear Plate) Gross Shear. Try a ⅜ plate of A36 steel. Then

$$\phi R_v = 1.0 \times 0.6 \times 36 \times 0.375 \times 17 = 138\text{ kips} > 107\text{ kips, ok}$$

Weld of Plate B to Column Web. This weld carries all of the beam shear R = 107 kips. The length of this weld is 17.75 in. Thus

$$D=\frac{107}{2\times17.75\times1.392}=2.17$$

A 3/16 fillet weld is indicated. Because this weld occurs on both sides of the column web, the column web thickness should satisfy the relationship $0.75 \times 0.6 \times 65t_w \geq 1.392 \times D \times 2$ or $t_w > 0.207$. Since the column web thickness is 0.485 in, the web can support the 3/16 fillets. The same result can be achieved using AISC LRFD Manual Table 10-2.

Weld of Plate B to Plates A. There is a shear flow $q = VQ/I$ acting on this interface, where $V = R = 107$ kips, Q is the statical moment of plate A with respect to the neutral axis of the I section formed by plates A as flanges and plate B as web. Thus

$$I=\frac{1}{12}\times0.375\times19.25^3+0.75\times12.5\times\left(\frac{19.25+0.75}{2}\right)^2\times2$$

$$=2100\text{ in}^4$$

$$Q=0.75\times12.5\times10=93.8\text{ in}^3$$

and

$$q=\frac{107\times93.8}{2100}=4.78\text{ kips/in}$$

Thus,

$$D=\frac{4.78}{2\times1.392}=1.72$$

Since plate A is ¾ in thick, the AISC minimum fillet weld is ¼ in.

The total shear flow force acting on plate A is 4.78 × 6.25 = 29.9 kips. This force does not affect the welds of stiffener A to the column. Rather, stiffener A can be considered an extension of the beam flange, and the shear flow force is taken as part of the flange force. Since the beam flange is full penetration welded to the stiffener A, no further analysis is required.

2.4 SHEAR CONNECTIONS

2.4.1 Introduction

Shear connections are the most common type of connections on every job. They are generally considered to be "simple" connections in that the beams supported by them are "simple" beams, that is, no bending moment at the beam ends. There are two basic types of shear connections, framed and seated.

2.4.2 Framed Connections

These are the familiar double-angle, single-angle, single-shear plate, and shear end-plate connections. They are called *framed connections* because they connect beams, web-to-web, directly. Figure 2.37*a* shows a typical double-angle connection and Fig. 2.37*b* shows a shear end-plate connection. These and other types of framed connections can be easily designed using the design aids (charts, tables) contained in the AISC *Manual of Steel Construction*. A shear end-plate and single plate shear connection will be designed in detail in the next two examples. The other types are designed in a similar manner.

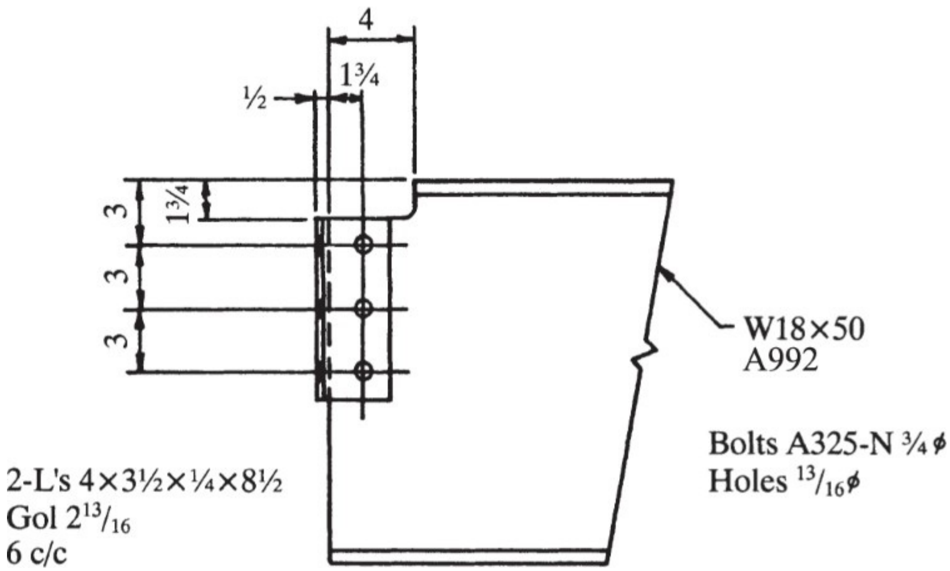


FIGURE 2.37*a* Double-angle framed connection.

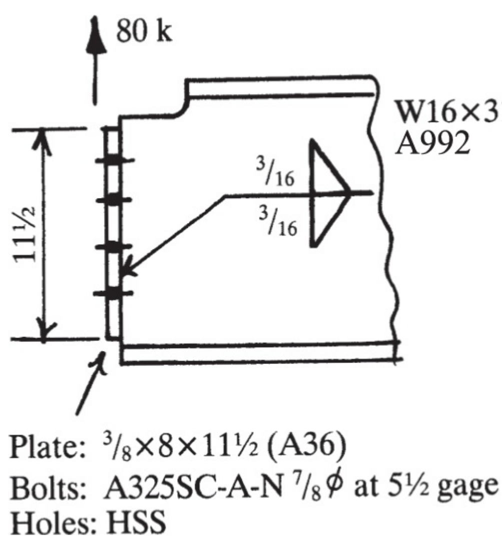


FIGURE 2.37*b* Shear end-plate connection.

Example: Shear End Plate Design One of the principal uses of shear end-plate connections is for skewed connections. Suppose the W16 beam of Fig. 2.37*b* is skewed 9½° (a 2 on 12 bevel) from the supporting beam or column as shown in Fig. 2.38. The nominal weld size is that determined from the analysis with the plate perpendicular to the beam web (Fig.

2.38*a*). This is denoted W', where $W' = 2.6/16 = 0.1625$. The effective throat for this weld is $t_e = 0.7071\ W' = 0.707(0.1625) = 0.115$ in. If the beam web is cut square, the gap on the obtuse side is 0.275sin(9.5) < 1/16, so it can be ignored.

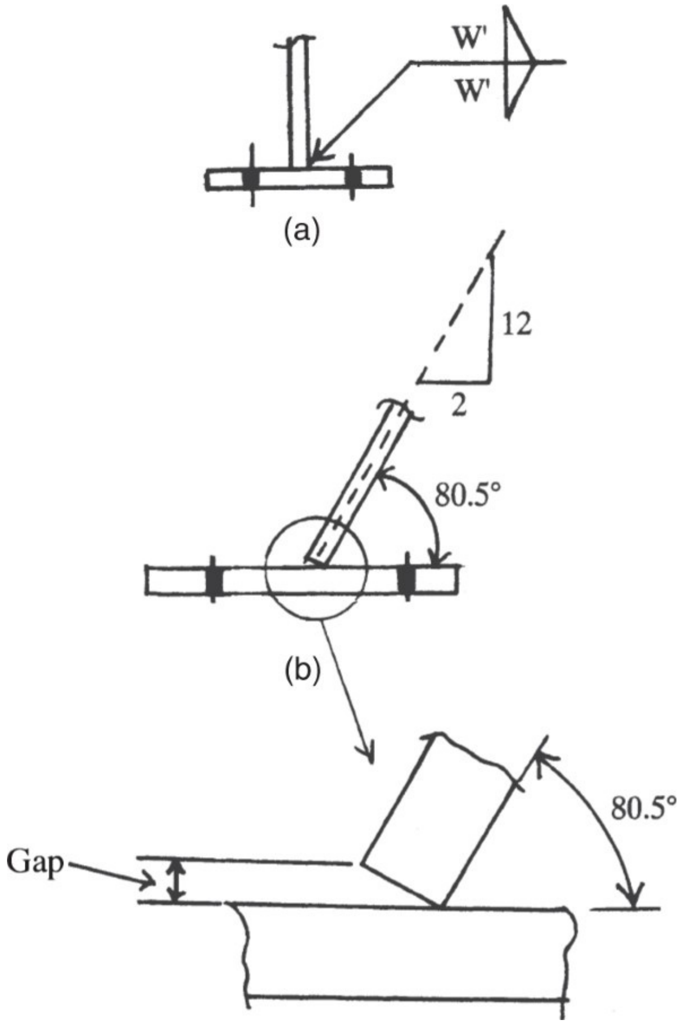


FIGURE 2.38 Geometry of skewed joint.

$$W = t_e\left(\frac{2\sin\Phi}{2}\right) + g$$

The weld size, W, for a skewed weld is

where Φ is the dihedral angle.

For the obtuse side, $\Phi = 90 + 9.5 = 99.5$,

$$W = 0.115\left(\frac{2\sin(99.5)}{2}\right) + 0 = 0.1755; \frac{3}{16} \text{ fillet weld}$$

For the acute side, $\Phi = 90 - 9.5 = 80.5$,

$$W = 0.115\left(\frac{2\sin(80.5)}{2}\right) + 0 = 0.1486; \frac{3}{16} \text{ fillet weld}$$

In this case, the fillet sizes remain the same as the orthogonal case. In general, the obtuse side weld will increase and the acute side weld will decrease, as will be seen in the next section.

2.4.3 Skewed Connections

The shear end-plate example of the previous section ended with the calculation of welds for a skewed connection. There are many types of skewed connections. The design recommendations for economy and safety have been reviewed by Kloiber and Thornton (1997). This section is largely taken from that paper.

Skewed Connections to Beams. The preferred skewed connections for economy and safety are single plates (Fig. 2.39) and end plates (Fig. 2.40). Single bent plates (Fig. 2.41) and eccentric end plates also work well at very acute angles. The old traditional double bent plate connections are difficult to accurately fit and are expensive to fabricate. There are also quality (safety) problems with plate cracking at the bend line as the angle becomes more acute.

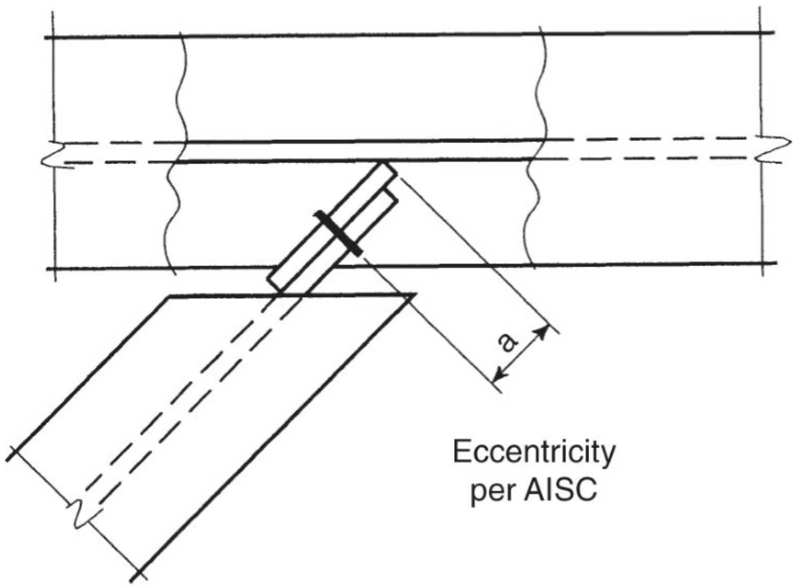


FIGURE 2.39 Shear tab (single plate). *(Courtesy of Kloiber and Thornton, with permission from ASCE.)*

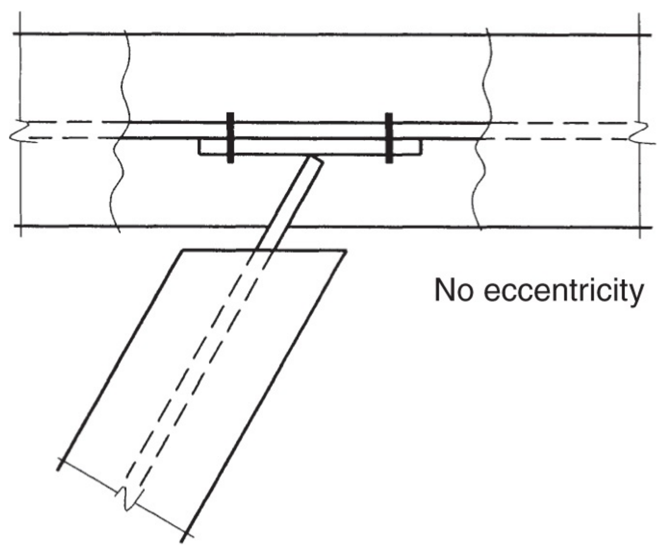


FIGURE 2.40 Shear end plate. *(Courtesy of Kloiber and Thornton, with permission from ASCE.)*

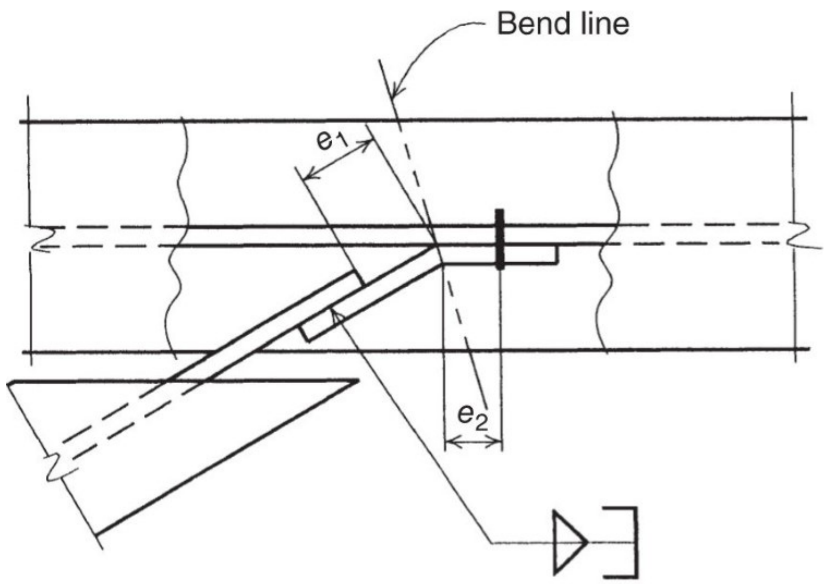


FIGURE 2.41 Bent plate. *(Courtesy of Kloiber and Thornton, with permission from ASCE.)*

Single plates (Fig. 2.39) are the most versatile and economical skewed connection with excellent dimensional control when using short slotted holes. While capacity is limited, this is usually not a problem because skewed members generally carry smaller tributary area. Single plates can be utilized for intersection angles of 90° to 30°. Traditionally, snug-tight bolts were preferred because they were more economical and greatly simplified installation. However, the advantages of TC bolt installation often make it more economical to pretension the bolts, though, since the bolts are not required to be pretensioned, no preinstallation verification is required for these connections. There are AISC 15th Edition Manual (2016) tables available, which can be used to select the required plate size and bolts along with the weld capacity for the required load. This connection has an eccentricity related to the parameter, *a*, of Fig. 2.39. The actual eccentricity depends on support rigidity, hole type, and bolt installation. The actual weld detail, however, has to be developed for the joint geometry. Welding details for skewed joints were discussed in Sec. 1.3.7.

End plates (Fig. 2.40) designed for shear only are able to provide more capacity than single plates and if horizontal slots are utilized with shug-tight bolts in bearing some dimension adjustment is possible. Holes gages can be adjusted to provide bolt access for more acute skews. A constructability problem can arise when there are opposing beams that limit access to the back side of the connection. These end-plate connections can be sized using the AISC (2016) tables to select plate size, bolts, and weld capacity. Note that there is no eccentricity with this joint. The weld detail, however, has to be adjusted for the actual geometry of the joint in a manner similar to the shear plate.

Single bent plates as in Fig. 2.41 can be sized for either welded connections using the procedures in the AISC *Manual of Steel Construction* for single angle connections. These involve two eccentricities, e_1 and e_2, from the bend line.

Eccentric end plates (Fig. 2.42) can be easily sized for the eccentricity, *e*, using the tables in the AISC *Manual of Steel Construction* for eccentric bolt groups.

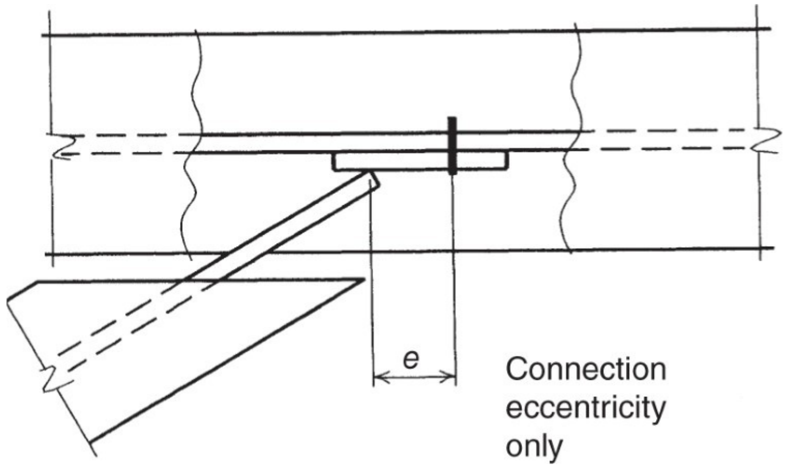


FIGURE 2.42 Eccentric end plate. *(Courtesy of Kloiber and Thornton, with permission from ASCE.)*

Skewed Connections to Columns. Skewed connections to wide-flange columns present special problems. Connections to webs have very limited access, and except for columns where the flange width is less than the depth, or for skews less than 30°, connections to flanges are preferred.

When connecting to column webs, it may be possible to use either a standard end plate or eccentric end plate as shown in Figs. 2.43 and 2.44. Single-plate connections should not be used unless the bolts are positioned outside the column flanges. In such cases, the connection should be checked as an extended shear tab as outlined later in this chapter.

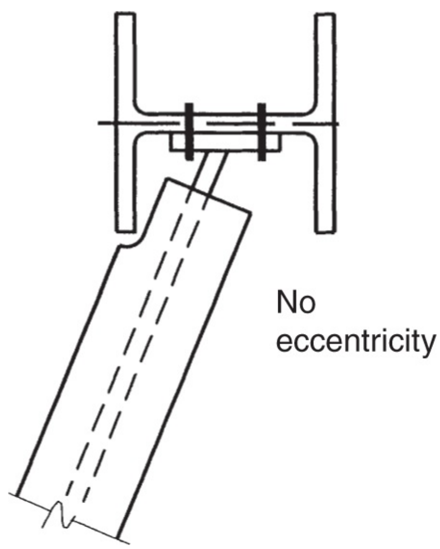


FIGURE 2.43 End plate. *(Courtesy of Kloiber and Thornton, with permission from ASCE.)*

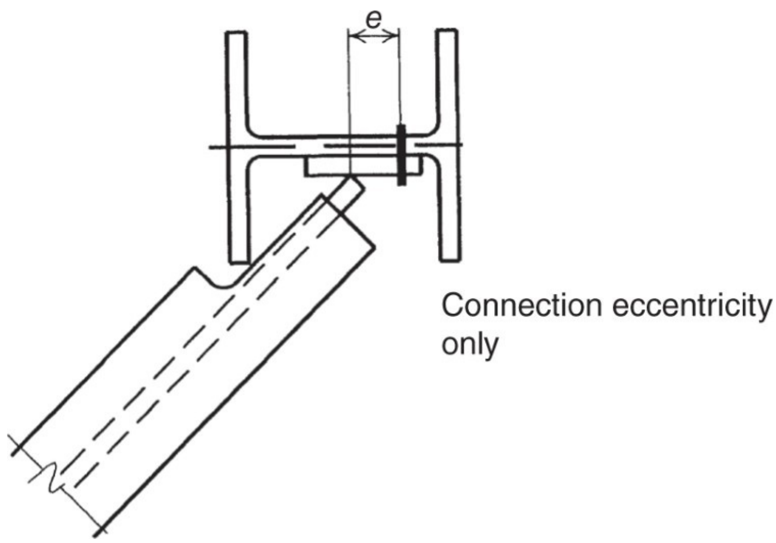


FIGURE 2.44 Eccentric end plate. *(Courtesy of Kloiber and Thornton, with permission from ASCE.)*

Skewed connections to column flanges will also be eccentric when the beam is aligned to the column centerline. However, if the beam alignment is centered on the flange, as shown in Fig. 2.45, the minor axis eccentricity is eliminated and the major axis eccentricity will not generally govern the column design. The connection eccentricity is related to the parameter, a, here in the same way as was discussed for Fig. 2.39.

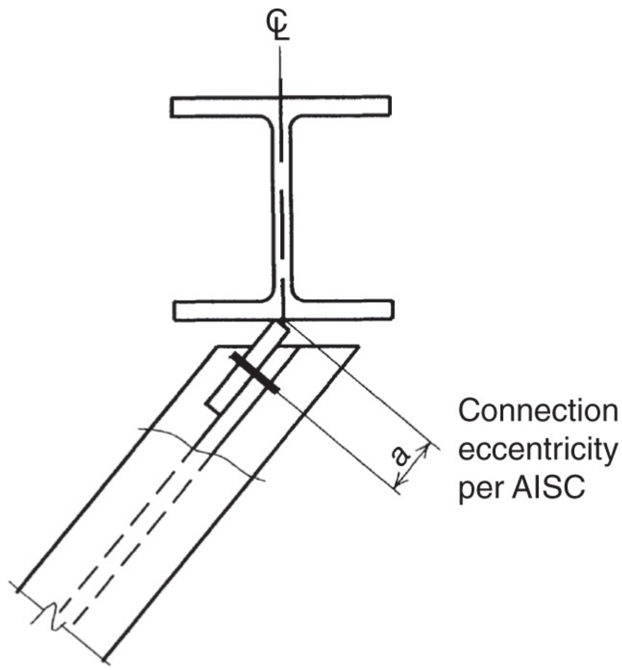


FIGURE 2.45 Single plate (extended shear tab). *(Courtesy of Kloiber and Thornton, with permission from ASCE.)*

When the beam is aligned to the column centerline, single plates (Fig. 2.46), eccentric end plates (Figs. 2.47 and 2.48), or single bent plates (Fig. 2.49) can be used. The eccentricity for each of these connections is again similar to that for the same connection to a beam web. An additional eccentricity, e_y, which causes a moment about the column weak axis, is present in these connections as shown in Figs. 2.46 through 2.49. The column may need to be designed for this moment.

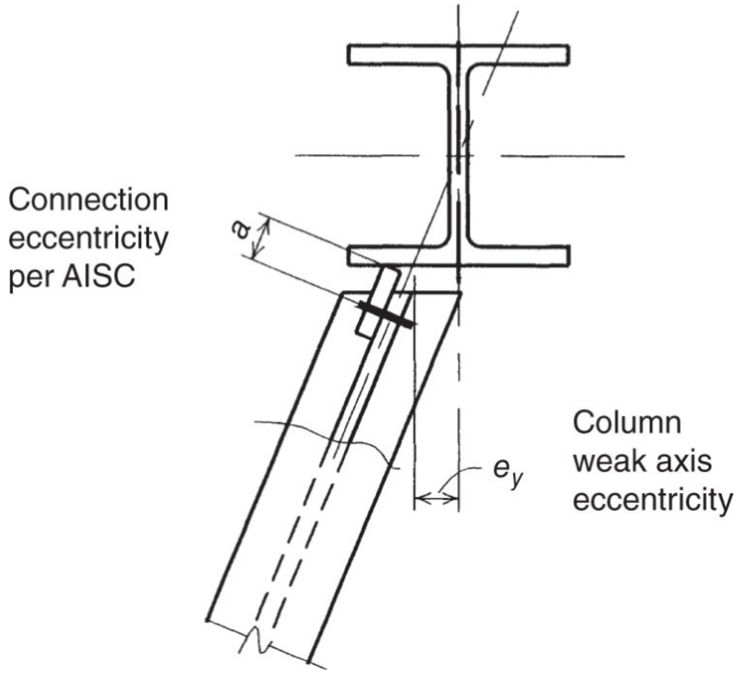


FIGURE 2.46 Single-plate (shear tab) gravity axis configuration. *(Courtesy of Kloiber and Thornton, with permission from ASCE.)*

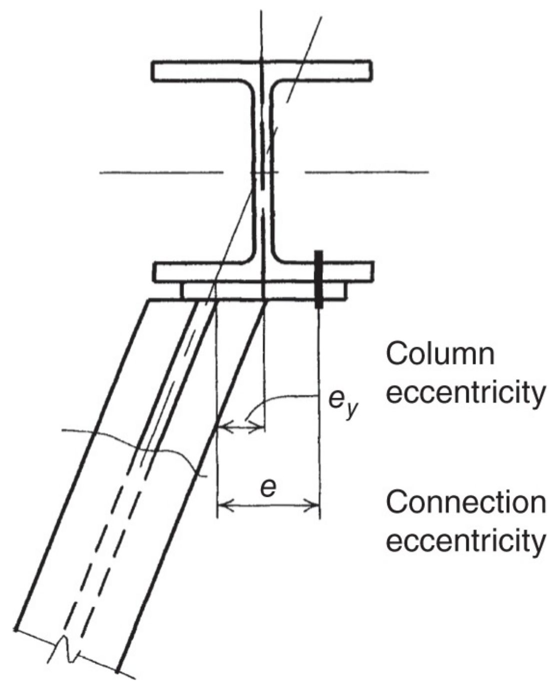


FIGURE 2.47 Eccentric shear end plate gravity axis configuration. *(Courtesy of Kloiber and Thornton, with permission from ASCE.)*

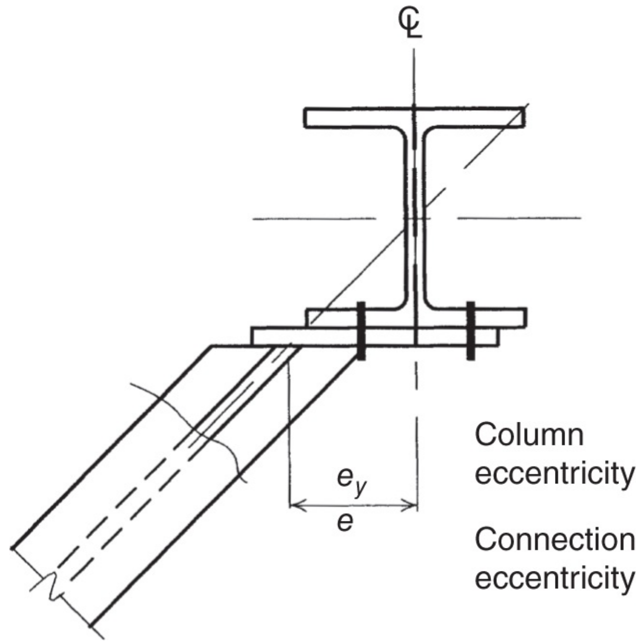


FIGURE 2.48 Eccentric shear end plate for high skew. *(Courtesy of Kloiber and Thornton, with permission from ASCE.)*

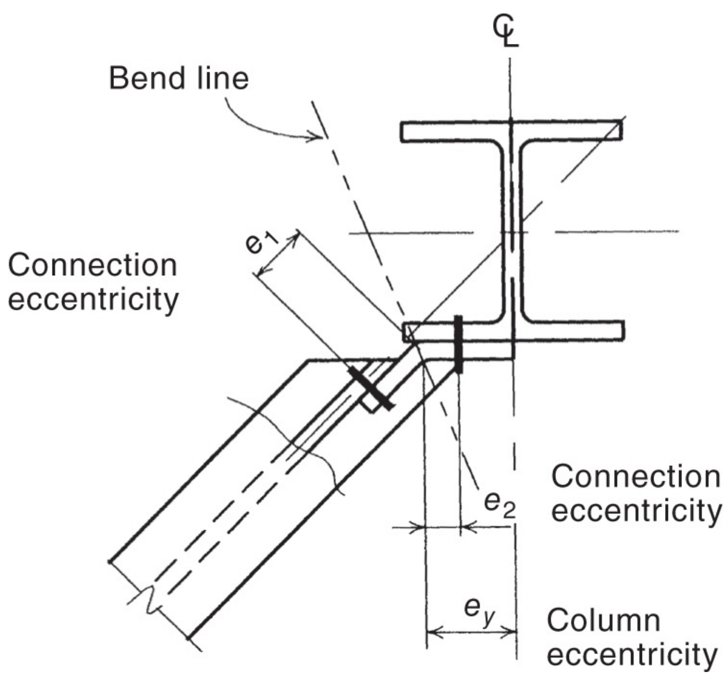


FIGURE 2.49 Single bent plate—one beam framing to flange. *(Courtesy of Kloiber and Thornton, with permission from ASCE.)*

A special skewed connection is often required when there is another beam framing to the column flange at 90°. If the column flange is not wide enough to accommodate a side-by-side connection, a bent plate can be shop welded to the column with matching holes for a second beam as shown in Fig. 2.50. The plate weld is sized for the eccentricity, e_2, plus any requirement for development of fill plate in the orthogonal connection, and the column sees an eccentric moment due to e_y, which is equal to e_2 in this case.

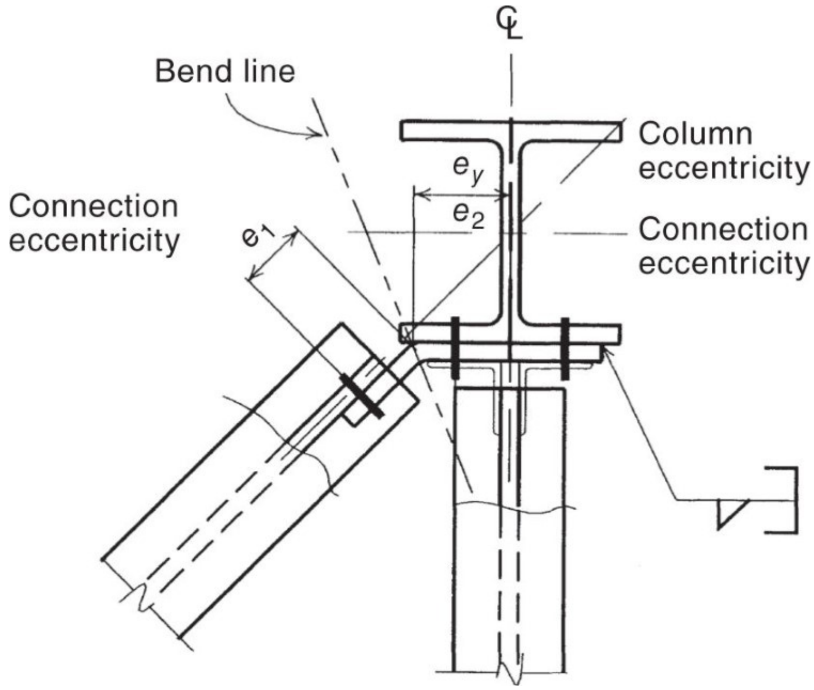


FIGURE 2.50 Single bent plate—two beams. *(Courtesy of Kloiber and Thornton, with permission from ASCE.)*

2.4.4 Seated Connections

The second type of shear connection is the seated connection, either unstiffened or stiffened (Fig. 2.51). As with the framed connections, there are tables in the *Manual of Steel Construction*, which aid in the design of these connections.

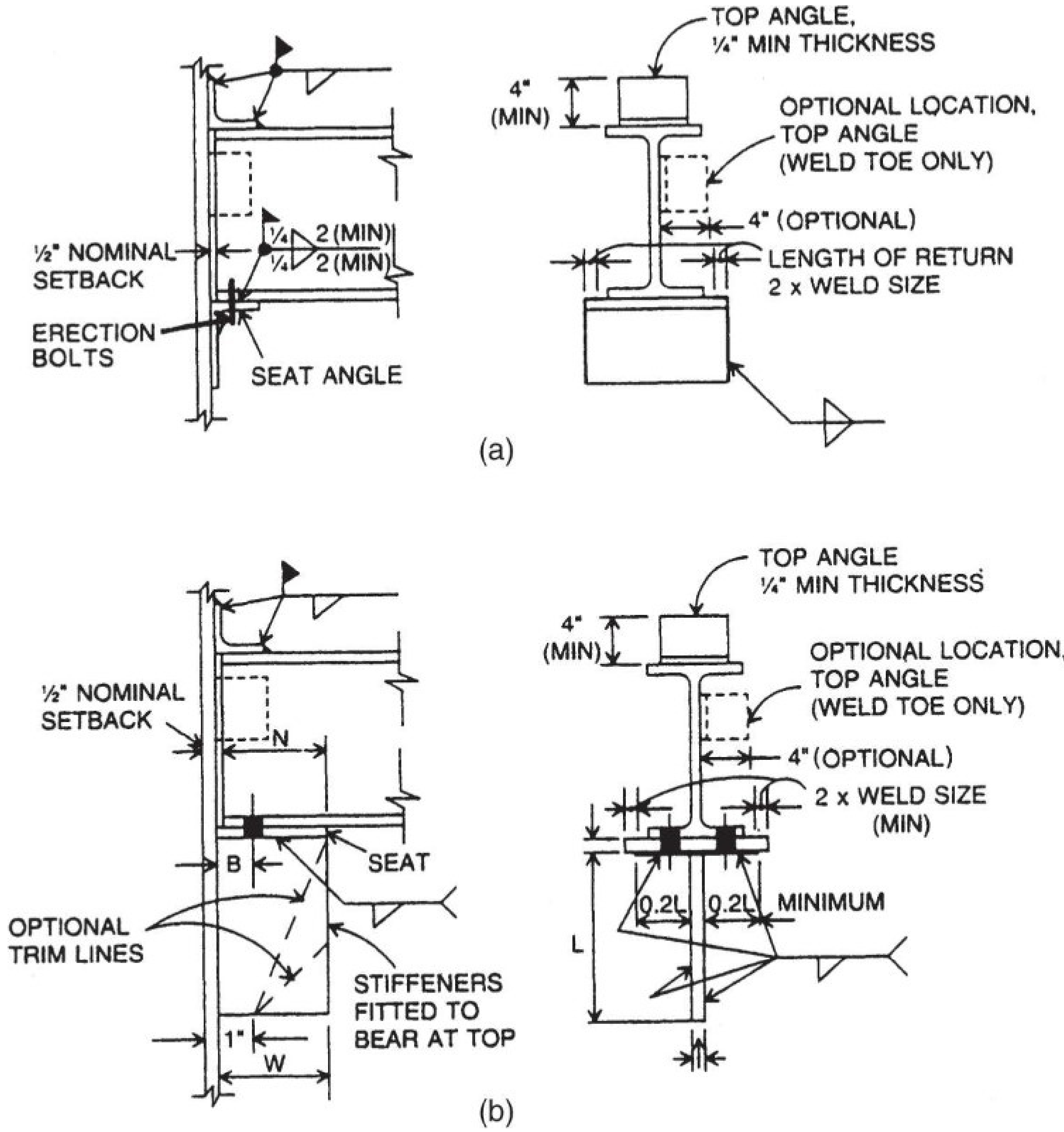


FIGURE 2.51 Standardized weld seat connections: (*a*) unstiffened seat and (*b*) stiffened seat.

The primary use for this connection is for beams framing to column webs. In this case, the seat is inside the flange or nearly so, and is not an architectural problem. It also avoids the erection safety problems associated with most framed connections where the same bolts support beams on both sides of the column web.

When a seat is attached to one side of the column web, the column web is subjected to a local bending pattern because the load from the beam is applied to the seat at some distance, *e*, from the face of the web. For stiffened seats, this problem was addresses by Sputo and Ellifrit (1991). The stiffened seat design tables (Tables 10-7 and 10-8) in the AISC 13th Edition *Manual of Steel Construction* reflect the results of their research. For unstiffened seats, column web bending also occurs, but no research has been done to determine its effect. This is the case because the loads and eccentricities for unstiffened seats are much smaller than for stiffened seats. Figure 2.52 presents a yield-line analysis that can be used to assess the strength of the column web. The nominal capacity of the column web is

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

where the terms are defined in Fig. 2.52, and

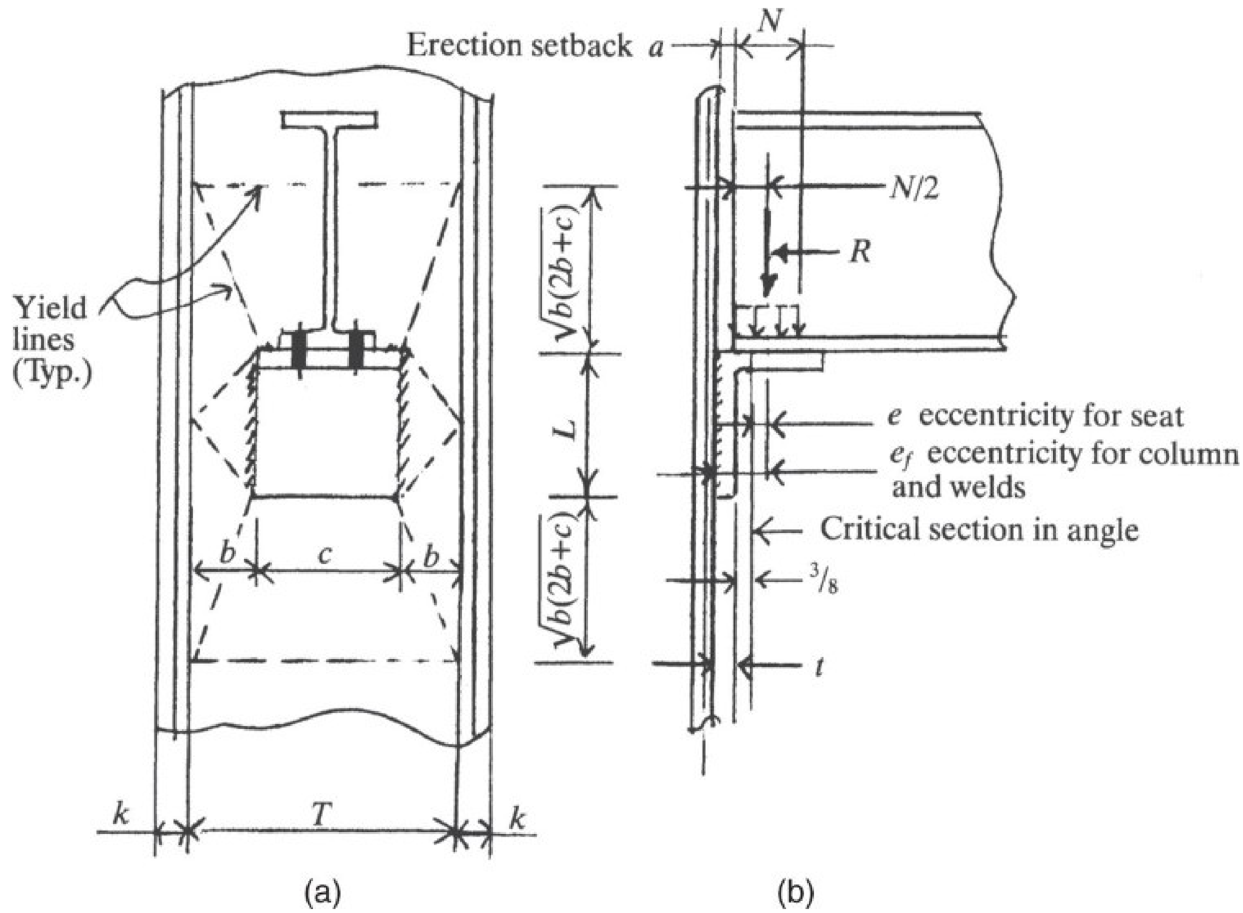


FIGURE 2.52 Column web yield lines and design parameters for unstiffened seated connection.

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

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

Since this is a yield limit state, $\phi = 0.9$ and $\Omega = 1.67$.

Example A W14 × 22 beam (A992) is to be supported on an unstiffened seat to a W14 × 90 column (A992). The given reaction (required strength) is 33 kips. Design the unstiffened seat.

The nominal erection set back a = ½ in. For calculations, to account for underrun, use a = ¾ in. Try a seat 6 in long (c = 6). In order to use Table 10-6 from the AISC *Manual of Steel Construction*, the required bearing length, N, must first be determined. Note that N is not the horizontal angle leg length less a, but rather it cannot exceed this value. The bearing length for an unstiffened seat starts at the end on the beam and spreads from this point, because the toe of the angle leg tends to deflect away from the bottom flange of the beam. The bearing length cannot be less than k and can be written in a general way as

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

where R_1 through R_6 are defined in the AISC *Manual of Steel Construction* pp. 9–48, and are tabulated in Table 9-4. For the W14 × 22,

$$\phi R_1 = 21.1, \phi R_2 = 11.5, \phi R_3 = 23.1, \phi R_4 = 2.86, \phi R_5 = 20.4, \phi R_6 = 3.82$$

Thus

$$N = \max\left\{\frac{33-21.1}{11.5}, \frac{33-23.1}{2.86} \text{ or } \frac{33-20.4}{3.82}, 0.735\right\}$$
$$= \max\{1.04, 3.46 \text{ or } 3.30, 0.735\} = 3.46 \text{ or } 3.30$$

Therefore, N is either 3.46 or 3.30 depending on whether $N/d < 0.2$ or $N/d > 0.2$, respectively. With $d = 13.7$, 3.46/13.7 = 0.253, and 3.30/13.7 = 0.241. Since clearly $N/d > 0.2$, $N = 3.30$ in.

It was stated earlier that $(N + a)$ cannot exceed the horizontal angle leg. Using a = ½ + ¼ = ¾, $N + a$ = 3.30 + 0.75 = 4.05, which establishes a required horizontal leg equal to at least 4 in.

The AISC *Manual of Steel Construction* Table 10-6 does not include required bearing lengths greater than 3¼ in. However, extrapolating beyond the table, it would seem that a 1-in angle would be an appropriate choice. Since there is no L6 × 4 × 1 available, use a 6 × 6 × 1. The extra length of the horizontal leg is irrelevant. Table 10-6 indicates that a 5/16 fillet weld of the seat vertical leg (the 6-in leg) to the column web is satisfactory (40.9 kips). Consider this to be a preliminary design, which needs to be checked.

The design strength of the seat angle critical section is

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

where the terms are defined in Fig. 2.52. From Fig. 2.52, $e_f = N/2 + a$ = 3.30/2 + 0.75 = 2.41 and $e = e_f - t - 0.375$= 2.41 – 1 – 0.375 = 1.04, e = 6. Then

$$\phi R_b = 0.9(36)\frac{(6)(1.0)^2}{4(1.04)} = 46.7 \text{ kips} > 33 \text{ kips, ok}$$

The weld sizes given in Table 10-6 will always be conservative because they are based on using the full horizontal angle leg minus a as the bearing length, N. The detailed check will be performed here for completeness. Using the eccentric weld Table 8-4 with $ex = ef = 2.41$, l = 6, a = 2.41/6 = 0.40, C is determined to be 2.81. The strength of the weld is calculated as

$$\phi R_{\text{weld}} = 0.75(2.81)(5)(6) = 63.2 \text{ kips} > 33 \text{ kips, ok}$$

Finally, checking the column web,

$$m_p = \frac{50(0.44)^2}{4} = 2.42 \frac{\text{kip-in}}{\text{in}}$$

$$T = 11.25$$

$$c = 6$$

$$L = 6$$

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

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

$$= 77.6 \text{ kips} > 33 \text{ kips, ok}$$

This completes the calculations for the example. The final design is shown in Fig. 2.53.

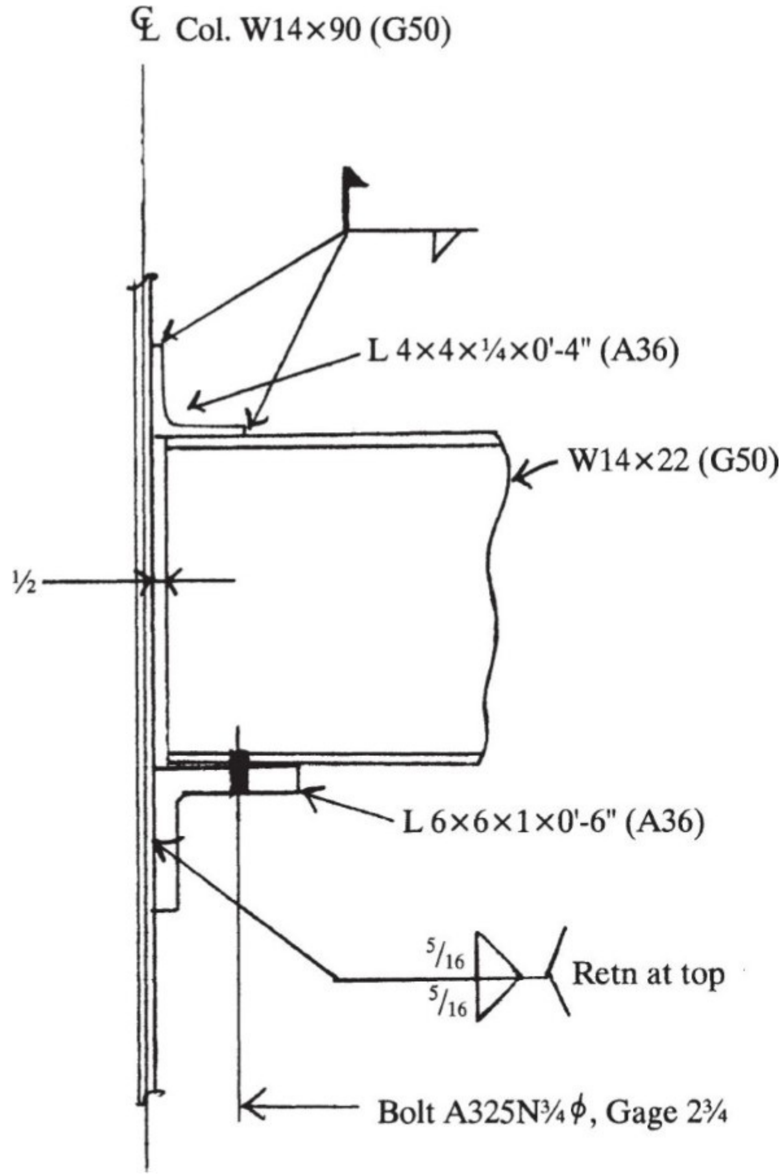


FIGURE 2.53 Unstiffened seat design.

2.4.5 Beam Shear Splices

If a beam splice takes moment as well as shear, it is designed with flange plates in a manner similar to the truss chord splice treated in Sec. 2.2.5.2. The flange force is simply the moment divided by the center-to-center flange distance for inside and outside plate connections, or the moment divided by the beam depth for outside plate connections. The web connection takes any shear. Two typical shear splices are shown in Fig. 2.54. These are common in cantilever roof construction. Figure 2.54*a* shows a four-clip angle splice. The angles can be shop bolted (as shown) or shop welded to the beam webs. The design of this splice is exactly the same as that of a double-angle framing connection. The shear acts at the faying surface of the field connection and each side is designed as a double-angle framing connection. If shop bolted all the bolts are in shear only; there is no eccentricity considered on the bolts. If shop welded, the shop welds see an eccentricity from the location of the shear at the field faying surface to the centroids of the weld groups. This anomaly is historical. The bolted connections derive from riveted connections, which were developed before it was considered necessary to satisfy “the niceties of structural mechanics” according to McGuire (1968).

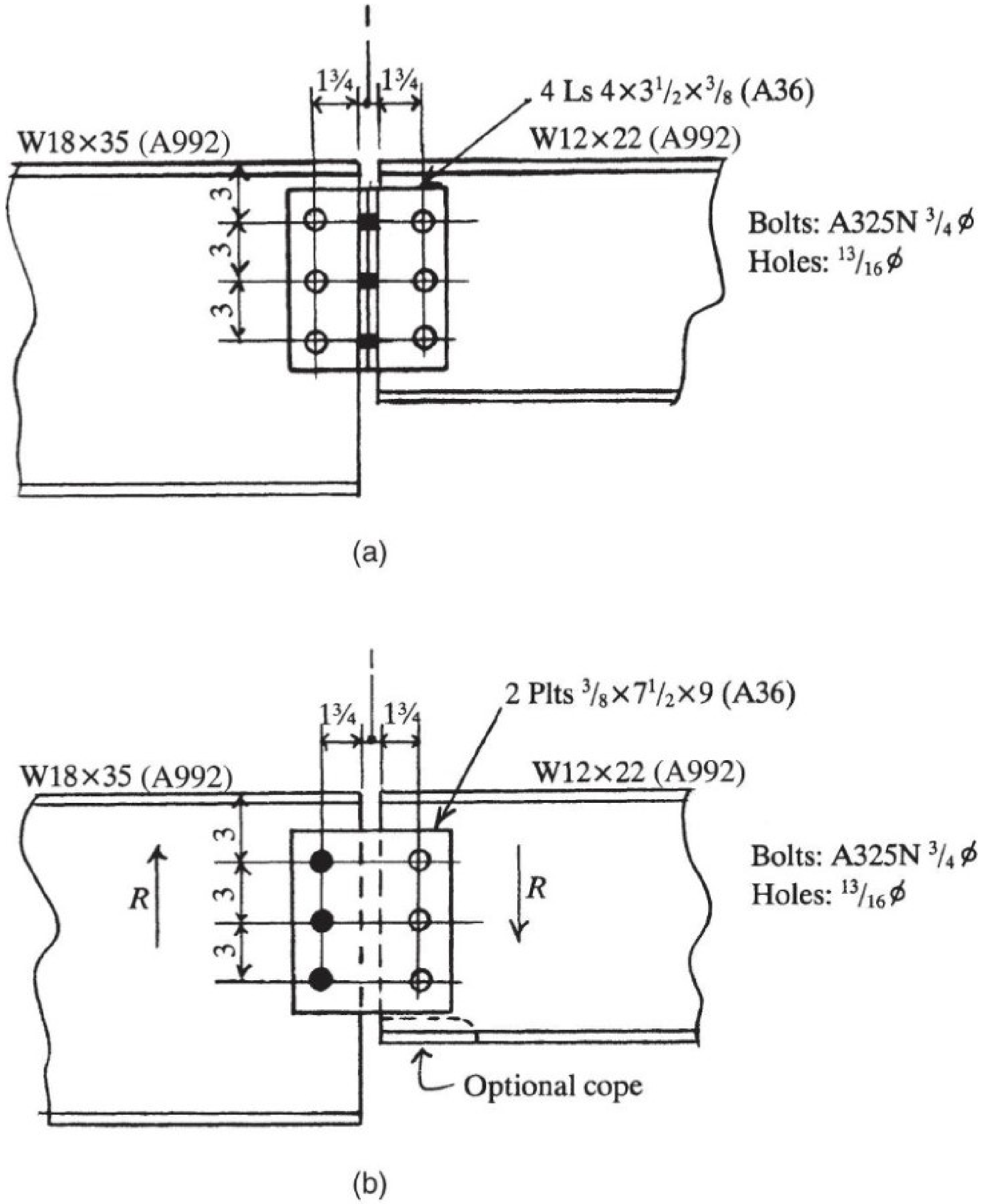


FIGURE 2.54 Typical shear splices: (*a*) shear splice with four angles and (*b*) shear splice with one or two plates.

A second type of shear splice uses one or two plates in place of the four angles. This type, shown in Fig. 2.54*b*, has moment capacity, but has been used for many years with no reported problems. It is generally less expensive than the angle type. Because it has moment capability, eccentricity on the bolts or welds cannot be neglected. It has been shown by Kulak and Green (1990) that if the stiffness on both sides of the splice is the same, the eccentricity is one-half the distance between the group centroids, on each side of the splice. This will be the case for a shop-bolted–field-bolted splice as shown in Fig. 2.54*b*. A good discussion on various shear splice configurations and the resulting eccentricities is given in the AISC *Manual of Steel Construction* (2016).

Example As an example of the design routine for the Fig. 2.54*b* splice, its capacity (design strength) will be calculated.

Bolts. Since the strength of the bolt group will be determined using Manual Table 7-7 and the direction and magnitude of the force on each bolt will not be known, bolt tearout will be determined based on the worst possible case. This is conservative. A more exact value can be obtained by applying the instantaneous center of rotation method to determine the magnitude and direction of the forces on the individual bolts.

The design "bolt value" will be the minimum of the bolt shear, bearing, and tearout:

Bolt shear

$$\phi r_v = (2)17.9 = 35.8 \text{ kips/bold}$$

Since the W12 × 22 has the thinner web, it will be checked for bearing and tearout

$$\text{Bearing at W12} \times 22 \text{ web}$$
$$\phi r = 0.75(2.4)(65)(0.75)(0.260) = 22.8 \text{ kips/bold}$$

Tearout at W12 × 22 web (assuming a maximum optional cope depth of 1½ in)

$$\phi r = 0.75(1.2)(65)\left(1.75 - \frac{0.813}{2}\right)(0.260) = 20.4 \text{ kips/bolt}$$

Bearing and tearout at splice plates does not govern by inspection.

From Table 7-7, for ex = 2.25 and $n = 3$, $C = 2.11$. Therefore,

$$2.11(20.4) = 43.0 \text{ kips}$$

Neglecting the tearout check as would have been done *prior* to the 3rd of the AISC LRFD *Manual of Steel Construction*, the bolt group capacity would have been 2.11(22.8) = 48.1 kips. The instantaneous center of rotation method assumes that the bolt is the weakest element. However, when the capacity of the group is limited instead by the strength of the connected material, an alternative force distribution can produce an increased calculated capacity (Thornton and Muir, 2004). If the capacity of the bolt group is optimized, the calculated capacity, considering bolt tearout, becomes 46 kips, still a considerable decrease from the capacity neglecting the tearout limit states, but a considerable increase from the 43 kips capacity that results from the worst case.

2.4.6 Extended Single-Plate Shear Connections (Shear Tabs)

Single-plate shear connections can be very economical connections. In-fill beams can be drilled on the fabricator's drill line with no further handling, since the beams will require none of the coping required for more traditional beam-to-beam connections. Beam-to-column-web connections are also made easier. Since the beam can be connected beyond the column flanges erection is greatly eased. Unlike double angle, end plate and sometimes single angle connections, there will be no common bolts at the support, so safety is also improved.

Example: Extended Single Plate Tab Connection (See Fig. 2.55)

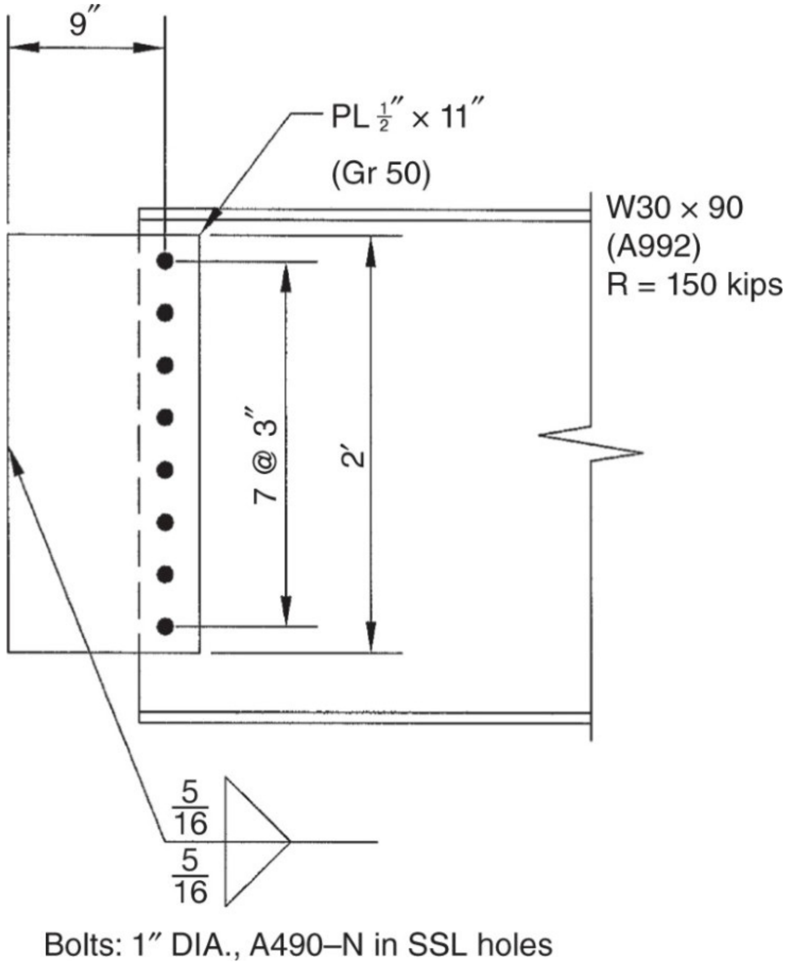


FIGURE 2.55 Extended single plate connection.

Inelastic bolt design. (From AISC 13th Edition Manual Table 7-7)

$$\begin{aligned} C &= 4.34 \\ \phi R_n &= C \times \phi r_n \\ &= 4.34 \times 40.0 \\ &= 174 \geq 150 \text{ kips, ok} \end{aligned}$$

Bearing/Tearout On Controlling Element
Bearing/Tearout Does Not Control

Maximum plate thickness: Due to the uncertainty related to the distribution of moments through the connection the plate and bolt group are sized such that yielding in the plate will preclude fracture of the bolts by redistributing the moments. It should be noted that this check uses the nominal bolt capacity without a factor of safety and discounts the 20% reduction in bolt shear strength assumed in the Specification to account for uneven force distribution in end-loaded connections. Since this is essentially a ductility check and not a strength limit state, this should not be considered a violation of the Specification.

Calculating the bolt value as described above:

$$R_n = \frac{\phi_v r_n (1.25)}{\phi_v} = \frac{40.0(1.25)}{0.75} = 66.7 \text{ kips/bolt}$$

$$\begin{aligned} M &= R_n \times C' \\ &= 66.7 \times 44.5 \\ &= 2970 \text{ kips-in} \end{aligned}$$

$$t_{\max} = \frac{6M}{F_y L^2} = \frac{6(2970)}{(50)(24)^2} = 0.619 \geq 0.5 \text{ ok}$$

Gross shear and bending interaction on plate

First the plate is checked to ensure buckling does not control

$$\lambda = \frac{L\sqrt{F_y}}{t_p\sqrt{47{,}500 + 112{,}000\left(\frac{L}{2a}\right)^2}}$$

$$= \frac{24\sqrt{50}}{0.5\sqrt{47{,}500 + 112{,}000\left[\frac{24}{2(9)}\right]^2}} = 0.683 \leq 0.7$$

Therefore, buckling does not control

$$\phi R_n = \frac{\phi F_y t_p L}{\sqrt{16\left(\frac{a}{L}\right)^2 + 2.25}}$$

$$= \frac{0.9(50)(0.5)(24)}{\sqrt{16\left(\frac{9}{24}\right)^2 + 2.25}} = 255 \text{ kips} \geq 150 \text{ kips, ok}$$

Net shear on plate

$$\phi R_n = \phi 0.6 F_y t_p \left[L - n\left(\phi_b + \frac{1}{16}\right)\right]$$

$$= (0.75)0.6(65)(0.5)[24 - 8(1.25)] = 204 \text{ kips} \geq 150 \text{ kips, ok}$$

Weld size required: Note the weld size is required to be ⅝ of the plate thickness to ensure that the plate yields and redistributes load prior to weld fracture.

$$w = \frac{5}{8} t_p = \frac{5}{8}(0.5) = 0.3125 \text{ use 5/16 in weld}$$

Block shear on plate

$$A_{nt} = t_p \left[L_e - 0.5 \left(\phi_h + \frac{1}{16} \right) \right]$$

$$= 0.5 \left[2 - 0.5 \left(1.3125 + \frac{1}{16} \right) \right] = 0.656 \text{ in}^2$$

$$A_{gv} = t_p [L_e + (n-1)b] = 0.5[1.5 + (7)3] = 11.3 \text{ in}^2$$

$$A_{nv} = t_p \left[L_e + (n-1)b - (n-0.5) \left(\phi_h + \frac{1}{16} \right) \right]$$

$$= 0.5[1.5 + (7)3 - (7.5)(1.25)] = 6.56 \text{ in}^2$$

$$\phi R_{bs} = \phi(0.6 F_u A_{nv} + U_{bs} F_u A_{nt}) \le \phi(0.6 F_y A_{gv} + U_{bs} F_u A_{nt})$$

$$= 0.75[0.6(65)(6.56) + 1.0(65)(0.656)]$$

$$= 224 \le 0.75[0.6(50)(11.3) + 1.0(65)(0.656)] = 284$$

$$= 224 \text{ kips} \ge 150 \text{ kips, ok}$$

It is generally assumed that beams are torsionally supported at their ends. Lack of torsional support can substantially reduce the flexural capacity of beams that are otherwise laterally unsupported. Generally, the torsional stiffness of end connections to beams that are fully braced by a diaphragm, such as a slab or a deck, is not an issue. However, though the AISC Specification does not contain a check for torsional stiffness, end connections for beams that are not laterally supported should be checked. The check presented here is based on Australian requirements, which assume lateral support only at the applied mid-span load. Assuming the W30 × 90 has a span of 28 ft:

$$k_s \ge 448{,}000 \frac{J}{L} \left[1 + \left(\frac{b_f d}{t_f L} \right)^2 \right]$$

$$\frac{3730(24)(0.5)^3}{9.0} \ge 448{,}000 \frac{2.84}{336} \left[1 + \left(\frac{(10.4)(29.5)}{(0.610)(336)} \right)^2 \right]$$

$$1240 \frac{\text{kips-in}}{\text{radian}} < 12300 \frac{\text{kips-in}}{\text{radian}}$$

Therefore, the beam cannot be considered to be torsionally restrained by the extended shear tab.

$$t_p = \sqrt[3]{\frac{12300(9.0)}{3730(24)}} = 1.07 \text{ in}$$

In order to provide sufficient torsional restraint, the shear tab thickness would need to be.

Interestingly, even a standard shear tab may not provided adequate torsional restraint in this instance. The required thickness of a standard tab with a 3-in distance from the bolt line to the weld can be calculated as:

$$t_p = \sqrt[3]{\frac{12,300(3.0)}{3730(24)}} = 0.744 \text{ in}$$

and at least a ¾ in shear tab would be required.

2.5 MISCELLANEOUS CONNECTIONS

2.5.1 Simple Beam Connections under Shear and Axial Load

As its name implies, a simple shear connection is intended to transfer shear load out of a beam while allowing the beam to act as a simply supported beam. The most common simple shear connection is the double-angle connection with angles shop bolted or welded to the web of the carried beam and field bolted to the carrying beam or column. This section, which is from Thornton (1995a), will deal with this connection.

Under shear load, the double-angle connection is flexible regarding the simple beam end rotation, because of the angle leg thickness and the gage of the field bolts in the angle legs. The AISC 13th Edition Manual, p. 10-9 recommends angle thicknesses not exceeding ⅝ in with the usual gages. Angle leg thicknesses of ¼ to ½ in are generally used, with ½-in angles usually being sufficient for the heaviest shear load. When this connection is subjected to axial load in addition to the shear, the important limit states are angle leg bending and prying action. These tend to require that the angle thickness increase or the gage decrease, or both, and these requirements compromise the connection's ability to remain flexible to simple beam end rotation. This lack of connection flexibility causes a tensile load on the upper field bolts, which could lead to bolt fracture and a progressive failure of the connection and the resulting collapse of the beam. It is thought that there has never been a reported failure of this type, but is perceived to be possible.

Even without the axial load, some shear connections are perceived to have this problem under shear alone. These are the single-plate shear connections (shear tabs) and the Tee framing connections. Recent research on the Tee framing connections (Thornton, 1996) has led to a formula (AISC 13th Edition Manual, pp. 9-13, 9-14) which can be used to assess the resistance to fracture (ductility) of double-angle shear connections. The formula is

$$d_{b_{\min}} = 0.163t\sqrt{\frac{F_y}{\tilde{b}}\left(\frac{\tilde{b}^2}{L^2}+2\right)}$$

where $d_{b_{min}}$ = the minimum bolt diameter (A325 bolts) to preclude bolt fracture under a simple beam end rotation of 0.03 radians

$\underset{\sim}{t}$ = the angle leg thickness

b = the distance from the bolt line to the k distance of the angle (Fig. 2.56)

L = the length of the connection angles

Note that this formula can be used for ASD and LRFD designs in the form given here. It can be used to develop a table (Table 2.1) of angle thicknesses and gages for various bolt diameters which can be used as a guide for the design of double-angle connections subjected to shear and axial tension. Note that Table 2.1 validates AISC's long-standing (AISC, 1970) recommendation (noted above) of a maximum ⅝-in angle thickness for the "usual" gages. The usual gages would be 4½ to 6½ in. Thus, for a carried beam web thickness of, say, ½ in, GOL will range from 2 to 3 in. Table 2.1 gives a GOL of 2½ in for ¾-in bolts (the most critical as well as the most common bolt size). Note also that Table 2.1 assumes a significant simple beam end rotation of 0.03 radian, which is approximately the end rotation that occurs when a plastic hinge forms at the center of the beam. For short beams, beams loaded near their ends, beams with bracing gussets at their end connections, and beams with light shear loads, the beam end rotation will be small and Table 2.1 does not apply.

TABLE 2.1 Estimated Minimum Angle Gages (GOL) for A36 Angles and A325 Bolts for Rotational Flexibility

	Minimum gage of angle (GOL)[a]		
Angle thickness (in)	¾-in-diameter bolt (in)	⅞-in-diameter bolt (in)	1-in-diameter bolt (in)
⅜	1⅜	1¼	1⅛
½	1⅞	1⅝	1½
⅝	2½	2⅛	1⅞
¾	3¼	2-11/16	2-5/16
1	6	4-5/16	3½

[a]Driving clearances may control minimum GOL.

As an example of a double-clip angle connection, consider the connection of Fig. 2.57. This connection is subjected to a shear load of 33 kips and an axial tensile load of 39 kips.

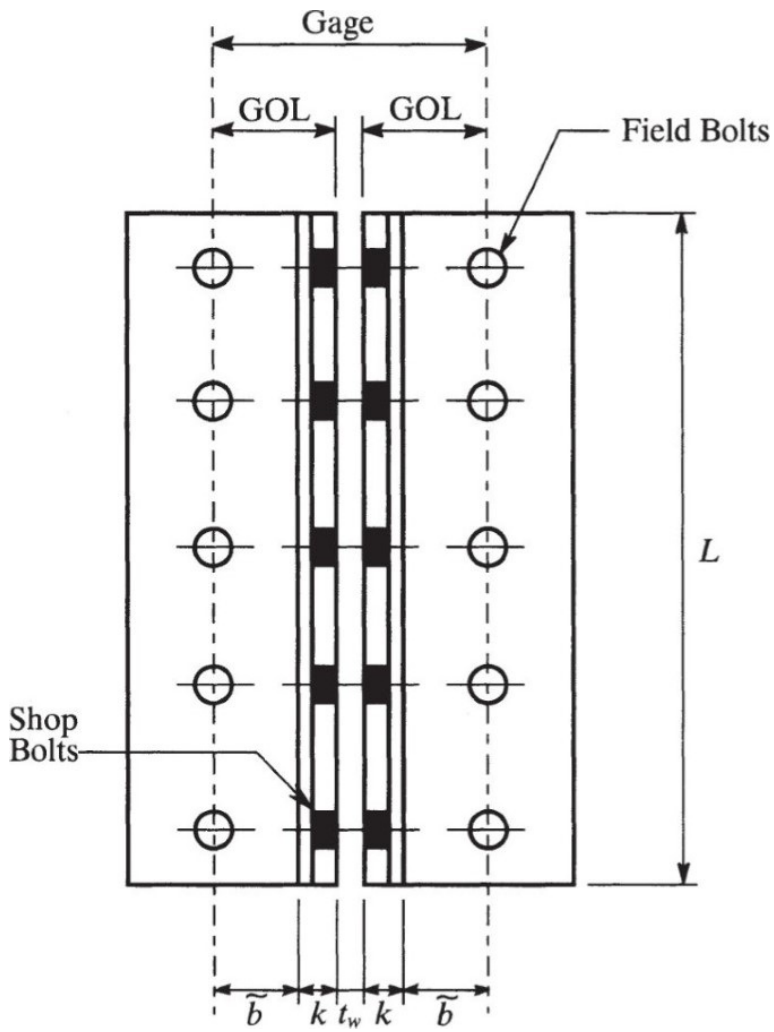


FIGURE 2.56 Geometry of double angles (shop-bolted shown).

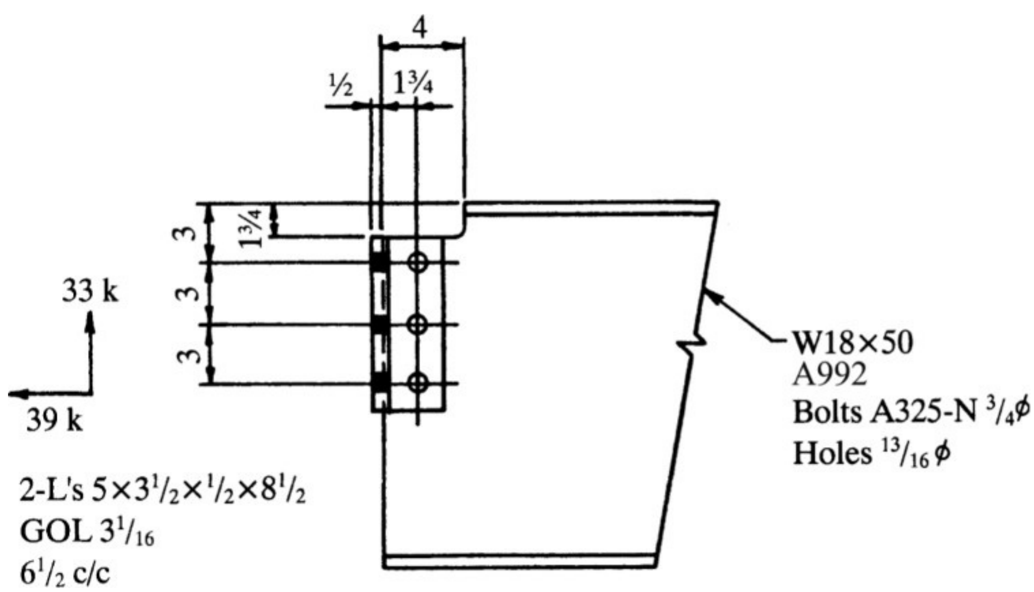


FIGURE 2.57 Framed connection subjected to axial and shear loads.

Shop Bolts. The shop bolts "see" the resultant load $R = \sqrt{33^2 + 39^2} = 51.1$ kips. The design shear strength of one bolt is $\phi r_v = 17.9$ kips in single shear, and 35.8 kips in double shear.

The beam web will govern the bearing and tearout values. The bearing strength is

$$\phi r_p = 0.75(2.4)(0.75)(0.355)(65) = 31.2 \text{ kips}$$

If the loads of 33 kips shear and 39 kips axial always remain proportional, that is, maintain the bevel of 10⅛ to 12 as shown in Fig. 2.58, the spacing requirement is irrelevant because there is only one bolt in line of force and the true edge distance is 1.94 or 2.29 in. The clear distance, L_c, is

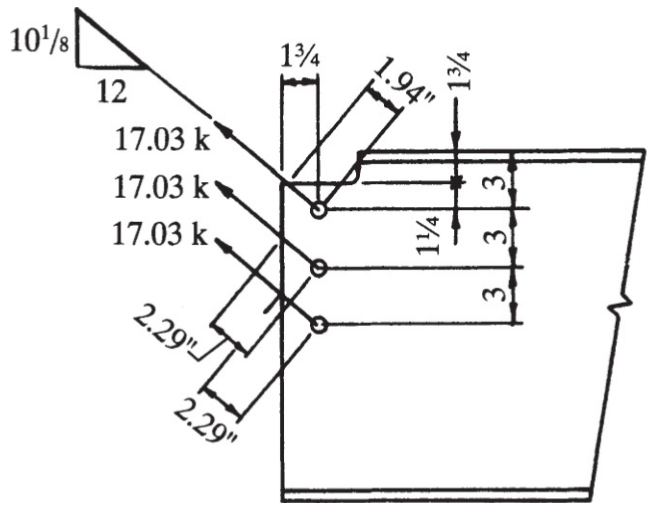


FIGURE 2.58 Edge distances along the line of action.

$$L_c = 1.94 - (0.8125/2) = 1.53 \text{ in}$$

In order to maintain equilibrium all bolts will be assumed to have the same strength based on this shortest edge distance the tearout capacity is

$$\phi r_v = 0.75(1.2)(1.53)(0.355)(65) = 31.8 \text{ kips}$$

The capacity of the bolt group is

$$\phi R_v = 3(31.2) = 93.6 \text{ kips} > 51.1 \text{ kips, ok}$$

Gross shear on clips is

$$\phi R_n = 1.0(0.6)(36)(8.5)(0.50)(2) = 184 \text{ kips} > 33 \text{ kips, ok}$$

Net shear on clips is

$$\begin{aligned}\phi R_n &= 0.75(0.6)(58)[8.5 - 3(0.875)](0.50)(2) \\ &= 154 \text{ kips} > 33 \text{ kips, ok}\end{aligned}$$

Block Shear Rupture (Tearout). A simple conservative way to treat block shear when shear and tension are present is to treat the resultant as a shear. Then, from Figs. 2.57 and 2.58,

$$A_{gv} = 7.25 \times 0.355 = 2.57 \text{ in}^2$$
$$A_{nv} = (7.25 - 2.5 \times 0.875) \times 0.355 = 1.80 \text{ in}^2$$
$$A_{gt} = 1.75 \times 0.355 = 0.621 \text{ in}^2$$
$$A_{nt} = (1.75 - 0.5 \times 0.875) \times 0.355 = 0.466 \text{ in}^2$$
$$F_u A_{nt} = 65 \times 0.466 = 30.3$$
$$\begin{aligned}\phi R_{bsv} &= 0.75\, [F_u A_{ntv} + \min(0.6 F_y A_{gtv}\, , 0.6 F_u A_{ntv}\,)] \\ &= 0.75\, [\, 30.3 + \min(0.6 \times 50 \times 2.57, 0.6 \times 65 \times 1.80)] \\ &= 75.4 \text{ kips} > 51.1 \text{ kips, ok}\end{aligned}$$

An alternate approach is to calculate a block shear rupture design strength under tensile axial load. From Fig. 2.60,

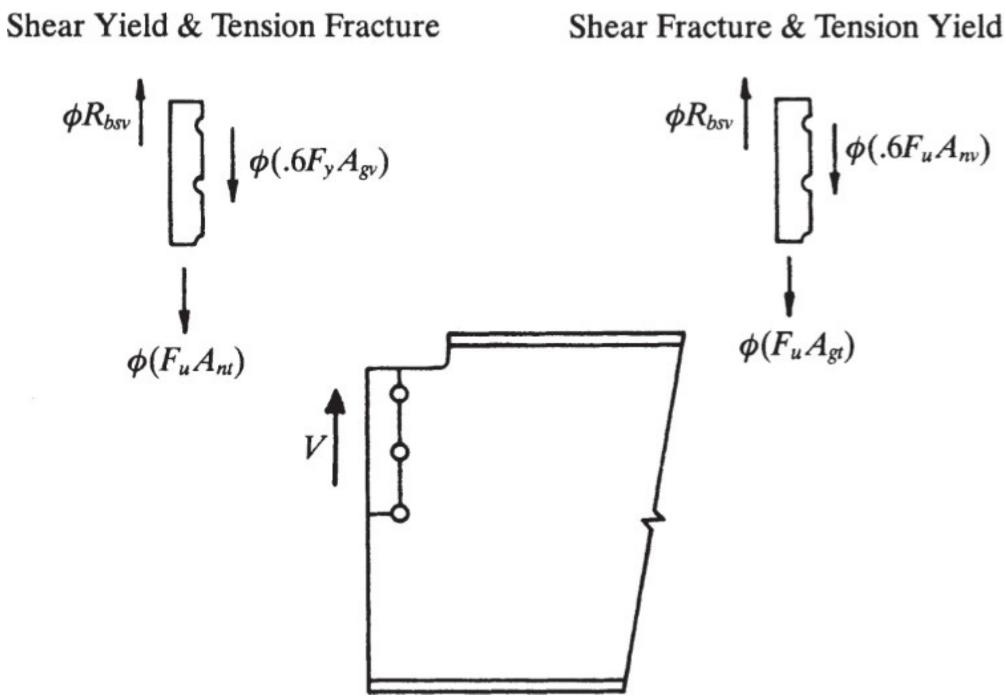


FIGURE 2.59 Block shear rupture under shear.

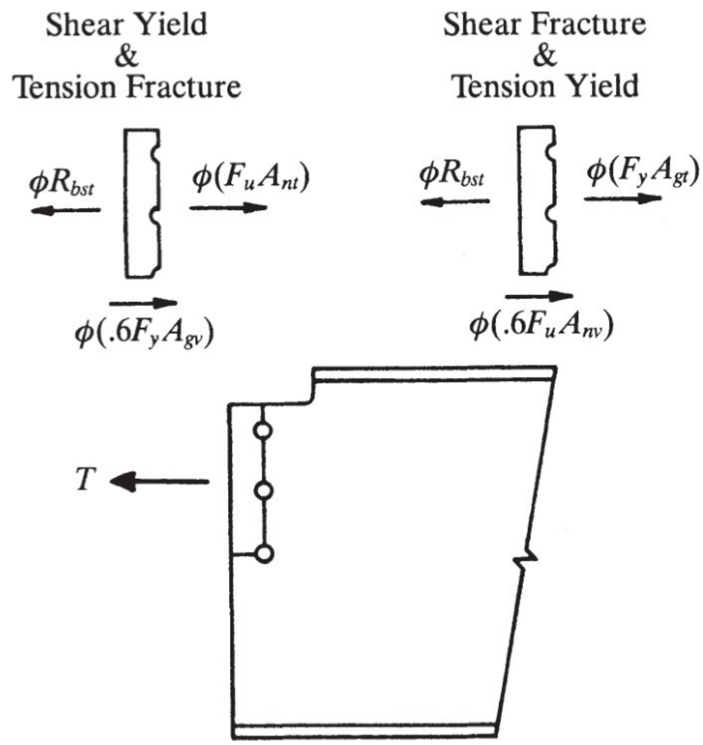


FIGURE 2.60 Block shear rupture under tension *T*.

$$
\begin{aligned}
A_{gv} &= 0.621 \text{ in}^2 \\
A_{nv} &= 0.466 \text{ in}^2 \\
A_{gt} &= 2.57 \text{ in}^2 \\
A_{nt} &= 1.80 \text{ in}^2 \\
F_u A_{nt} &= 65 \times 1.80 = 117 \\
\phi R_{bsv} &= 0.75\ [F_u A_{\text{ntv}} + \min(0.6F_y A_{\text{gtv}},\ 0.6F_u A_{\text{ntv}})] \\
&= 0.75\ [\ 117 + \min(0.6 \times 50 \times 0.621,\ 0.6 \times 65 \times 0.466)] \\
&= 101 \text{ kips}
\end{aligned}
$$

Using an elliptical interaction equation, which is analogous to the von Mises (distortion energy) yield criterion,

$$\left(\frac{V}{\phi R_{bsv}}\right)^2 + \left(\frac{T}{\phi R_{bst}}\right)^2 \leq 1$$

where V is the factored shear and T the factored tension. Then

$$\left(\frac{33}{75.4}\right)^2 + \left(\frac{39}{101}\right)^2 = 0.34 < 1 \text{ ok}$$

This interaction approach is always less conservative than the approach using the resultant $R = \sqrt{V^2 + T^2}$ as a shear because $\phi R_{bst} > \phi R_{bsv}$ for the geometries of the usual bolt positioning in double-angle connections with two or more bolts in a single vertical column. The resultant approach, being much simpler as well as conservative, is the method most commonly used.

Connection Angles. Figure 2.57 shows angles 5 × 3½ × ⅝, but assume for the moment that ¼ angles are to be checked. The shop legs are checked for the limit states of bearing, gross shear and gross tension, and net shear and net tension. Net shear rupture and net tension rupture will control over block shear rupture with the usual connection geometries, that is, 1¼ edge and 1¼ end distances. Since the sum of the clip angle thicknesses = 0.24 + 0.25 = 0.5 > 0.355, the beam web and not the shop legs of the clip angles will control.

Prying Action. The AISC LRFD Manual has a table to aid in the selection of a clip angle thickness.

The preliminary selection table, Table 15.1, indicates that a ⅝ angle will be necessary. Trying *L*s 5 × 3½ × ⅝, and following the procedure of the AISC Manual,

$$b = \frac{6.5 - 0.355 - 2 \times 0.625}{2} = 2.45$$

$$a = \frac{10.355 - 6.5}{2} = 1.93 (< 1.25 \times 2.45 = 3.06\ \text{ok})$$

$$b' = 2.45 - \frac{0.75}{2} = 2.08$$

$$a' = 1.93 + \frac{0.75}{2} = 2.31$$

$$\rho = \frac{2.08}{2.31} = 0.90$$

$$p = \frac{8.5}{3} = 2.83$$

$$\delta = 1 - 0.8125/2.83 = 0.71$$

The shear per bolt V = 33/6 = 5.5 kips <15.9 kips, ok. The tension per bolt T = 39/6 = 6.5 kips. Because of interaction,

$$\phi F_t' = 5\,0.75\left[1.3F_{nt} - \left(\frac{F_{nt}}{\phi F_{nv}}\right) f_v\right] \le \phi F_{nt}$$

With f_v = 5.5/0.4418 = 12.5 ksi,

$$\phi F_t' = 0.75\left[1.3 \times 90 - \left(\frac{90}{0.75 \times 54}\right) 12.5\right] = 66.9\ \text{ksi} < 67.5\ \text{ksi}$$

Use $\phi F_t' = 64.3$ ksi, and $\phi r_t' = 66.9 \times 0.4418 = 29.6$ kips/bolt. Since T = 6.5 kips < 28.4 kips, the bolts are satisfactory independent of prying action. Returning to the prying action calculation

$$t_c = \sqrt{\frac{4.44 \times 29.6 \times 2.08}{2.83 \times 58}} = 1.29 \text{ in}$$

$$\alpha' = \frac{1}{0.71 \times 1.90}\left[\left(\frac{1.29}{0.625}\right)^2 - 1\right] = 2.42$$

Since α′ = 2.42, use α′ = 1. This means that the strength of the clip angle legs in bending is the controlling limit state. The design strength is

$$T_d = 28.4\left(\frac{0.50}{1.60}\right)^2 (1 + 0.71) = 7.30 \text{ kips} > 6.5 \text{ kips, ok}$$

The *L*s 5 × 3½ × ½ are satisfactory.

Ductility considerations. The ⅝-in angles are the maximum thickness recommended by the AISC Manual, for flexible shear connections. Using the formula introduced at the beginning of this section,

$$d_{b_{\min}} = 0.163t\sqrt{\frac{F_y}{\tilde{b}}\left(\frac{\tilde{b}^2}{L^2} + 2\right)}$$

with $t = 0.625$, $F_y = 36$, $\tilde{b} = 3.0625 - 1.125 = 1.94$, $L = 8.5$

$$d_{b_{\min}} = 0.163 \times .625\sqrt{\frac{36}{1.94}\left(\frac{1.94^2}{8.5^2} + 2\right)} = 0.63 \text{ in}$$

Since the actual bolt diameter is 0.75 in, the connection is satisfactory for ductility.

As noted before, it may not be necessary to make this check for ductility. If the beam is short, is loaded near its ends, or for other reasons is not likely to experience very much simple beam end rotation, this ductility check can be omitted.

This completes the calculations for the design shown in Fig. 2.57.

2.5.2 Reinforcement of Axial Force Connections

It sometimes happens that a simple beam connection, designed for shear only, must after fabrication and erection be strengthened to carry some axial force as well as the shear. In this case, washer plates can sometimes be used to provide a sufficient increase in the axial capacity. Figure 2.61 shows a double-angle connection with washer plates that extend from the toe of the angle to the *k* distance of the angle. These can be made for each bolt, so only one bolt at a time need be removed, or if the existing load is small, they can be made to encompass two or more bolts on each side of the connection. With the washer plate, the bending strength at the "stem" line, section a-a of Fig. 2.61 is

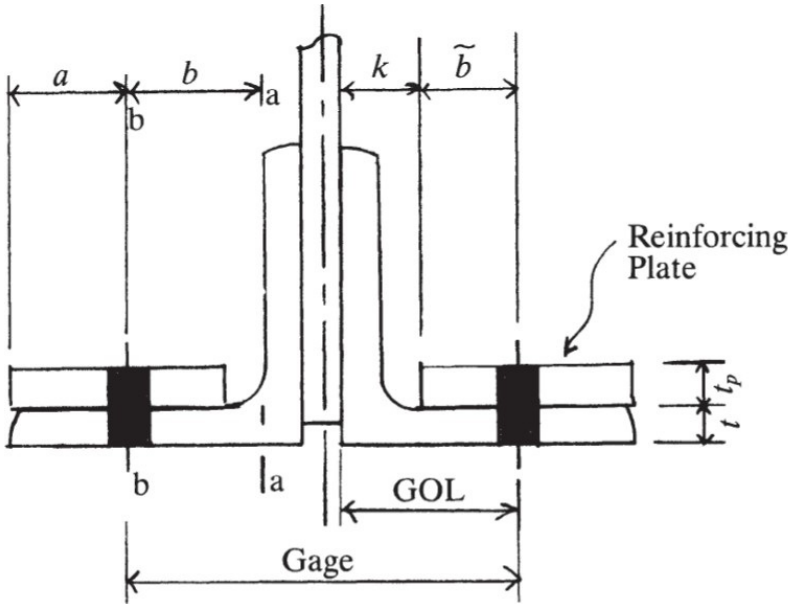


FIGURE 2.61 Prying action with reinforcing (washer) plate.

$$M_n = \frac{1}{4} F_u p t^2$$

while that at the bolt line, section b-b, is

$$M'_n = \alpha\delta \frac{1}{4} F_u p(t^2 + t_p^2) = \alpha\delta \frac{1}{4} F_u p t^2 \left(1 + \frac{t_p^2}{t^2}\right) = \alpha\delta\eta M_n$$

where $\eta = 1 + \left(\frac{t_p}{t}\right)^2$ and the remaining quantities are in the notation of the AISC 13th Edition Manual (2005). With the introduction of η, the prying action formulation of the AISC Manual can be generalized for washer plates by replacing δ wherever it appears by the term dη. Thus

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

and

$$T_d = \phi r_t \left(\frac{t}{t_c}\right)^2 (1 + \alpha'\delta\eta)$$

All other equations remain the same.

As an example of the application of this method, consider the connection of Fig. 2.62. Assume this was designed originally for a shear of 60 kips, but now must carry an axial force of 39 kips when the shear is at 33 kips. Let us check the axial capacity of this connection. The most critical limit state is prying action because of the thin angle leg thickness. From Fig. 2.62

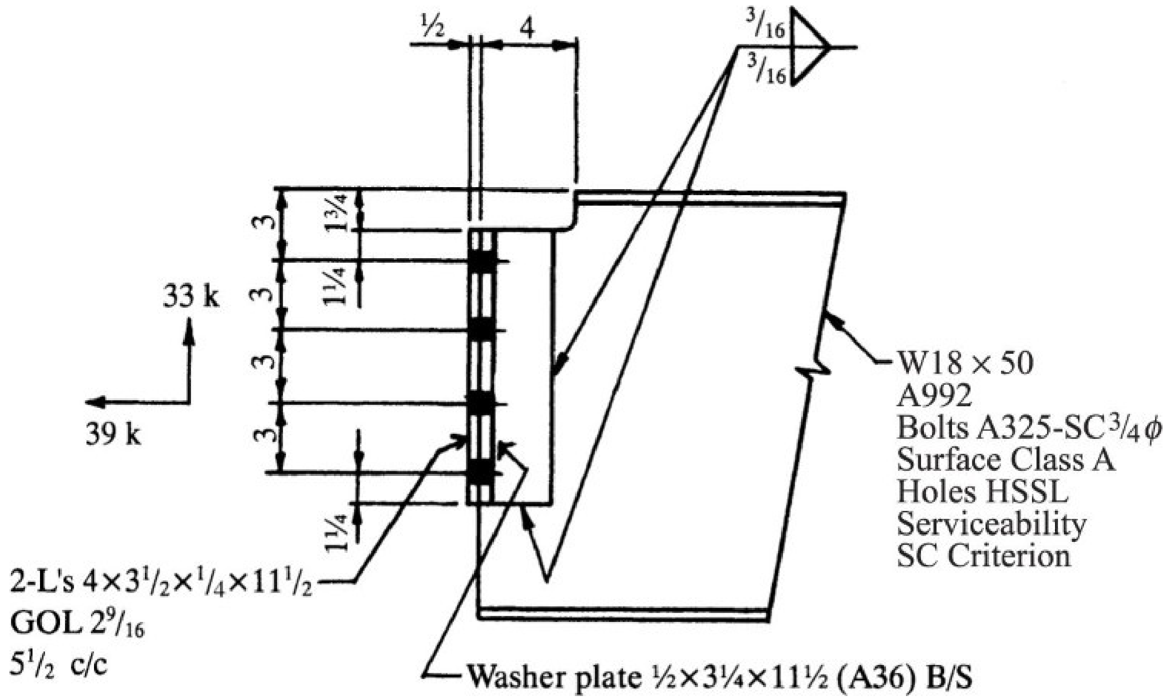


FIGURE 2.62 A shear connection needing reinforcement to carry axial load of 39 kips.

$$b = \frac{5.5 - 0.355 - 0.25}{2} = 2.45$$

$$a = \frac{8 + 0.355 - 5.5}{2} = 1.43$$

$$1.25 \times 2.45 = 3.06 > 1.43$$

Use a = 1.43. Then b' = 2.08, a' = 1.81, ρ = 1.15, δ = 0.72, V = 33/8 = 4.125 kips/bolt. The holes are HSSL (horizontal short slots), so ϕr_v = 9.41 kips/bolt. Since 4.125 < 9.41, the bolts are ok for shear (as they obviously must be since the connection was originally designed for 60 kips shear). Because this is a shear connection, the shear capacity is reduced by the tension load by the factor $1 - T/(1.13T_b)$, where T is the applied load per bolt and T_b is the specified pretension.

Thus, the reduced shear design strength is

$$\phi r_v' = 9.41\left(1 - \frac{39}{1.13 \times 28 \times 8}\right) = 7.96 \text{ kips/bolt} > 4.125 \text{ kips/bolt, ok}$$

Now, checking prying action, which includes the bending of the angle legs,

$$F_{nt}' = 1.3F_{nt} - \frac{F_{nt}}{\varphi F_{nv}} f_{rv} = 1.3 \times 90 - \frac{90}{0.75 \times 54}\left(\frac{4.125}{\pi \times 0.375^2}\right) = 96.3 \text{ ksi, use 90 ksi}$$

$$B = 0.75 \times 90 \times \pi \times 0.375^2 = 29.8 > 6.21 \text{ kips, ok}$$

$$t_C = \sqrt{\frac{4.44 \times 29.8 \times 2.08}{2.875 \times 58}} = 1.29$$

$$\alpha' = \frac{1}{0.72 \times 2.15}\left[\left(\frac{1.29}{0.25}\right)^2 - 1\right] = 15.8$$

Since α′ > 1, use α′ = 1, and

$$T_d = 29.8\left(\frac{0.25}{1.29}\right)^2 (1.72) = 1.93 \text{ kips} < 4.875 \text{ kips, no good}$$

Thus, the ¼-in angle legs fail. Try a ½-in washer plate. Then

$$\eta = 1 + \left(\frac{0.5}{0.25}\right)^2 = 5.00$$

$$\alpha' = \frac{1}{0.72 \times 5.00 \times 2.15}\left[\left(\frac{1.29}{0.25}\right)^2 - 1\right] = 3.31$$

Since α′ > 1 use α′ = 1

$$T_d = 29.8\left(\frac{0.25}{1.29}\right)^2 (1 + 0.72 \times 5.00) = 5.15 \text{ kips} > 4.875 \text{ kips, ok}$$

Therefore, the ½-in washer plates enable the connection to carry 5.15 × 8 = 41.2 kips > 39 kips, ok.

If ductility is a consideration, the ductility formula can be generalized to

$$d_{b_{\min}} = 0.163t\sqrt{\eta}\sqrt{\frac{F_y}{\tilde{b}}\left(\frac{\tilde{b}^2}{L^2} + 2\right)}$$

With $\tilde{b} = GOL - k = 2\frac{9}{16} - \frac{13}{16} = 1.75$

$$d_{b_{\min}} = 0.163 \times 0.25\sqrt{5.00}\sqrt{\frac{36}{1.75}\left(\frac{1.75^2}{11.5^2} + 2\right)} = 0.588 \text{ in} < 0.75 \text{ in, ok}$$

2.5.3 Extended Tab with Axial

An alternative to the connection shown in Sec. 2.5.1 would be to use an extended tab designed to carry the axial force as shown in Fig. 2.63.

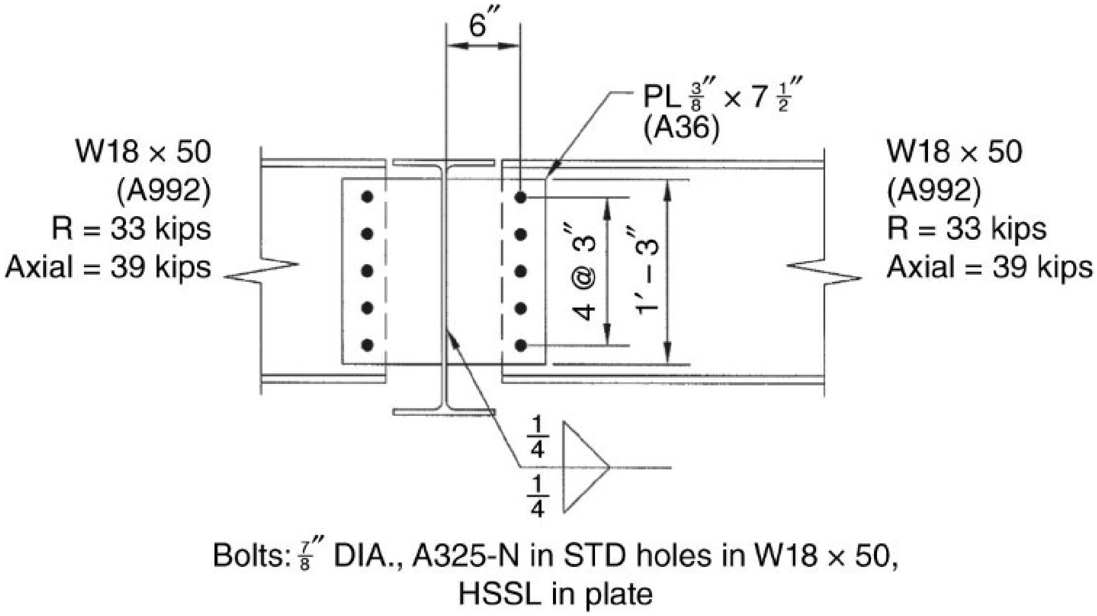


FIGURE 2.63 Extended single plate connection with axial load.

$$\text{Resultant load} = (V^2 + T^2)^{0.5} = 51.1 \text{ kips}$$
$$\phi = \tan^{-1}(39/33) = 49.8°$$

Inelastic Bolt Design. From AISC Manual Table 7-7 at 45° >, ex = 6, $n = 5$

$$\begin{aligned} C &= 2.88 \\ \phi R_n &= C \times \phi r_n \\ &= 2.88 \times 24.3 \\ &= 70.0 \geq 51.1 \text{ kips, ok} \end{aligned}$$

Bearing/Tearout on Controlling Element. Bearing/tearout does not control.

Maximum Plate Thickness. The transfer of the axial force is clear. However there are still uncertainties about the distribution of eccentricities so it is recommended to maintain the ductility requirement relating bolt strength to plate strength.

The ultimate bolt capacity can be calculated as

$$R_n = \frac{\phi_v r_n (1.25)}{\phi_v} = \frac{24.3(1.25)}{0.75} = 40.5 \text{ kips/bolt}$$

$$\begin{aligned} M &= R_n C' \text{ (From AISC Manual Table 7-7 at 0°) >, } C' = 17.1 \\ &= 40.5 \times 17 \\ &= 689 \text{ kips/in} \end{aligned}$$

$$t_{\max} = \frac{6M}{F_y L^2} = \frac{6(689)}{(36)(15)^2} = 0.510 \geq 0.375, \quad \text{ok}$$

Gross Shear, Axial, and Bending Interaction on Plate. Axial capacity (compression). It is conservatively assumed that the beam is not restrained from moving laterally. In many instances, the presence of a composite slab will provide restraint. In such cases, the use of K = 0.65 will be more appropriate.

$$K = 1.2$$

$$r = \frac{t_p}{\sqrt{12}} = \frac{0.375}{\sqrt{12}} = 0.108$$

$$\frac{Ka}{r} = \frac{1.2(6)}{0.108} = 66.7$$

$$\phi F_{cr} = 25.7$$

From AISC Manual Table 4-22.

$$\phi R_c = \phi F_{cr} L t_p = 25.7(15)(0.375) = \text{kips} > 39 \text{ kips, ok}$$

Bending Capacity

$$\lambda = \frac{L\sqrt{F_y}}{t_p\sqrt{47{,}500 + 112{,}000\left(\frac{L}{2a}\right)^2}}$$

$$= \frac{15\sqrt{36}}{0.375\sqrt{47{,}500 + 112{,}000\left[\frac{15}{2(6)}\right]^2}} = 0.509 \le 0.7$$

Flexual buckling does not control

$$F_{cr} = F_y Q = 36$$

$$\phi M_b = \frac{\phi F_y t_p L^2}{4} = \frac{0.9(36)(0.375)(15)^2}{4} = 683 \text{ kips-in} > 33 \times 6$$

$$= 198 \text{ kips-in} \quad \text{ok}$$

Shear Capacity

$$\phi R_v = \phi 0.6 F_y t_p L = 1.0(0.6)(36)(0.375)(15) = 122 \text{ kips-in}$$

Interaction Generally AISC does not require interaction between shear and normal stresses to be checked. However, including this interaction more accurately predicts test results, so it is included here. Additionally, the interaction equations from Chapter H of the Specification are used to combine the effects of the axial and bending forces.

Since $\frac{P}{\phi R_c} = \frac{39}{145} = 0.269 \geq 0.2$ Use (H1-1a AISC Manual)

$$\left(\frac{P}{\phi R_c} + \frac{8}{9}\frac{Va}{\phi M_b}\right)^2 + \left(\frac{V}{\phi R_v}\right)^2$$

$$= \left(\frac{39}{145} + \frac{8}{9}\frac{33(6)}{683}\right)^2 + \left(\frac{33}{122}\right)^2 = 0.329 \leq 1.0 \text{ ok}$$

Net Shear, Axial, and Bending Interaction on Plate. Axial capacity (tension).

$$\phi R_t = \phi F_u t_p [L - n(\phi_b + 1/16)]$$

$$= 0.75(58)(0.375)[15 - 4(0.875)] = 163 \text{ kips}$$

Bending Capacity. The net bending capacity is assumed to be the same as the gross bending capacity. This is based on testing reported by Mohr (2005).

$$\phi M_b = 683 \text{ kips-in}$$

Shear Capacity

$$\phi R_t = \phi 0.6 F_u t_p [L - n(\phi_b + 1/16)]$$

$$= 0.75(0.6)(58)(0.375)[15 - 4(0.875)] = 97.9 \text{ kips}$$

Interaction. Sinceuse $\frac{P}{\phi R_t} = \frac{39}{163} = 0.239 \geq 0.2$ use (H1-1a AISC Manual)

$$\left(\frac{P}{\phi R_c} + \frac{8}{9}\frac{Va}{\phi M_b}\right)^2 + \left(\frac{V}{\phi R_v}\right)^2$$

$$= \left(\frac{39}{163} + \frac{8}{9}\frac{33(6)}{683}\right)^2 + \left(\frac{33}{97.9}\right)^2 = 0.361 \leq 1.0 \text{ ok}$$

Weld Capacity

$$\theta = \tan^{-1}\left(\frac{T}{V}\right) = \tan^{-1}\left(\frac{39}{33}\right) = 49.8°$$

$$\phi R_w = 2(1.392) LD (1 + 0.5 \sin^{1.5}\theta)$$

$$= 2(1.392)(15)(4)[1 + 0.5 \sin^{1.5}(49.8)] = 223 \text{ kips} \geq 51.1 \text{ kips, ok}$$

Block Shear on Plate. L-shaped tearout.

Check Block Shear due to Shear Load

$$A_{nt} = t_p[L_e - 0.5(\phi_h + 1/16)] = 0.375[1.5 - 0.5(1.125 + 1/16)] = 0.340 \text{ in}^2$$

$$A_{gv} = t_p[L_e + (n-1)b] = 0.375[1.5 + (5-1)3] = 5.06 \text{ in}^2$$

$$A_{nv} = t_p[L_e + (n-1)b - (n-0.5)(\phi_h + 0.0625)]$$

$$= 0.375[1.5 + (5-1)3 - (5-0.5)(1)] = 3.38 \text{ in}^2$$

$$\phi R_{bsv} = \phi(0.6F_u A_{nv} + U_{bs} F_u A_{nt}) \leq \phi(0.6F_y A_{gv} + U_{bs} F_u A_{nt})$$

$$= 0.75[0.6(58)(3.38) + 1.0(58)(0.340)]$$

$$= 103 \leq 0.75[0.6(36)(5.06) + 1.0(58)(0.340)] = 96.8$$

$$= 96.8 \text{ kips} > 733 \text{ kips ok}$$

Check Block Shear due to Axial Load

$$A_{nt} = t_p[L_e + (n-1)b - (n-0.5)(\phi_h + 1/16)]$$

$$= 0.375[1.5 + (4)3 - (4.5)(1.0)]$$

$$= 3.38 \text{ in}^2$$

$$A_{gv} = t_P L_e$$

$$A_{nt} = 0.375 \times 1.5 = 0.563$$

$$A_{nt} = t_p[L_e - 0.5(\phi_h + 1/16)]$$

$$= 0.375[1.5 - 0.5(1.1875)] = 0.339 \text{ in}^2$$

$$\phi R_{bst} = \phi(0.6F_u A_{nv} + U_{bs} F_u A_{nt}) \leq \phi(0.6F_y A_{gv} + U_{bs} F_u A_{nt})$$

$$= 0.75[0.6(58)(0.339) + 1.0(58)(3.38)]$$

$$= 156 \leq 0.75[0.6(36)(0.563) + 1.0(58)(3.38)] = 156$$

$$= 156 \text{ kips} > 39 \text{ kips ok}$$

Check Combined Shear and Axial Block Shear

$$\left(\frac{V}{\phi R_{bsv}}\right)^2 + \left(\frac{T}{\phi R_{bst}}\right)^2 = \left(\frac{33}{96.8}\right)^2 + \left(\frac{39}{156}\right)^2 = 0.179 \leq 1.0 \text{ ok}$$

U-Shaped Tearout

$$A_{nt} = t_p[(n-1)(b-(\phi_h + 1/16))]$$

$$= 0.375[(5-1)(3-(1.0))] = 3.00 \text{ in}^2$$

$$A_{gv} = 2t_p L_e = 2(0.375)(1.5) = 1.13 \text{ in}^2$$

$$A_{nv} = 2t_p[L_e - 0.5(\phi_h + 1/16)]$$

$$= 2(0.375)[1.5 - 0.5(1.19)] = 0.679 \text{ in}^2$$

$$\phi R_{bst} = \phi(0.6F_u A_{nv} + U_{bs} F_u A_{nt}) \le \phi(0.6F_y A_{gv} + U_{bs} F_u A_{nt})$$

$$= 0.75[0.6(58)(0.679) + 1.0(58)(3.00)]$$

$$= 148 \le 0.75[0.6(36)(1.13) + 1.0(58)(3.00)] = 149$$

$$= 149 \text{ kips} \ge 39 \text{ kips, ok}$$

Block Shear on Beam Web due to Axial Load

$$A_{nt} = t_p[(n-1)(b-(\phi_h + 1/16))]$$

$$= 0.355[(5-1)(3-(1.0))] = 2.84 \text{ in}^2$$

$$A_{gv} = 2t_p L_e = 2(0.355)(1.5) = 1.07 \text{ in}^2$$

$$A_{nv} = 2t_p[L_e - 0.5(\phi_h + 1/16)]$$

$$= 2(0.355)[1.5 - 0.5(1.1875)] = 0.643 \text{ in}^2$$

$$\phi R_{bst} = \phi(0.6F_u A_{nv} + U_{bs} F_u A_{nt}) \le \phi(0.6F_y A_{gv} + U_{bs} F_u A_{nt})$$

$$= 0.75[0.6(65)(0.643) + 1.0(65)(2.84)]$$

$$= 157 \le 0.75[0.6(50)(1.07) + 1.0(65)(2.84)] = 163$$

$$= 157 \text{ kips} \ge 39 \text{ kips, ok}$$

There is no block shear limit state on the beam web. This completes the calculations for this example.

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*Actually the shear stress is not elastic because it is assumed uniform. The slender beam theory elastic shear stress would have a parabolic distribution with a peak stress of $9.24 \times 1.5 = 13.9$ ksi at the center of the section.

CHAPTER 3
WELDED JOINT DESIGN AND PRODUCTION

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3.1 INTRODUCTION

Welding is an established and essential tool of the steel construction industry. Before welding was possible, rivets were used to create structural members, and connect them. Today, welding is used to construct plate girders and box sections, and to connect structural members together reliably and cost-effectively. Along with the contributions of high-strength bolts, welding has rendered riveting obsolete.

Welding permits shapes, plates and even steel castings to be connected in nearly endless combinations. Steel components can be directly connected without the need for mechanical fasteners and the associated connection materials. Welded connections are aesthetically pleasing, directly satisfying “form ever follows function” criteria. Steels of various strength levels or thicknesses can be joined together, optimizing designs by strategically placing materials of higher capacity into regions of higher demand. The versatility of welding gives the designer greater freedom than any other method of joining.

Connections are critical to the performance of structural systems, and welded connections are no exception. Accordingly, when welding is improperly used, whether through incorrect design or detailing of the connection, or when a weld is made improperly during fabrication or erection, the connection may fail. Nearly everyone involved with the design, detailing, fabrication, erection, and inspection of welded structures needs to have some knowledge of welding. This chapter focuses on the production aspects of welding; other chapters deal with the design and detailing of welded connections, and inspection of welded connections. This chapter deals with the weld joint itself and what is required to deposit quality weld metal of required strength in that joint.

3.2 WELDING CODES AND STANDARDS

A variety of welding-related codes and specifications govern the design, fabrication, erection, and inspection of welded steel structures. AISC standards generally address the design requirements for the structure, while AWS standards typically focus on welding issues. Of necessity, there is some overlap between the coverage of AISC and AWS standards, and a few differences between these standards have been produced by the separate ANSI consensus committees. A general summary of commonly used welding-related standards is contained below.

3.2.1 AISC Specifications

AISC 360 Specification for Structural Steel Buildings. For steel building construction in the United States, the primary standard is the AISC Specification for Structural Steel Buildings, herein called the AISC Specification. References in this chapter are to the 2016 edition of that standard. The AISC Specification contains a variety of welding-related requirements, including but not limited to the following:

- Acceptable steel designations (A3.1)
- Acceptable filler metals (A3.5)
- Requirements for splices in heavy sections (J1.5)
- Beam copes and weld access holes (J1.6)
- Welds in combination with bolts (J1.8)
- Details of groove welds (J2.1)
- Details of fillet welds (J2.2)
- Available strength of welded joints (Table J2.5)
- Shop fabrication/welding issues (M2)
- Field erection/welding issues (M4)
- Weld quality control issues (N)
- Weld details for fatigue (Appendix 3)
- Welding issues associated with existing structures (Appendix 5)

J2 of the AISC Specification invokes all the provisions of AWS D1.1, except as noted in the AISC Specification (AISC, 2016).

AISC 341 Seismic Provisions for Structural Steel Buildings. The AISC Seismic Provisions were developed to augment the AISC Specification, adding provisions deemed necessary for high-seismic applications, which require capability to dissipate energy through controlled inelastic deformations in major seismic events. Members and connections in the seismic force resisting system (SFRS), including the welds that join various members, are subject to the special requirements contained in the AISC Seismic Provisions.

The AISC Seismic Provisions contain some welding-related requirements although most are covered by reference to AWS D1.8. AISC Seismic Provisions are subject to change, and all references contained within this chapter refer to the 2016 edition of this standard (AISC, 2016a).

3.2.2 AWS Specifications

AWS D1.1 Structural Welding Code—Steel. The AWS D1.1 Structural Welding Code—Steel, herein referred to as AWS D1.1, is a comprehensive welding code governing the design of connections, including connection details, welding procedures, acceptable base metals, filler metals and welding joint details, fabrication and erection requirements, welder qualification requirements, stud welding provisions, inspection requirements, and provisions for welding on existing structures. In addition to welding issues, D1.1 contains requirements for nonwelding metal working operations, such as thermal cutting, and heat curving, and stress relieving. AWS D1.1 is intended to govern projects involving steel of 3 mm (⅛ in) thick or thicker, with a minimum specified yield strength not greater than 690 MPa (100 ksi). The base metal types include carbon and low-alloy steels (AWS, 2015).

Major clauses (i.e., chapters) of AWS D1.1 include the following:

- Clause 1—General Requirements covers the scope of the code, limitations on its use, key definitions, and an outline of the responsibilities of the major parties involved with welding steel structures.
- Clause 2—Design of Welded Connections is divided into three parts. Part A deals with provisions common to all structures governed by the code. Part B addresses general requirements applicable to “nontubular connections,” that is, anything other than tubular connections, whether statically or dynamically loaded. Part C covers nontubular connections subject to cyclic loading.
- Clause 3—Prequalification is devoted solely to prequalified welding procedure specifications (WPSs). For a WPS to be prequalified, it must comply with all the provisions of this section of the code. Requirements include prequalified steels, filler metals, preheat levels, weld joint details, welding processes, and welding parameters.
- Clause 4—Qualification addresses the two subjects of WPS qualification, and welding personnel qualification. The types of tests necessary for qualification as well as limitations on the application of various qualification tests are fully detailed therein.
- Clause 5—Fabrication covers general fabrication practices and techniques required for all work performed in conformance with this code, whether the WPSs employed are prequalified or qualified by test. Some workmanship standards are included in this section.
- Clause 6—Inspection outlines the responsibilities of the various inspectors associated with steel construction. Inspection tasks are outlined, and some workmanship criteria are contained in this section. The techniques to be used with the various nondestructive testing methodologies are outlined, and acceptance criteria are supplied for different applications.
- Clause 7—Stud Welding details the requirements for the welding of shear studs, either by the stud welding process, or by use of other arc welding processes (e.g., SMAW, FCAW).
- Clause 8—Strengthening and Repairing Existing Structures briefly reviews the fundamental issues that must be addressed before modifications of existing structures are undertaken.
- Clause 9—Tubular Structures deals with construction with hollow structural sections (HSS) whether round, square, or rectangular. The chapter is divided into six parts, dealing with Design, Prequalified WPSs, WPS Qualification, Performance Qualification, Fabrication, and Inspection.

AWS D1.1 also contains a series of annexes, and a helpful commentary that assists the user in correctly applying the code. Some annexes are mandatory (i.e., part of the code) while others are not. Annex J contains terms and definitions used in the code.

AWS D1.1 is subject to change, and all references contained within this chapter are to the 2015 edition of this standard.

AWS D1.2 Structural Welding Code—Aluminum. D1.2 is the aluminum counterpart to AWS 1.1 (AWS, 2014). The topics covered in D1.2 are similar to those addressed in D1.1 with the exception that design stresses are not covered; instead, the Aluminum Design Manual as published by the Aluminum Association is referenced instead.

AWS D1.3 Structural Welding Code—Sheet Steel. AWS D1.3 covers welding of structural sheet and strip steels, including cold-formed members equal to or less than 5 mm (3/16 in) thick. Applications wherein sheet steel is joined to supporting structural steel, such as decking to beams, are also covered. When AWS D1.1 and AWS D1.3 are specified, the applicable provisions of each apply.

AWS D1.3 is subject to change, and all references contained within this Guide refer to the 2008 edition of this standard (AWS, 2008).

AWS D1.4 Structural Welding Code—Reinforcing Steel. AWS D1.4 covers welding of reinforcing steel (rebar) to itself, as well as reinforcing steel to plate or shapes. Appropriate applications for D1.4 include welding of embed plates as well as various forms of composite construction. D1.4 lists the various grades of weldable reinforcing steel, the required preheat levels, required filler metals as well as prescribing the details of welded connections involving the generally cylindrical reinforcing steel to flat surfaces, as well as cylindrical to cylindrical.

AWS D1.4 is subject to change, and all references contained within this Guide refer to the 2011 edition of this standard (AWS, 2011).

AASHTO/AWS D1.5 Bridge Welding Code. The AASHTO/AWS D1.5 Bridge Welding Code is a joint standard of the AWS and the American Association of State Highway and Transportation Officials (AASHTO). The code covers both redundant and nonredundant (fracture critical) steel highway bridges. While D1.5 and D1.1 have many similar provisions, there are several significant differences as well as a variety of subtle differences. D1.5 generally requires that WPSs be qualified by test, with a few exceptions such as certain SMAW procedures. Qualification testing involves Charpy V-Notch (CVN) specimens and all weld metal tensile specimens. NDT requirements are specified in D1.5.

AWS D1.5 is subject to change, and all references contained within this Guide refer to the 2010 edition of this standard (AWS, 2010).

AWS D1.6 Structural Welding Code—Stainless Steel. AWS D1.6 is analogous to D1.1, but covering the topic of stainless steel instead of carbon steel. Coverage includes requirements for welding various grades of stainless to stainless, as well as stainless to carbon steel.

AWS D1.6 is subject to change, and all references contained within this Guide refer to the

2007 edition of this standard (AWS, 2007).

AWS D1.7 Guide for Strengthening and Repair. For projects involving strengthening and repair of existing structures, AWS D1.1 requires the Engineer to “establish a comprehensive plan for the work” (AWS D1.1 clause 8.1). To assist the Engineer in this task, AWS D1.7 was developed. Unlike D1.1, D1.7 is a guide, not a code. Accordingly, D1.7 contains many suggestions and recommendations, but does not mandate anything. D1.7 includes guidance on these subjects: weldability, evaluation of existing welds, testing and sampling, heat straightening, strengthening and damage repair.

AWS D1.7 is subject to change, and all references contained within this Guide refer to the 2010 edition of this standard (AWS, 2010b).

AWS D1.8 Structural Welding Code—Seismic Supplement. AWS D1.8 contains the additional provisions intended to be applied to connections that are designed to resist yield level stresses or strains during design earthquakes. Just as the AISC Seismic Provisions augment the AISC Specification, so AWS D1.8 supplements AWS D1.1. When AWS D1.8 is specified, all the provisions of AWS D1.1 still apply, unless modified or superseded by AWS D1.8. In AWS D1.8, it is assumed that the structure has been designed in accordance with the AISC Seismic Provisions.

AWS D1.8 is subject to change, and all references contained within this Guide refer to the 2009 edition of this standard (AWS, 2009).

3.3 STRUCTURAL STEELS FOR WELDED CONSTRUCTION

The structural engineer normally specified the steel grade to be used, with a focus on the required mechanical properties, the minimum specified yield strength being of prime interest. For welded connections, attention must also be given to the weldability of the steel, that is, how easily the steel can be welded. AISC and AWS specifications list steels suitable for welded construction.

3.3.1 AWS D1.1 Steel Listings

Prequalified Steels. Table 3.1 of AWS D1.1 lists prequalified steel grades—materials that may be used with prequalified welding procedure specifications and without qualification testing. These are steel grades with a history of satisfactory service and with known, good weldability. It is generally recommended that prequalified steel grades be specified when welding is anticipated, although this may not always be possible or practical.

TABLE 3.1 Weld Metal Volumes for Different Thicknesses and Joint Types

Metric, with weld metal volumes listed in kg/m					
	Thickness (mm)				
	10	20	40	80	160
Single V $R = 6$ mm $\alpha = 45°$	0.95	2.51	7.65	25.92	94.26
Single bevel $R = 6$ mm $\alpha = 45°$	0.94	2.84	8.87	30.62	112.61
Single U $R = 0$ mm $\alpha = 20°$ $f = 3$ mm $r = 6$ mm	0.68	1.85	5.03	14.75	47.60
Single J $R = 0$ mm $\alpha = 45°$ $f = 3$ mm $r = 10$ mm	0.52	1.92	7.17	27.35	106.24
Double V $R = 0$ mm $\alpha = 60°$ $f = 3$ $S_1 = S_2$	0.10	0.43	1.82	7.40	29.79
Double bevel $R = 0$ mm $\alpha = 45°$ $f = 3$ $S_1 = S_2$	0.09	0.37	1.56	6.38	25.70
Double U $R = 0$ mm $\alpha = 20°$ $f = 3$ mm $r = 6$ mm	0.67	1.64	3.79	8.94	22.59
Double J $R = 0$ mm $\alpha = 45°$ $f = 3$ mm $r = 10$ mm	0.43	1.28	3.59	11.16	34.43

Imperial (English) with weld metal volumes listed in lb/ft					
	Thickness (in)				
	3/8	3/4	1-1/2	3	6
Single V R = ¼ in α = 45°	0.62	1.61	4.81	16.09	58.06
Single bevel R = ¼ in α = 45°	0.68	1.81	5.56	18.96	69.27
Single U R = 0 in α = 20° f = 1/8 in r = ¼ in	0.38	1.19	3.23	9.35	29.79
Single J R = 0 mm α = 45° f = ⅛ in r = ⅜ in	0.30	1.14	4.31	16.57	64.58
Double V R = 0 in α = 60° f = ⅛ in $S_1 = S_2$	0.06	0.26	1.09	4.48	18.12
Double bevel R = 0 in α = 45° f = ⅛ in $S_1 = S_2$	0.05	0.22	0.94	3.87	15.63
Double U R = 0 in α = 20° f = ⅛ in r = ¼ in	0.43	1.08	2.50	5.88	14.67
Double J R = 0 in α = 45° f = ⅛ in r = ⅜ in	0.25	0.77	2.17	6.46	20.96

All of the prequalified steels have a minimum specified yield strength of 620 MPa (90 ksi) or less. This is consistent with the AWS D1.1 philosophy that prequalified welding procedure specifications are limited to steels with a maximum yield strength of 620 MPa (90 ksi). All of the listed steel grades have both mechanical property controls and compositional limits that are appropriate for the welding processes and conditions specified within the code.

AWS D1.1 Approved Steels. Contained in Table 4.9 of AWS D1.1 is a list of "code-approved base metals" along with a listing of matching strength filler metals and preheat values. Two types of steels are listed in Table 4.9: those with a minimum specified tensile strength that exceeds 620 MPa (90 ksi), that is, they exceed the limit for use with prequalified WPSs, and newer steels that do not yet have a sufficient history of satisfactory usage for the D1 committee to comfortably place into Table 3.1 of AWS D1.1.

The WPSs used to join these code-approved steels will require qualification testing. Once the WPS is successfully qualified, the test may be used to support welding other steel combinations in accordance with Table 4.8. Avoidance of WPS qualification testing by specification of prequalified steel grades is always desirable, but for the higher strength steels where prequalified WPSs are unavailable, the use of Table 4.9 steels is desirable.

AWS D1.1 Unlisted Steels. Steel grades not listed in Table 3.1 or Table 4.9 of AWS D1.1 are known as unlisted steels. A steel grade may be unlisted for several reasons. The steel may have poor weldability and as a result it has been deliberately omitted. Alternatively, the steel grade may be new and may have good weldability, but simply has not yet been incorporated into the code. Some steel grades are excluded, not because of poor weldability, but because their mechanical properties are not sufficiently defined. This is the case for some of the AISI/SAE grades of steels, wherein only chemical compositions are specified. Finally, AWS D1.1 only recognizes steels classified to U.S. standards, such as ASTM and API (American Petroleum Institute) standards. Steels classified by other standards may have excellent properties, although they have not been incorporated into AWS D1.1, which is primarily a U.S.-based standard.

3.3.2 AISC Specification Treatment of Unidentified Steels

The AISC Specification (in Section A3.1b) permits the use of unidentified steels for "unimportant members or details where the precise physical properties and weldability of the steel would not affect the strength of the structure." In order to comply with both the AISC Specification and AWS D1.1, either the unidentified steel must comply with the AWS requirements for unlisted materials, or the WPS must be qualified by test. The latitude offered by AWS D1.1 clause 3.4 is important when unidentified steels are encountered.

3.3.3 Welding Requirements for Specific Steels

Weathering Steels. Weathering steels are able to resist atmospheric corrosion, precluding the need for paint or coating systems. Included in this category of steels are ASTM A588, A852, A709 Grade 50W, HPS 50W, HPS 70W, and HPS 100W, A606, A847, A514, as well as the first weathering steel A242 (which is nearly obsolete). Each of these steels has specific fabrication requirements, but the general provisions applicable to this group of weathering steels will be reviewed. Weathering steels all contain sufficient alloy content to offer resistance to atmospheric corrosion. Popular for bridge construction, weathering steels have also been used for buildings, amphitheaters, light poles, transmission towers, and other structures.

A special requirement associated with welding weathering steels in general involves the selection of the filler metals, with specific focus on ensuring the weld has atmospheric corrosion resistance equal to that of the base metal. Several approaches may be taken. First, all welds on weathering steel structures may be made with alloy filler metals that deposit weld metal with a sufficient alloy content so that the deposit has a weathering composition. While a variety of alloys may be used, a common choice is to use nickel-bearing filler metals, typically with a nominal nickel content of 1 percent or greater. Prequalified filler metals for prequalified weathering steels are listed in Table 3.4 of AWS D1.1.

A second approach involves the use of carbon steel filler metals for single pass welds of a restricted size. During welding, some of the weathering steel base metal melts and becomes part of the weld deposit. Smaller single pass fillet welds, for example, experience sufficient admixture (mixing of base metal and filler metal) to give the resultant weld enough alloy to have weathering characteristics. The level of admixture depends on the welding process in addition to the weld size. AWS D1.1 prescribes the conditions, by maximum weld size and by process, under which this approach may be used (AWS D1.1, clause 3.7.3). It may allow the contractor to employ filler metals that are used for standard carbon steel applications. The carbon steel materials are less expensive to purchase, and more importantly, it is not necessary to reconfigure the welding equipment with different filler metals as jobs of different steels flow through a shop.

Other than attention to the chemistry of the weld metal, weathering steels such as A588 have good weldability and the welding is very similar to nonweathering steels of similar strengths. Quenched and tempered weathering steels such as A852 and A709 HPS70W may require additional welding controls due to their higher strength and Q&T processing.

Quenched and Tempered Steels. A variety of steels are processed at the producing mill by quenching and tempering (Q&T). The quenching operation hardens the steel, while the tempering operation increases its toughness and ductility. One of the first popular Q&T steels for structural applications was ASTM A514, which is a martensitic steel with 100 ksi minimum specified yield strength. ASTM A514 can be, and is, successfully welded every day, but it can be problematic when the proper procedures are not followed.

Because Q&T steels gain their strength by controlled quenching and tempering, the welding process must be controlled to minimize softening or hardening of the heat-affected zone (HAZ), as well as to maintain adequate toughness. The degree of control necessary depends on the specific steel involved.

The controls required include minimum and maximum levels of preheat and interpass temperature, as well as heat input. The goal is to control the cooling rate experienced by the HAZ, and yet provide sufficient preheat to avoid cracking in the weld and HAZ. Accordingly, tables have been developed that give the maximum allowable heat input for different levels of preheat and interpass temperatures. AWS D1.5 provides a table with preheat ranges, and acceptable heat input limits within those ranges, that are in turn a function of the thickness of the steel being joined. Welding within a more restrictive envelope of acceptable parameters is different than typical practice for most carbon steel applications, and thus can present additional challenges.

Quenched and Self-Tempered Steels. Another method of processing steel is quenching and self-tempering (QST), which is a variation of the thermo-mechanical control process (TMCP) rolling technique. Steel shapes made by this process are quenched in a traditional manner, but the quenching does not cool the entire cross-section of the shape. The residual thermal energy in the core of the shape then tempers the quenched outer surfaces without the application of additional thermal energy, hence the term self-tempering.

ASTM A913 is a QST steel, available in four grades: 50, 60, 65, and 70 (corresponding to metric yield strengths of 345, 410, 450, and 485 MPa). The material specification limits the carbon content to lower levels, and requires the steel be under certain carbon equivalent levels. As a result the steel has good weldability and can be welded with reduced preheat levels. AWS D1.1 permits A913 Grades 50, 60, and 65 to be welded with a 0°C (32°F) preheat, providing the filer metal complies with a maximum diffusible hydrogen level of 8 mL/100 g.

Preheat might be required to compensate for restraint and other factors, but the permitted low level of preheat speaks to the good weldability of the material. A913 Grade 70 steel is prequalified by AWS D1.1 with preheat levels that fall in line with other 485 MPa (70 ksi) materials.

Multigrade Steels. Some steels are marketed as “dual-grade,” or “triple-grade,” or other multigrade variations. Some people view this as somewhat of a cheat on the part of the supplier, but such steels are more restrictive and better defined than steels bearing but one grade, since multigraded steels must meet all the requirement of all the listed specifications. Multigraded steels are possible because of the overlap between specification requirements. It is common for a single heat of steel to meet all the requirements of A36, A572 Gr 50, and A992.

Welding techniques and procedures for welding on multigraded steels should be such that all the requirements of each individual grade are met. This is not difficult to do as the requirements for similar steels are also similar. Where there are differences, it is prudent to apply the more restrictive requirements to the multigraded material.

3.4 WELDING AND THERMAL CUTTING PROCESSES

There are approximately 100 different welding and thermal cutting processes. Currently, in the fabrication and erection of steel buildings, four welding processes dominate (FCAW, GMAW, SAW, and SMAW), as do three thermal cutting processes (OFC, PAC, and AAC). These processes, plus a few others that are occasionally used for specialized applications, will be covered in this section.

The choice of welding process is usually left up to the contractor, as the contractor is typically best positioned to select the optimal process for a given application. In unique situations, the engineer may specify a special process, or processes for specific applications in the contract documents, but this practice is uncommon. The selection of the welding process is typically considered part of the “means and methods” of construction, and the choice of process may significantly affect the cost of a project.

When properly used, all of the welding processes listed in AWS D1.1 are capable of producing welds with the requisite quality for building construction. Of course, any welding process can be abused, and all can produce welds of poor quality if improper procedures are used, or if the welder’s skills are inadequate.

Although the selection and control of the welding process is typically the responsibility of the contractor, it is important that all parties involved understand these processes in order to ensure high quality and economical fabrication. Particularly when problems arise on a project, the engineer may be required to become involved with welding process issues, and a basic knowledge of how the process operates will aid in resolving construction problems.

3.4.1 Shielded Metal Arc Welding

AWS A3.0 defines shielded metal arc welding (SMAW) as “an arc welding process with an arc between a covered electrode and the weld pool. The process is used with shielding from the decomposition of the electrode covering, without the application of pressure, and with filler metal coming from the electrode." (AWS, 2010a). Often called "stick welding," SMAW is a common fusion welding process. SMAW is used in the shop and in the field for erection and repairs (Figs. 3.1 and 3.2).

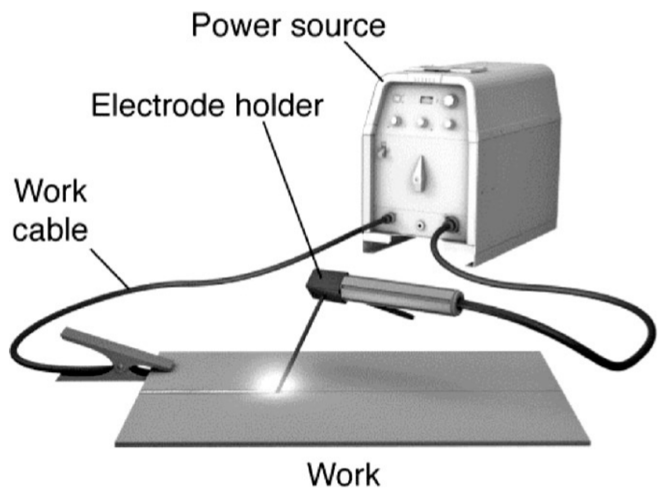

FIGURE 3.1 SMAW process.

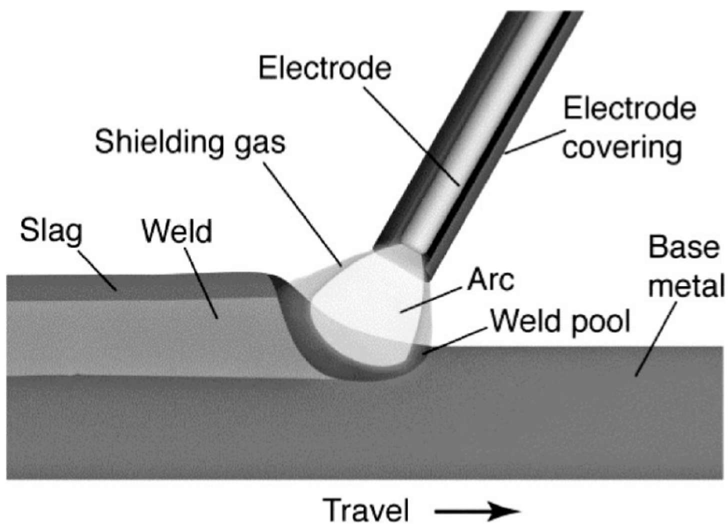

FIGURE 3.2 SMAW process details.

SMAW is sometimes called "manual" welding. With SMAW, the welder manually moves the arc along the length of the joint, as well as manually feeds the electrode toward the weld pool. Considerable skill is required to accomplish these two tasks. The welder must maintain a specific arc length, the distance from the end of the electrode to the weld pool. If the arc length is too long, the arc will go out; if too short, the electrode will fuse to the weld pool.

Flexibility is perhaps the greatest advantage of SMAW. With one power supply, a wide range of materials can be welded by simply changing the electrode to be used, and adjusting the power source output setting. The same machine can weld thin or thick material, as well as steel and stainless steel. Welding can be performed in all positions.

The greatest shortcoming of SMAW is its relative inefficiency. The electrodes are used in lengths of 200 to 400 mm (9 to 18 in), and approximately 50 mm (2 in) of each electrode is unusable and must be discarded. Once the electrode is consumed, the welder must stop welding, remove the stub, insert another electrode, and start welding again. These work interruptions are inherent to the process, and reduce production rates accordingly. SMAW also requires a fair amount of operator skill to achieve acceptable welds.

While SMAW was extensively used in production environments in the 1940s and 1950s, it is less commonly used today in developed countries where wages are high. On the other hand, in less developed countries where labor costs are low and capital for equipment may be scarce, SMAW continues to be popular. In 2015, in the structural field, SMAW is used for tack welding, smaller erection projects and for making weld repairs.

3.4.2 Flux Cored Arc Welding

AWS A3.0 defines flux cored arc welding (FCAW) as "an arc welding process using an arc between a continuous filler metal electrode and the weld pool. The process is used with shielding from a flux contained within the tubular electrode, with or without additional shielding from an externally supplied gas, and without the application of pressure." FCAW electrodes are supplied on spools, coils, reels, and other devices that may contain as little as 0.5 kg (1 lb) or up to 500 kg (1000 lb) or more. While not literally "continuous," the long lengths of electrode permit welding to be performed without the interruptions inherent to SMAW (Figs. 3.3 and 3.4).

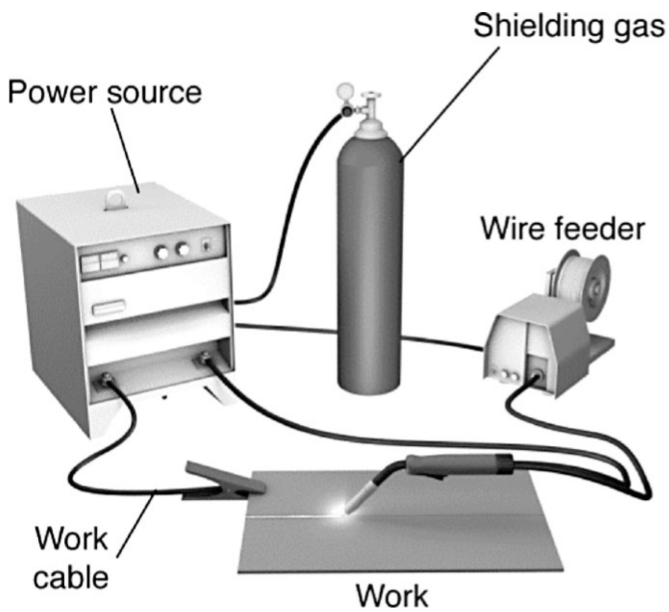

FIGURE 3.3 FCAW process.

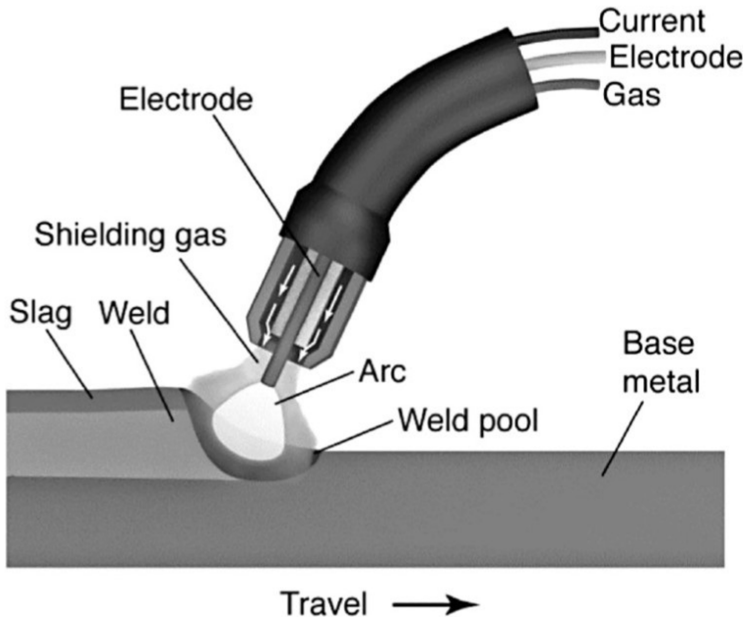

FIGURE 3.4 FCAW process details.

In FCAW, the electrode is fed through a welding gun by a wire feeder, a mechanical device that delivers the electrode at a regulated rate. This eliminates one of the skills required by the SMAW welder; the welder no longer needs to manually deliver the electrode to the joint. Accordingly, FCAW is known as “semiautomatic” welding since the welder simply propels the electrode along the length of the joint. Due to the nature of the power supplies used for FCAW, the arc length is automatically regulated, eliminating the need for the welder to maintain this critical distance. FCAW requires less skill than SMAW. FCAW can also be performed in an automatic mode where the welding torch is mounted on a mechanical device that moves the electrode along the length of the joint.

Within the welding gun is a hollow copper tube called a contact tube or contact tip. Electrical energy is transferred from the gun to the electrode at this point. A short length of electrode, typically around 25 mm (1 in) extends beyond the end of the contact tip during welding. This short length of electrode is capable of conducting considerable current, much more than is possible with SMAW electrodes. Thus, the combination of the “continuous” electrode with the higher welding currents makes FCAW more productive than SMAW.

Two versions of flux cored arc welding exist: gas shielded FCAW (or FCAW-G) and self-shielded FCAW (or FCAW-S). Shielding gas is required for FCAW-G whereas FCAW-S requires none. A variety of shielding gasses may be used with FCAW-G, with carbon dioxide or argon/carbon dioxide mixtures being common for steel applications. In general, the gas shielded version is more flexible with a greater range of available filler metals. The self-shielded version is ideal for welding outdoors where wind may disturb the gas shield associated with FCAW-G.

When a weld is made with FCAW, a slag covering remains behind on the surface of the completed weld. The slag must be removed upon completion of the weld, an activity that adds to the cost of making the weld.

FCAW began to replace SMAW in many high production situations in the 1960s and 1970s and is common in a variety of industries today. The flux contained in the electrode allows the process to handle moderate levels of scale and rust as well as other contaminants on the surface of the material being joined. With the proper electrode and procedure, all position welding is possible. In general, FCAW may be viewed as a replacement for SMAW, albeit at the cost of more expensive equipment and less versatility. FCAW-G is one of primary processes used for shop fabrication, and FCAW-S is the dominant process used for field erection.

3.4.3 Gas Metal Arc Welding

AWS A3.0 defines gas metal arc welding (GMAW) as "an arc welding process, using an arc between a continuous filler metal electrode and the weld pool. The process is used with shielding from an externally supplied gas and without the application of pressure." Often called "MIG" welding (metal inert gas), the process is similar in concept to FCAW except that the electrode contains no flux, and the finished weld does not have an extensive slag coating. GMAW (shown in Figs. 3.5 and 3.6) can be used semiautomatically or automatically, and is commonly used for robotic applications.

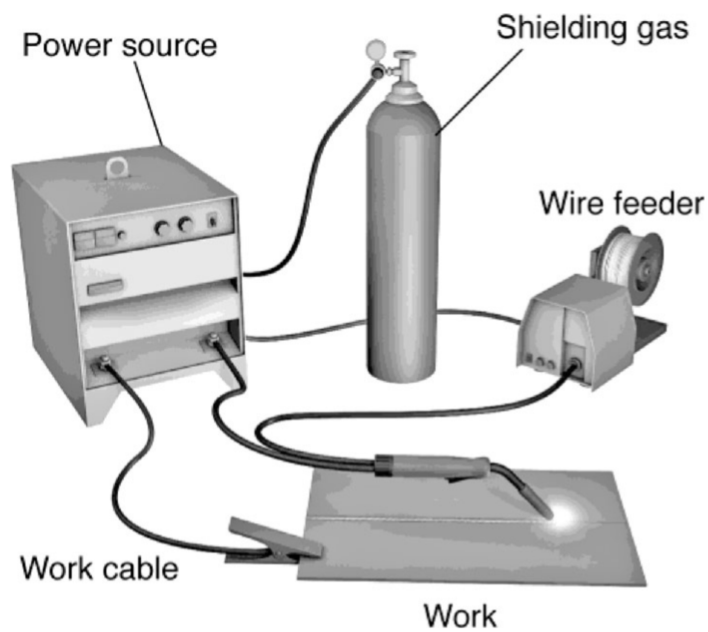

FIGURE 3.5 GMAW process.

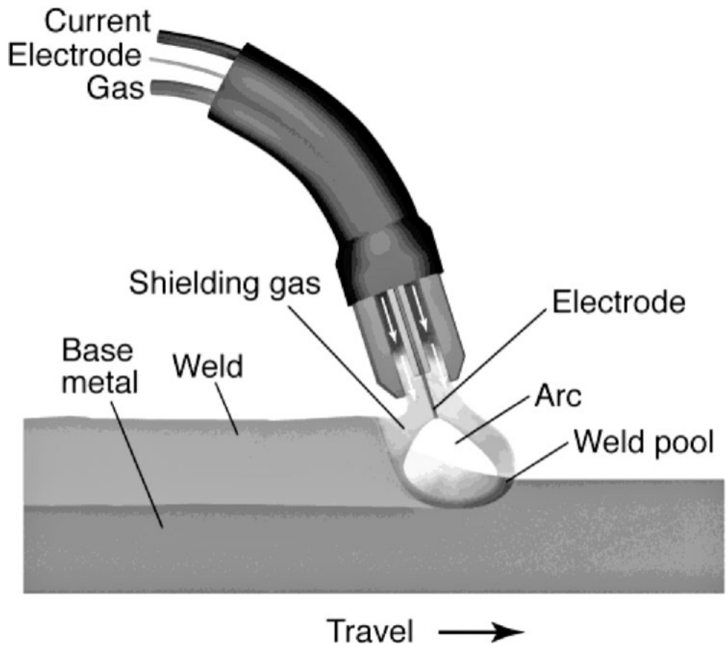

FIGURE 3.6 GMAW process details.

In its most basic form, GMAW can use the same type of equipment as FCAW. GMAW may use either solid or metal-cored electrodes. Solid electrodes are essentially wires of a specified composition and diameter. Metal-cored electrodes, also called “composite” electrodes, are tubular electrodes that contain metal powders in the core. While they are similar in construction to FCAW electrodes, the essential difference is that GMAW metal-cored electrodes do not contain flux and slag forming ingredients. A wide range of shielding gasses may be used with GMAW, depending on the material being welded and the mode of metal transfer (discussed later). Carbon dioxide may be used for welding on steel with GMAW. Since CO_2 is not inert in the presence of the arc (it breaks down into the constituent components and becomes “active”), GMAW that uses this gas may be called “metal active gas” (MAG) welding. Purely inert gases such as argon and helium can be used, typically in conjunction with minor components of other gases, such as oxygen. Blends of two gasses are typical, and mixtures of three or more gases are sometimes used.

Since GMAW has no flux on or inside the electrode, the finished weld contains little or no slag, eliminating the postwelding operation of slag removal associated with processes like SMAW and FCAW. This makes GMAW ideal for automatic and robotic welding operations, as well as for multiple pass welds. The absence of flux in the electrode, however, explains a limitation of GMAW: the process is less tolerant of surface contaminants such as scale and rust. For out of position welding, the slag associated with SMAW and FCAW can be used to support the liquid metal; without this slag, welding out of position with GMAW is more difficult. Depending on the filler wire used small silicon “islands” may be present on the weld face or toes. Although these silicon spots are not deleterious to the weld in any way, they may cause issues if the part is to be painted as the islands may flake off.

The nature of the arc and the transfer of metal from the electrode to the puddle can take on a variety of forms with GMAW, depending on the welding parameters, and shielding gas. Distinctions in the type of transfer are typically identified as modes of transfer. For structural steel welding, four modes of transfer are common. One transfer mode, short-circuit transfer, deserves special focus since it is often problematic when applied to steels of the typical thicknesses associated with structural steel construction.

GMAW Spray Transfer. Spray transfer, as applied to GMAW, is defined in AWS A3.0 as "metal transfer in which molten metal from a consumable electrode is propelled axially across the arc in small droplets." As shown in Fig. 3.7, the droplets are smaller in diameter than the electrode diameter, which itself is small. Spray transfer is associated with relatively high current and voltage levels. High-quality welds with particularly good appearance are obtained. The shielding gas used in spray arc transfer for steel applications is composed of at least 80 percent argon. Typical mixtures would include 90/10 argon/CO_2, and 95/5 argon/O_2. Due to the intensity of the arc and gravitational pull on the weld puddle, spray transfer for steel applications is restricted to welding in the flat and horizontal positions. Further, spray transfer is restricted to welding on relatively thick materials.

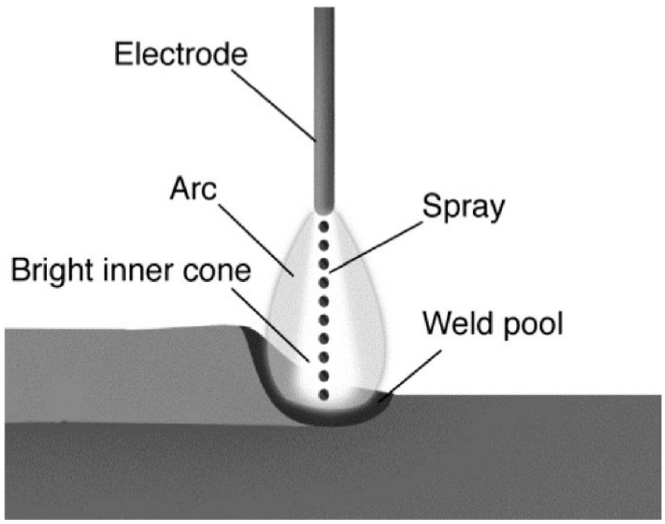

FIGURE 3.7 GMAW spray transfer process.

GMAW Globular Transfer. Globular transfer is defined in AWS 3.0 as the "transfer of molten metal in large drops from a consumable electrode across the arc," as illustrated in Fig. 3.8. The droplets are typically larger in diameter than the electrode. The large droplets hitting the weld puddle can create considerable spatter, droplets from which may fuse to the surface of the base metal, away from the weld. The arc is relatively rough and the weld appearance is inferior to GMAW spray transfer. Two compelling reasons, however, encourage the use of globular transfer in steel applications: first, lower cost CO_2 shielding gas can be used and secondly, the heating effects on the welding gun and the welding operator are less with globular transfer versus spray transfer.

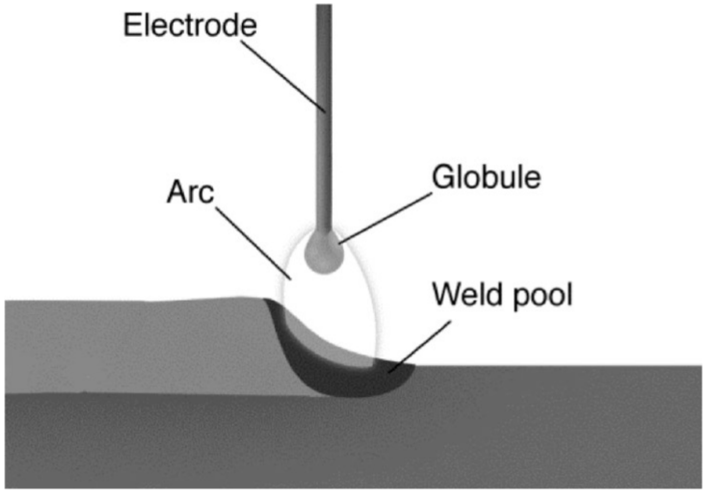

FIGURE 3.8 GMAW globular transfer process.

Short-Circuit GMAW. Short-circuit GMAW (GMAW-S) is defined in AWS A3.0 as a "gas metal arc welding process variation in which the consumable electrode is deposited during repeated short circuits." An electrical "short circuit" is achieved when two separate electrical conductors physically touch each other and current is conducted across the interface. For short-circuit GMAW, the short occurs between the electrode and the workpiece: there is no arc at this point. High electrical currents flow through the short circuit, causing the electrode to overheat and essentially "explode," eliminating the short. Momentarily, the arc is reestablished and molten metal is transferred to the weld pool. As shown in Fig. 3.9, the electrode is fed into the weld pool again, creating another short and repeating the sequence, which occurs hundreds of times per second.

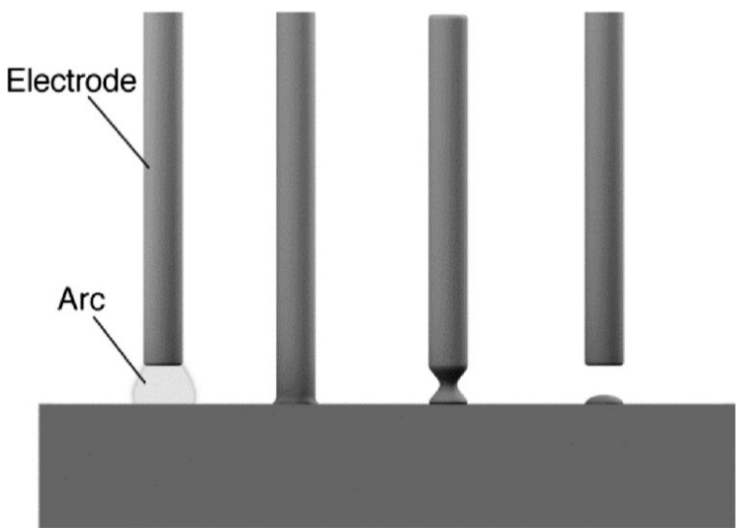

FIGURE 3.9 GMAW short-circuit transfer process.

While the process mode is officially called short-circuit gas metal arc welding, it is known by a variety of names, including "short arc" or "short-circuit transfer." GMAW-S is ideally suited for welding on thin sections of material 3 mm (⅛ in) or less (although it can be used on thicker materials) because it is a relatively "cold" welding process. This low energy mode of transfer allows the process to be used on steel when welding in the vertical and overhead position.

A major disadvantage of GMAW-S is directly related to the “cold” nature of the transfer mode: GMAW-S is notorious for a welding defect known as incomplete fusion, more commonly called “cold lap.” The energy associated with a GMAW-S welding procedure may be sufficient to melt the filler metal, creating a weld bead, but insufficient to cause the weld metal to fuse to the base metal. The weld may have good visual appearance, but the connection may have limited or no overall strength, making this possibility a major concern. D1.1 does not outright prohibit GMAW-S, but requires special qualification testing for the welding procedures, and special welder qualification tests for those that will use the process.

Pulsed Spray Transfer GMAW. In gas metal arc welding, pulsed spray transfer is defined in AWS A3.0 as a “variation of spray transfer in which the welding power is cycled from a low level to a high level, at which point spray transfer is attained, resulting in a lower average voltage and current.” The welding power supply automatically pulses between a higher energy setting where metal is transferred from the electrode to the weld pool, to a lower energy setting where the arc is maintained, but no metal is transferred. The cycle repeats itself hundreds of times per second. The high energy setting is similar to that used with spray transfer, thus the term “pulsed spray.” The process may be called “pulsed arc” or “pulse welding.”

The pulsing of the energy from high to low values reduces energy input into the weld joint. This enables pulsed spray transfer GMAW (GMAW-P), shown in Fig. 3.10, to be used out of position (unlike GMAW spray transfer) when welding on steel. Compared with GMAW-S, which can be used out of position, GMAW-P has significantly less spatter and is much more resistant to incomplete fusion.

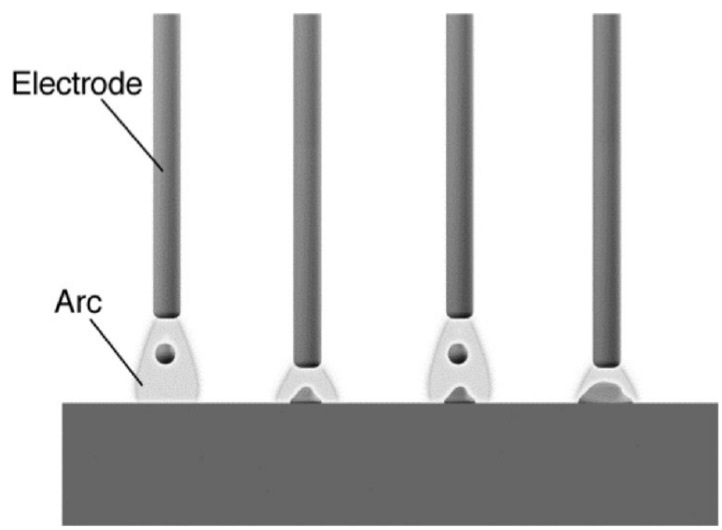

FIGURE 3.10 GMAW pulsed transfer process.

Because of the reduction in the overall energy required to make a given sized weld, GMAW-P can be used on thinner materials and generates less welding fume than GMAW-S. With less energy delivered to the joint, reductions in welding distortion are possible.

GMAW-P requires the use of more complex and expensive welding equipment, and the welding procedures are slightly more complicated. However, developments in welding power supplies have overcome many of these drawbacks, making GMAW-P a common mode of transfer.

3.4.4 Submerged Arc Welding

AWS A3.0 defines submerged arc welding (SAW) as "an arc welding process using an arc or arcs between a bare metal electrode and the weld pool. The arc and molten metal are shielded by a blanket of granular flux on the workpieces. The process is used without pressure and with filler metal from the electrodes and sometimes from a supplemental source (welding rod, flux, or metal granules)." Often called "subarc," SAW is a fusion welding process with the arc buried under a blanket of granular flux, as shown in Figs. 3.11 and 3.12. The flux for SAW performs the same function as flux for SMAW or FCAW welding: cleansing the weld from surface contaminants and then forming a slag to protect the molten weld pool.

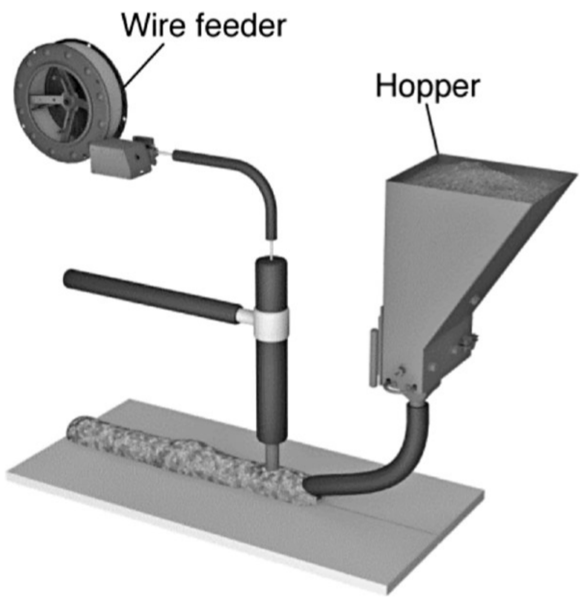

FIGURE 3.11 SAW process.

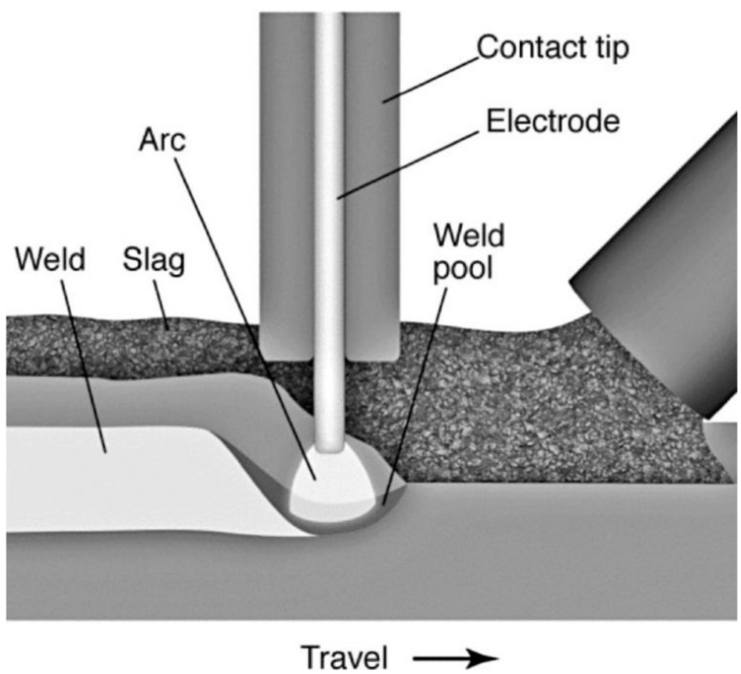

FIGURE 3.12 SAW process details.

Since the SAW arc is completely covered by the flux, welds are made without the flash, spatter, sparks, or smoke that characterizes the open-arc processes. The flux also covers the weld pool, making it impossible for the welder to observe its size and shape. As a result, SAW is typically used in the automatic mode where the travel speed of the electrode is regulated by a mechanical device. Semiautomatic operation is also possible, but the welder must rely on the characteristics of the slag covering to regulate the travel speed.

SAW is capable of high productivity because it can use high welding currents, resulting in higher deposition rates and deeper penetration. For even higher deposition rates, a second or third electrode (or even more) can be added into the system to increase productivity further. Welds made under the protective layer of flux are excellent in appearance and spatter free. Another benefit of the SAW process is freedom from the open arc. This means that the welder is not required to use the standard protective helmet, and multiple welding operations can be conducted in small areas without the need for extensive shields to guard the operators from arc flash. The process produces very little smoke, which is another production advantage, particularly in situations with reduced ventilation.

SAW is restricted to welding in the flat and horizontal position. For shop fabrication, the use of positioners, or simple reorientation of the weldment, can facilitate in-position welding. However, field conditions prohibit such opportunities, and thus restrict the suitability of SAW. The process is popular for making the longitudinal welds on plate girders and box columns, as well as for web and flange splices on components for these members.

3.4.5 Gas Tungsten Arc Welding

AWS A3.0 defines gas tungsten arc welding (GTAW) as “an arc welding process using an arc between a tungsten electrode (nonconsumable) and the weld pool. The process is used with shielding gas and without the application of pressure.” In this fusion welding process, often called “TIG” welding, the tungsten electrode conducts the welding current from the torch to the workpiece, as shown in Fig. 3.13. The tungsten electrode is not intended to deliver molten metal to the weld pool (although this can happen, leading to weld imperfections). Filler metal, if needed for the application, is supplied from a rod that is not electrically charged (Fig. 3.14). The filler metal is dipped into the weld pool and melted by the heat of the pool. Argon is the typical shielding gas, although helium or argon-helium mixtures may be used as well.

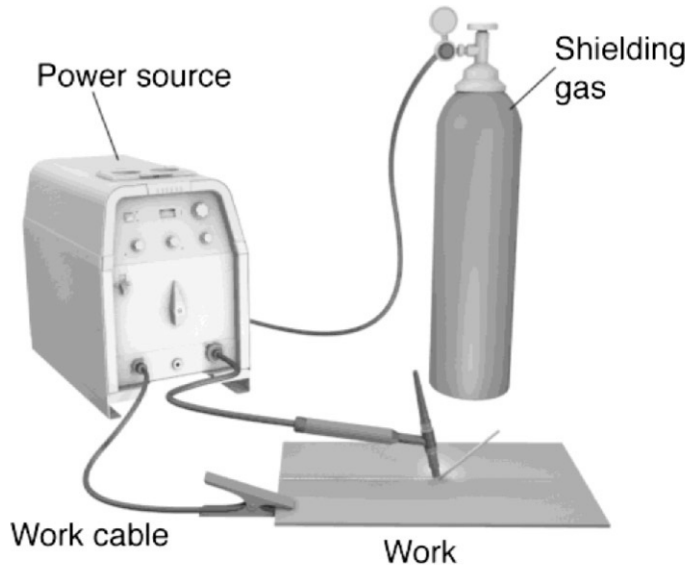

FIGURE 3.13 GTAW process.

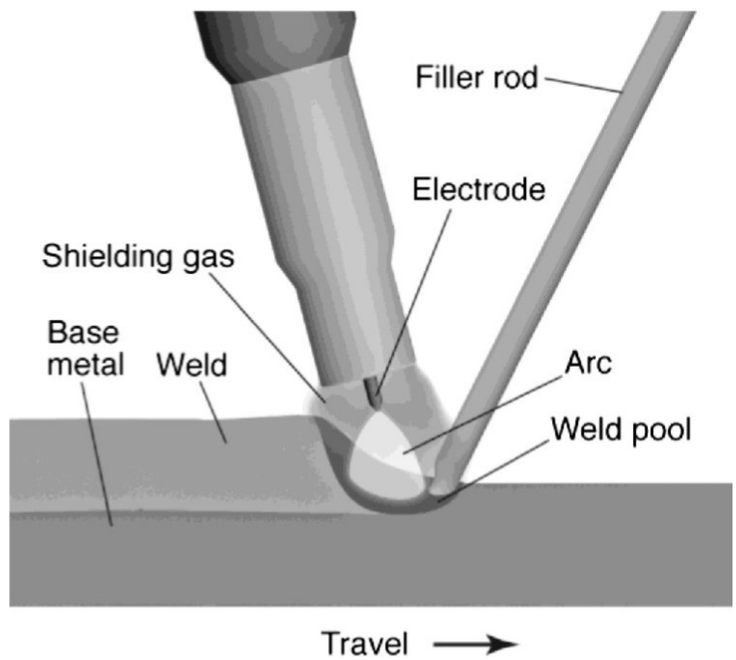

FIGURE 3.14 GTAW process details.

GTAW is ideally suited to weld nonferrous materials such as stainless steel and aluminum, and is very effective for joining thin sections. Highly skilled welders are required for GTAW, but the resulting weld quality can be excellent. The process is often used to weld exotic materials, such as titanium. Critical repair welds, as well as root passes in pressure piping, are typical applications. The completed GTAW weld, when properly made, has excellent appearance, and contains no slag that requires removal. Since GTAW has limited capacity to handle surface contaminants, the base metal must be relatively clean.

GTAW is inherently slow, and thus, often used only in situations where no other process is viable. It is typically used in the manual mode, although automated and robotic applications are possible. The process is not commonly used for structural applications.

3.4.6 Arc Stud Welding

AWS A3.0 defines arc stud welding (SW) as “an arc welding process using an arc between a metal stud, or similar part, and the other workpiece. The process is used without filler metal, with or without shielding gas or flux, with or without partial shielding from a ceramic or graphite ferrule surrounding the stud, and with the application of pressure after the fraying surfaces are sufficiently heated.” Most of the molten metal and any contamination are expelled from the weld area as the stud is mechanically forced into the weld pool (Fig. 3.15).

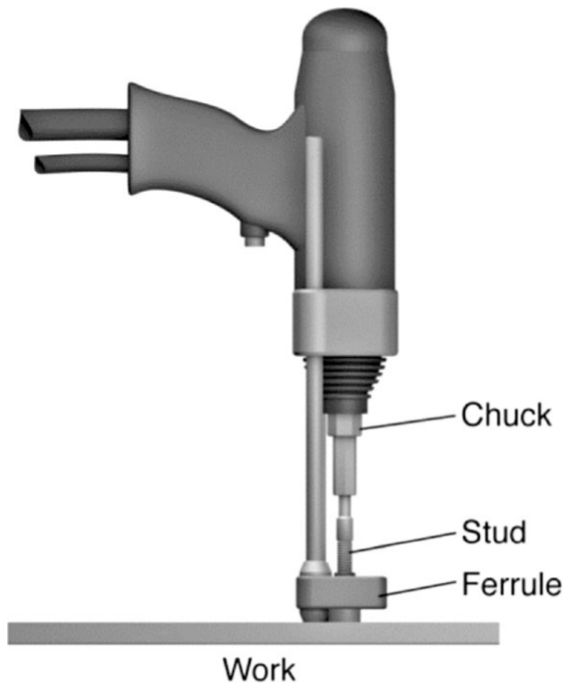

FIGURE 3.15 SW process.

SW is used to attach headed shear stud connectors to beams to facilitate composite action when the studs are embedded in concrete. The process is automated and fairly simple to use. The keys to obtaining a quality weld are to weld on relatively clean materials, use clean studs, and obtain the proper balance between welding current and arcing time.

3.4.7 Electroslag Welding

AWS A3.0 defines electroslag welding (ESW) as “a welding process producing coalescence of metals with molten slag, melting the filler metal and the surfaces of the workpieces. The weld pool is shielded by this slag, which moves along the full cross section of the joint as welding progresses.” Technically, it is not arc welding, but resistance welding. ESW uses a solid or tubular electrode that is fed through an electrically conductive, hot slag that melts the electrode, adding metal to the weld pool. ESW is used for vertical-up welding, with the weld pool contained by copper dams (or shoes) on the sides of the weld; metallic backing that fuses to the weld can also be used (Fig. 3.16). Groove welds in butt and T joints are the most common applications. The welds are completed in a single pass.

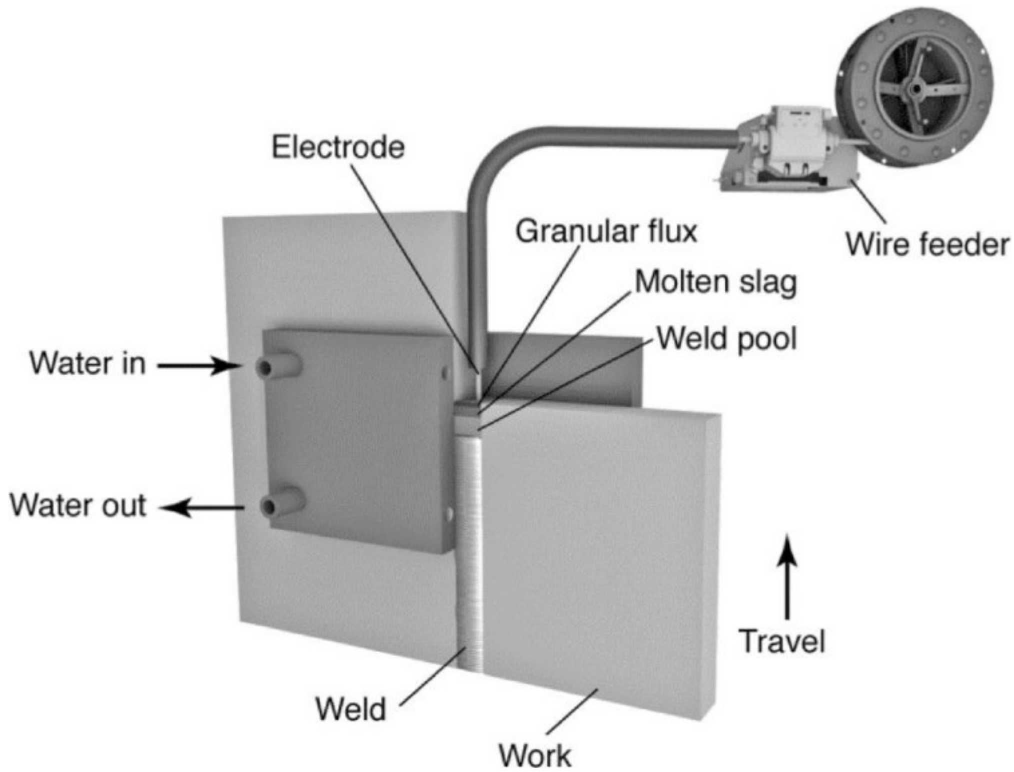

FIGURE 3.16 ESW process.

When electroslag welding is started, it functions like SAW; an arc buried under the flux melts the base metal, filler metal and flux, forming a slag. However, unlike SAW, the slag for ESW is electrically conductive. After a slag blanket is established, the electrical current is conducted from the electrode, through the slag, and into the workpiece. The high currents transferred through the slag keep it hot. As the electrode is fed through this hot slag, it melts, and molten metal drips from the electrode into the weld pool. No arc is involved, except when the process is started.

Very high deposition rates can be obtained with ESW, leading to productivity gains. Normally, the joint details involve square edge preparations, eliminating plate beveling costs. In some cases, material handling is reduced—plates do not need to be flipped as is the case for double-sided welds made with SAW, for example. Angular distortion can be reduced, as compared to single-sided welds in V and bevel grooves. ESW is ideal for thicker materials, and typical applications are 25 mm (1 in) thick or greater. Due to the high levels of heat input, often 10 times or more than typical of SAW, the heat-affected zone (HAZ) is large.

For structural applications, ESW is used for flange splices, joined before plate girders are made, and for the "blind weld" that is often required when stiffeners are installed in box columns.

3.4.8 Oxyfuel Cutting

Oxyfuel Gas Cutting. AWS A3.0 defines oxyfuel gas cutting (OFC) as "a group of oxygen cutting processes using heat from an oxyfuel gas flame." The process relies on the chemical reaction of oxygen with the metal at elevated temperatures. OFC is used to cut steels and to prepare bevel and V grooves. In this process, the metal is heated to its ignition temperature, or kindling point, using a series of preheat flames. After this temperature is attained, a high-velocity stream of pure oxygen is introduced, which causes oxidation to occur. The force of the oxygen stream blows the oxides out of the joint, resulting in a clean cut (Fig. 3.17). The oxidation process also generates additional thermal energy, which is radially conducted into the surrounding steel, increasing the temperature of the steel ahead of the cut. The next portion of the steel is raised to the kindling temperature, and the cut proceeds.

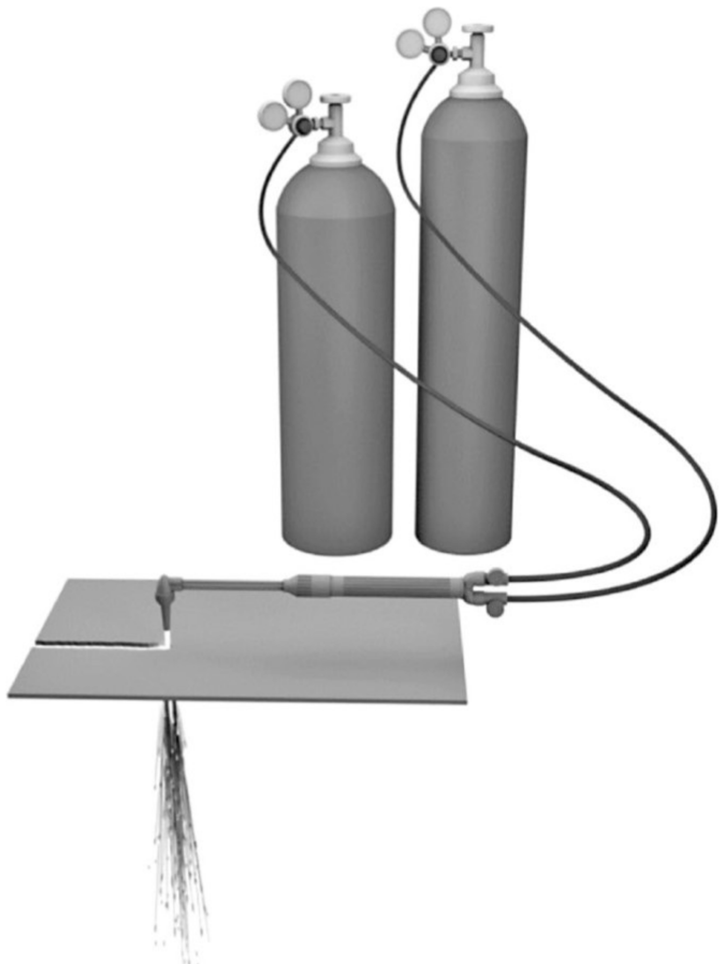

FIGURE 3.17 OFC process.

The process may be called burning (which is an apt term, since the operation depends on oxidation), or flame cutting. A variety of fuel gasses can be used, including acetylene, natural gas, propane, and others.

Because oxyfuel cutting is a combustion process that generates thermal energy, the process is well suited for cutting even very thick sections of steel, up to 300 mm (12 in) and greater. Oxidization resistant materials such as stainless steel and aluminum cannot be cut with conventional oxyfuel cutting methods.

Oxyfuel cutting of steel is done more rapidly than with mechanical cutting systems. The cut can be curved, beveled and configured in ways that are difficult to achieve by mechanical means. Simple hand cutting systems are very economical, and the equipment is highly portable. Dimensional control is more difficult with oxyfuel cutting than with mechanical cutting systems. The process inherently involves showers of sparks and molten metal, creating potential fire hazards. When applied to hardenable steels, the edge may become extremely hard, brittle and crack-sensitive. The localized heating and cooling can cause the cut parts to distort.

3.4.9 Plasma Arc Cutting

Plasma arc cutting (PAC) is "an arc cutting process employing a constricted arc and removing molten metal with a high-velocity jet of ionized gas issuing from the constricting orifice" according to AWS A3.0 (Fig. 3.18). PAC was developed initially to cut materials that do not permit the use of the oxyfuel process—stainless steel and aluminum. It was found, however, that plasma arc cutting offered economic advantages when applied to thinner sections of carbon steel, especially those less than 25 mm (1 in) thick. PAC can cut thin steels at higher speeds than oxyfuel cutting, providing the power supply is of ample capacity. The higher travel speeds have corollary advantages such as reduced distortion and reduced metallurgical changes on the cut surface.

FIGURE 3.18 PAC process.

Because the thermal energy generated during the oxidation process with oxyfuel cutting is not present in plasma cutting, PAC is not economical when thick sections of steel are cut. PAC is used to cut thick sections of stainless steel and aluminum where oxyfuel cutting is not viable. PAC may be used manually, or it can be automated. Larger parts can be on large cutting tables. The tables may be filled with water, and the parts being cut submerged in the water. This keeps noise, smoke, and distortion to a minimum.

3.4.10 Air Carbon Arc Cutting and Gouging

AWS A3.0 defines air carbon arc cutting (CAC-A) as “a carbon arc cutting process variation removing molten metal with a jet of air.” Air carbon arc gouging is the same process, but is used to produce cavities in material, either for the preparation of U- or J-groove profiles, for back-gouging joints, or for excavating material for a weld repair. The air carbon arc gouging system (Fig. 3.19) utilizes an electric arc to melt the base material; a high-velocity jet of compressed air subsequently blows the molten material away.

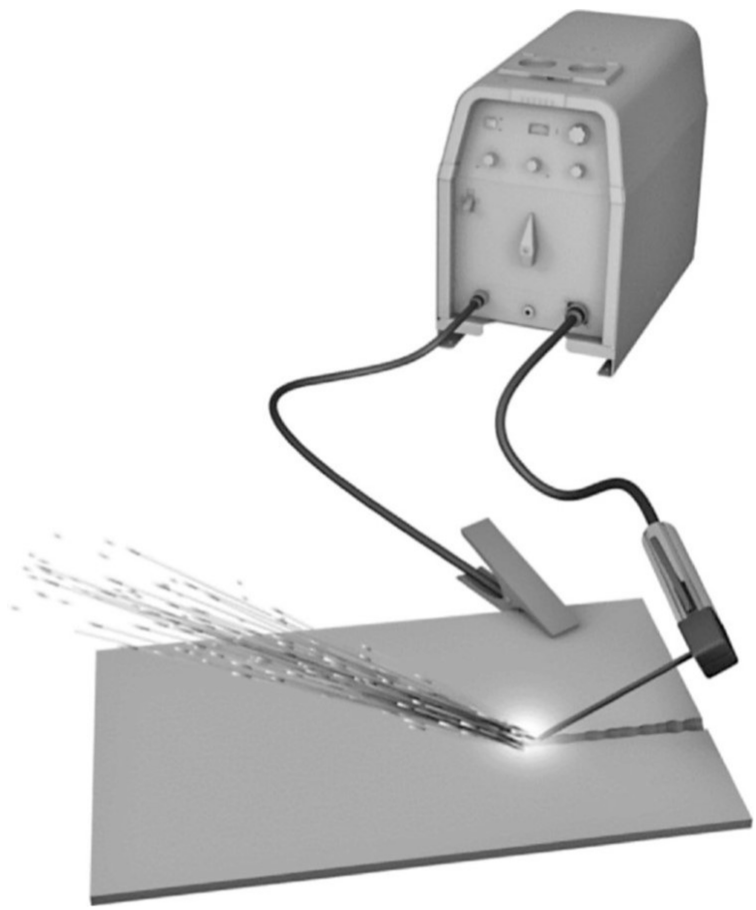

FIGURE 3.19 CAC-A and gouging process.

The process uses a standard welding power source and typically a copper-coated carbon electrode. Gouging electrodes come in a range of sizes: larger electrodes require more energy but remove metal more rapidly. Air carbon arc gouging may be manually or automatically performed, the latter being more popular for the preparation of U- and J-groove weld profiles. Metal removal is fast and the equipment simple and relatively inexpensive. The process generates considerable noise, smoke, and a shower of sparks, all of which can create safety hazards.

3.5 WELDED JOINT DESIGN

For the purposes of this chapter which focuses on the production side of welded connections, it will be assumed that the engineer or connection detailer has specified in the contract documents or design drawings the type of weld that is required (CJP, PJP, fillet, etc.), and the size and length of the weld. This section of chapter 3 will deal with the welded joint design considerations necessary to ensure a weld that meets the structural requirements, that the requisite quality is achieved, and the weld can be made in an economical way. Three weld types will be considered: CJPs, PJPs, and fillets. Properly identifying a welding joint on a drawing is done by using the symbols listed in ANSI/AWS A2.4:2012, *Standard Symbols for Welding, Brazing and Nondestructive Examination.*

AWS D1.1 prequalified options will be discussed in this section. While alternate details can be qualified by test, most structural welding utilizes prequalified WPSs, and accordingly, prequalified joint details are required.

The typical and preferred practice is for the connection designer to specify the weld type and size, and not specify the nature of the groove weld details, leaving that decision up to the contractor doing the work. This section will assume that practice is followed, and will focus on why a contractor may select one detail over another.

A key factor in selecting various joint details suitable for a given weld type is the issue of cost. Generally speaking, economy is achieved when the details of the welded joint are such that the required strength can be achieved with the least volume of weld metal. The issues of cost are more complex than merely the volume of weld metal required to fill the joint; other factors such as the cost of preparing the joint must also be considered. The material handling costs must also be evaluated; welds that can be completed without flipping plates over may prove to be less costly even though more weld metal may be required. Angular distortion may be easier to control with double-sided details, and some details resist lamellar tearing better than others. Selection of ideal joint details is an art and because of the myriad factors involved, the best solution for one contractor may be different than the ideal solution for another contractor.

Table 3.1 compares the volume of weld metal required to fill V-, bevel, U-, and J-groove details in various thicknesses. Given the general assumption that weld metal volumes are proportional to the overall welding cost, the relative economics can be seen. As will be explained in the various sections on the weld types, Table 3.1 does not tell the whole story. Material preparation costs are not the same, material handling costs are ignored in the table, as are the costs of backgouging and backing.

The prequalified joint details have a long history of providing conditions conducive to good fusion and profiles that discourage weld cracking. However, the D1.1 warns against the thoughtless use of prequalified joint details in clause 2.3.5.4, as follows: “The joint details described in 3.12 and 9.10 (PJP) and 3.13 and 9.11 (CJP) have repeatedly demonstrated their adequacy in providing the conditions and clearances necessary for depositing and fusing sound weld metal to base metal. However, the use of these details shall not be interpreted as implying consideration of the effects of welding process on base metal beyond the fusion boundary nor suitability of the joint detail for a given application.” Accordingly, in clause 1.4.1, the engineer is obligated by AWS D1.1 to “…determine the suitability of all joint details to be used in a welded assembly.”

3.5.1 CJP Groove Welds

When the drawings call for a CJP weld, the welded connection is expected to develop the full strength of the attached material. The drawings may prohibit left-in-place steel backing in which case the contractor will either select from the prequalified, double-sided options, or use steel backing and require its removal after welding.

Unless otherwise restricted, there are five groove weld types from which to choose: square, V-, bevel-, U-, or J-groove details. For each of these, there is a single-sided and double-sided option. Some groove weld details cannot be used in some joint details. For example, V-groove details are impossible to apply to T joints. The general characteristics, applications, advantages, and limitations of each of these five groove weld types are discussed.

Square Groove Welds. As shown in Fig. 3.20, square groove welds have edges that are cut square, or 90°, to the surface of the material being joined. Typically used in butt joints, square groove welds can be used in T or corner joints in some cases. In all cases, D1.1 limits the thickness of material in which square groove welds can be used. The root opening may be zero (i.e., butted tight) or may contain a gap.

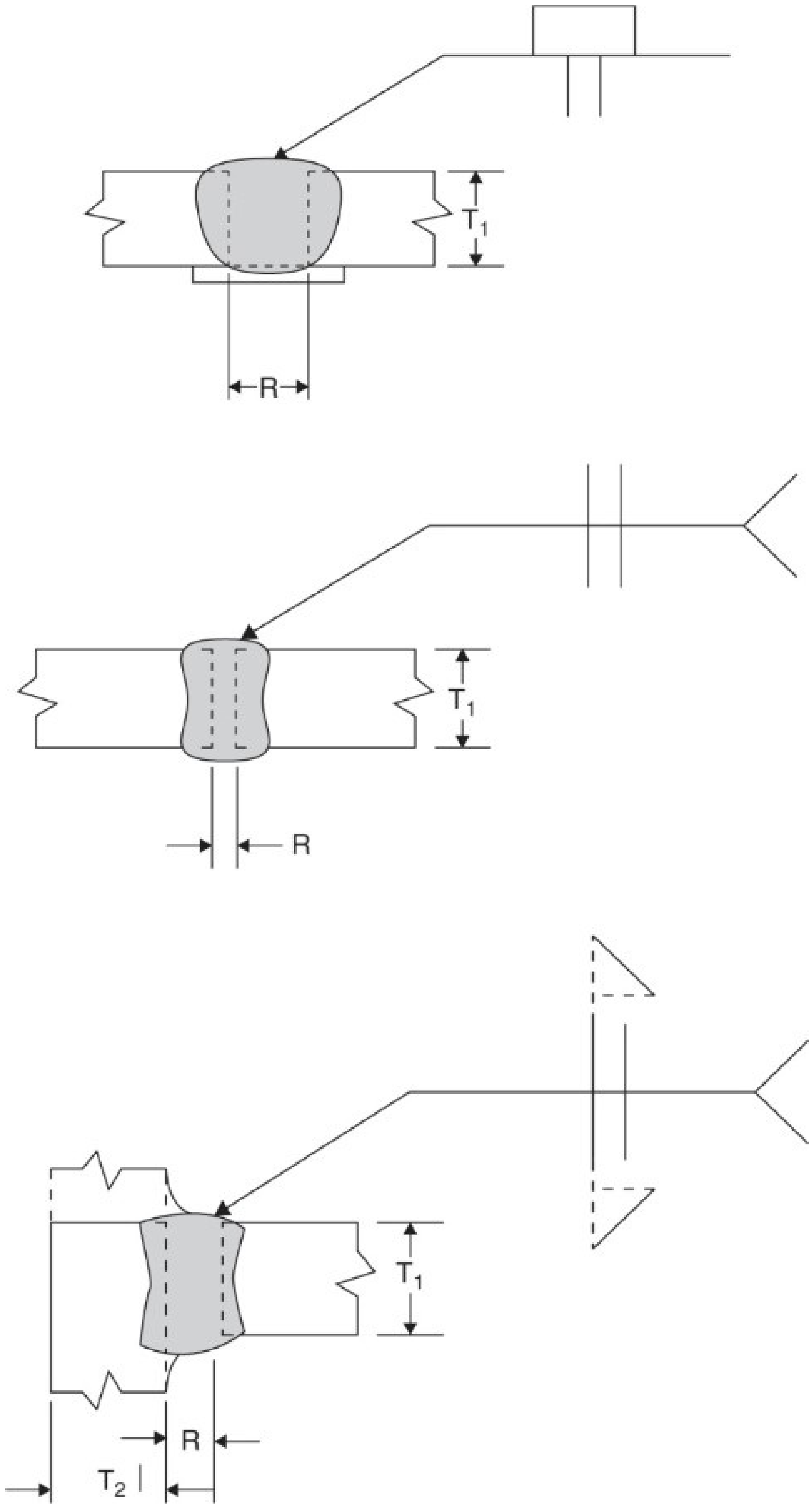

FIGURE 3.20 Square groove joint details. (AWS D1.1/D1.1M:2015, Figure 3.4, adapted and reproduced with permission of the American Welding Society, Miami, FL.)

The greatest advantage of these joints is achieved from the simplicity of the joint preparation: the joint may be the as-received edge of the part, a sheared or a thermally cut surface. Fit-up tolerances are tighter for this detail and the most obvious limitation is on the thickness of the prequalified joint. Most structural connections will involve thicker materials, making this option unavailable in terms of prequalified WPSs.

V-Groove Welds. V-groove details are illustrated in Fig. 3.21. Single-sided V-grooves may use steel backing, in which case they will have a positive root opening dimension and no root face dimension. Alternately, single V-groove weld may require back gouging, in which case there will be no backing and a root face dimension. Double-sided V-grooves will have no backing but may have a small root opening and small root face dimension. V-grooves can be applied to butt and corner joints.

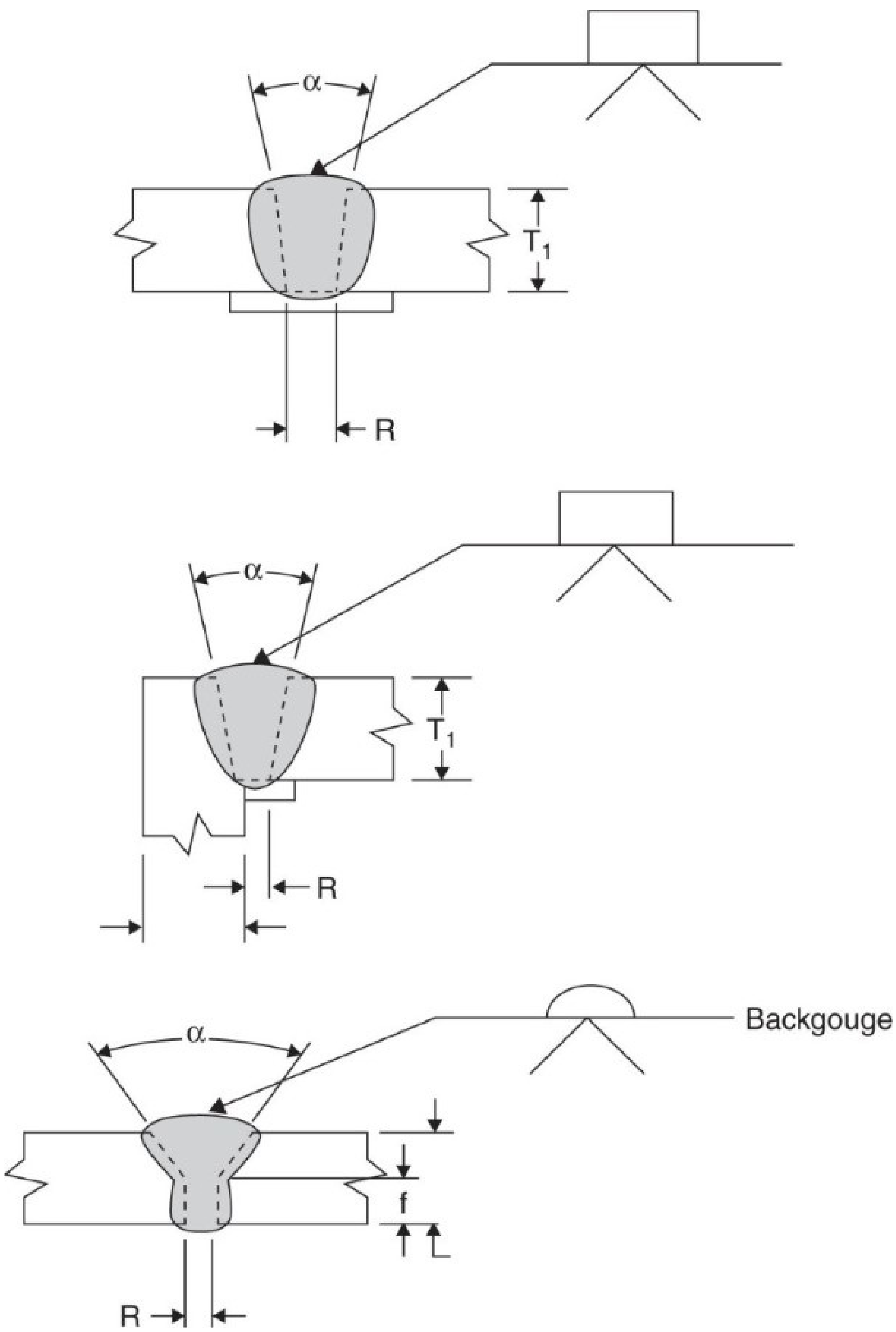

FIGURE 3.21 V-groove joint details. (AWS D1.1/D1.1M:2015, Figure 3.4, adapted and reproduced with permission of the American Welding Society, Miami, FL.)

V-groove welds are often used for flat position butt splices. The inclined surfaces accommodate sidewall fusion a bit more easily than do bevel-groove welds. The single V-groove welds with backing are more tolerant of fit-up variations than are the single V details that require backgouging. Single-sided V-groove welds are economical choices for steel less than about 40 mm (1.5 in) thick. For greater thicknesses, up to approximately 100 mm (4 in) thicknesses, double V-grooves are an economical choice.

For material over 100 mm (4 in) thicknesses, the special double V-grooves with a spacer bar can be utilized (Fig. 3.22). The spacer bar functions as an internal backing bar. The key advantage of this detail is that the joint can be prepared with traditional flame cutting equipment, yet it utilizes smaller included angles than would be required for traditional double V-groove details (which would have no root opening, but larger included angles). After the first side is at least partially welded, the entire spacer bar is gouged away before welding on the second side begins.

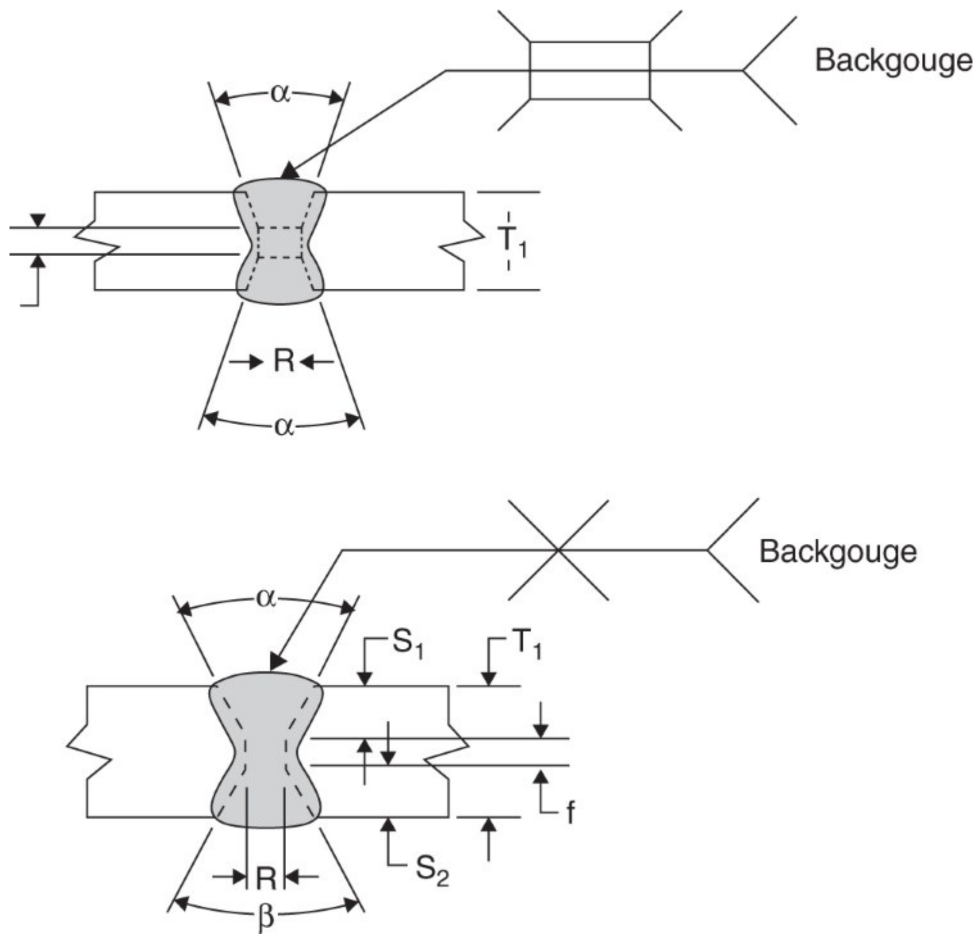

FIGURE 3.22 Double V-groove joint detail with and without spacer bar. (AWS D1.1/D1.1M:2015, Figure 3.4, adapted and reproduced with permission of the American Welding Society, Miami, FL.)

Bevel-Groove Welds. Bevel-groove welds involve one square edge, and one beveled edge, as shown in Fig. 3.23. Since only one edge needs to be beveled, joint preparation costs may be reduced. Bevel-groove welds are ideal for horizontal grooves where the horizontal member ideally supports the molten metal during welding. The vertical edge may be more difficult to consistently fuse than when both members have inclined surfaces as is the case with V-groove welds.

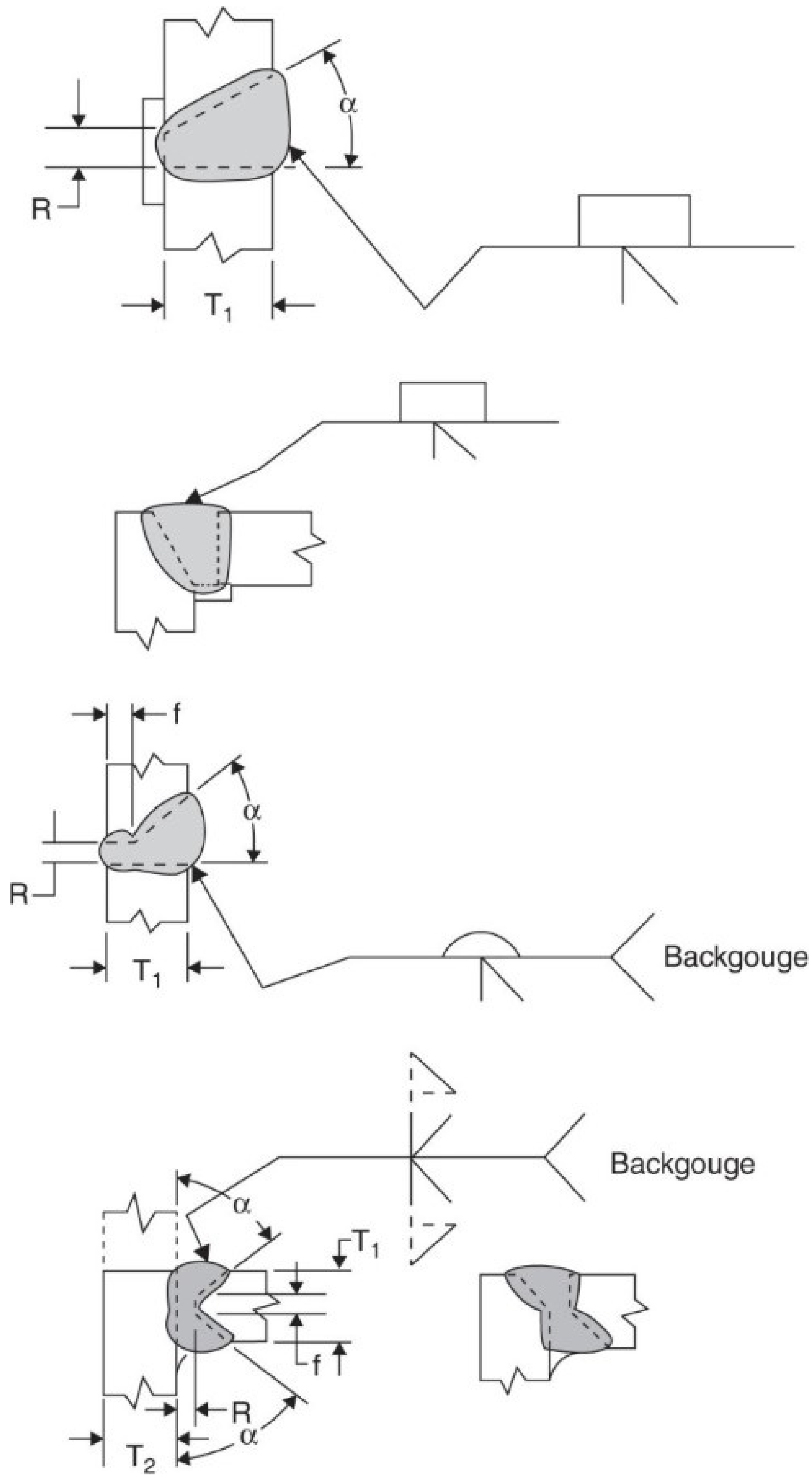

FIGURE 3.23 Bevel- and double bevel-groove joint details. (AWS D1.1/D1.1M:2015, Figure 3.4, adapted and reproduced with permission of the American Welding Society, Miami, FL.)

Since bevel-groove joints are nonsymmetrical, when it is important to identify the member to be beveled, the weld symbol arrow uses a double break, or two bends, and the arrow points to the member to be beveled as shown in Fig. 3.23.

U-Groove Welds. U-groove weld details are shown in Fig. 3.24. The geometry is defined in terms of a root radius, an included angle, and a root face dimension. U-groove weld details have root conditions that are ideal for achieving the root pass: the U-shaped cavity in the root is free of the planer intersections or "corners" associated with bevel and groove welds where the joint faces meet the backing. As a result, U-groove details have less likelihood of trapping slag in the root.

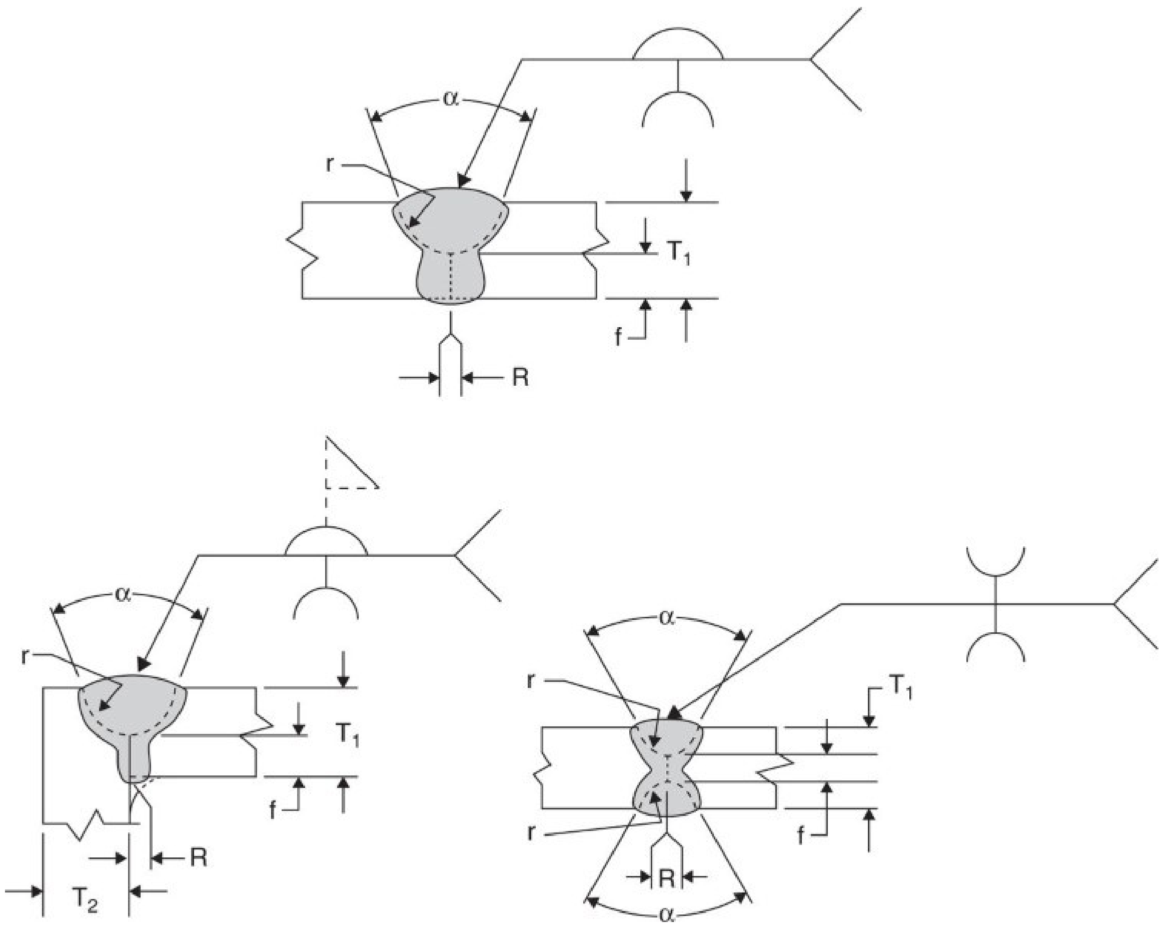

FIGURE 3.24 U-groove and double U-groove joint details. (AWS D1.1/D1.1M:2015, Figure 3.4, adapted and reproduced with permission of the American Welding Society, Miami, FL.)

U-groove details have "built-in" backing: the root of the weld, along with the root face dimension, keeps the weld metal from dripping through the joint. The root will be backgouged and welded from the opposite side to obtain complete joint penetration.

Perhaps the chief advantage of U-groove welds is that they require less weld metal to fill a joint in similar steel thicknesses than do the V-groove or bevel-groove options, due to the smaller included angles that are used. See Table 3.1. This advantage is partially offset by the higher joint preparation cost. U-groove details are either machined, or prepared by arc-air cutting; when AAC is employed, the operation is usually done automatically.

J-Groove Welds. J-groove details, shown in Fig. 3.25, have some of the same advantages and applications as do bevel groove details, yet require less weld metal volume. Like U-groove details, J-groove details are more expensive and complicated to prepare. As the joint thickness increases, J-groove details become more economical.

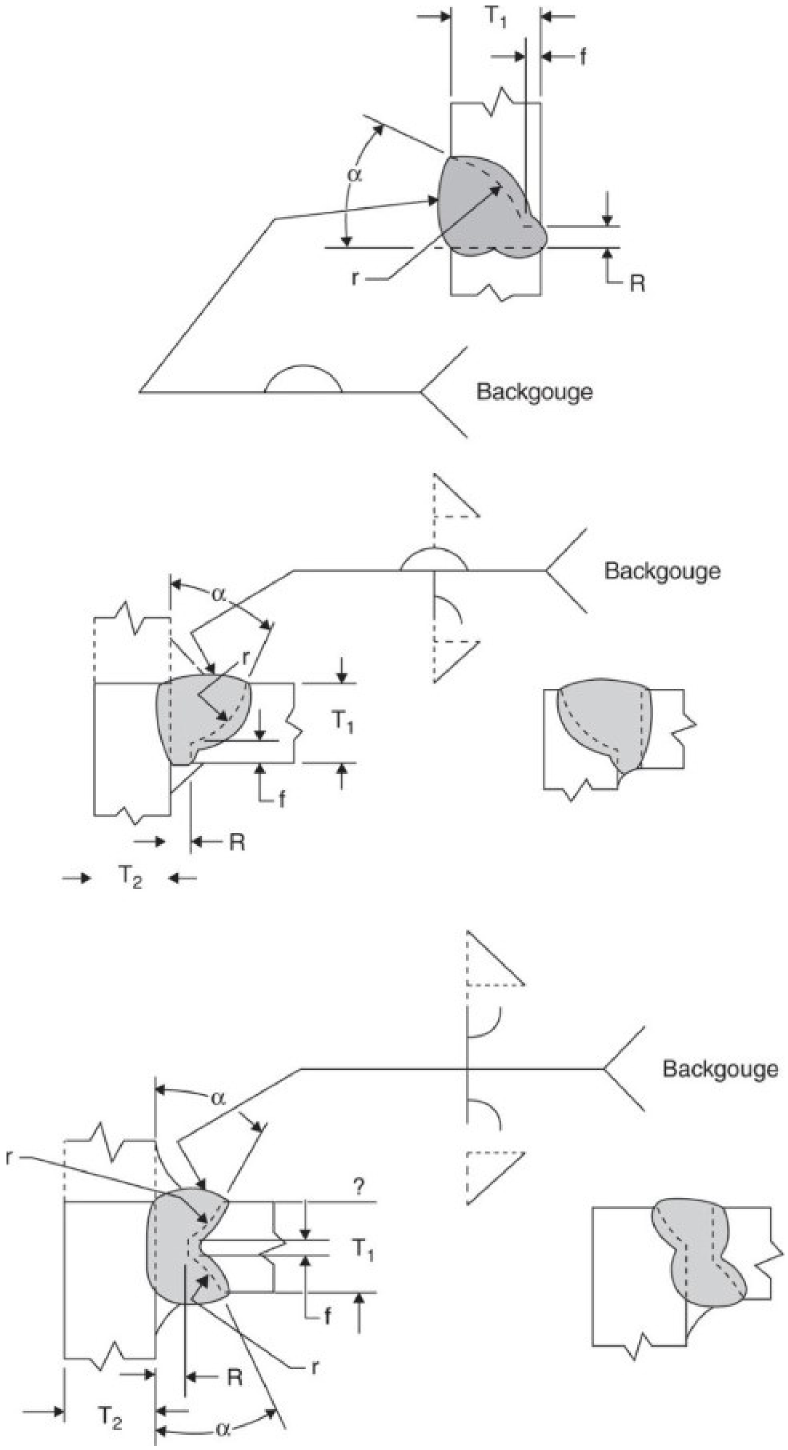

FIGURE 3.25 J-groove and double J-groove joint details. (AWS D1.1/D1.1M:2015, Figure 3.4, adapted and reproduced with permission of the American Welding Society, Miami, FL.)

Since J-groove details are nonsymmetrical like bevel-groove details, the weld symbol uses the break in the arrow line to point to the surface that will receive the J groove.

3.5.2 PJP Groove Welds

PJP groove welds may use square edge, V-groove, bevel-groove, U-groove, or J-groove preparations, just like the CJP counterpart. PJPs may be single or double sided, just like CJPs. Unlike CJPs, PJPs never use backing, and unlike CJPs, the whole cross section of the joint will not be fused with PJPs (Figs. 3.26 to 3.30).

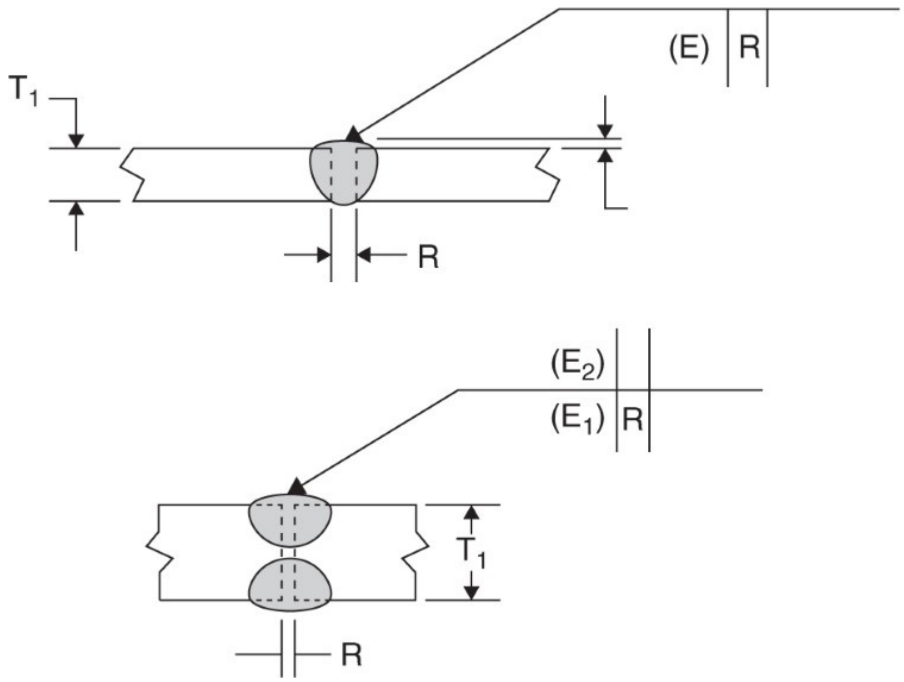

FIGURE 3.26 PJP square groove joint details. (AWS D1.1/D1.1M:2015, Figure 3.2, adapted and reproduced with permission of the American Welding Society, Miami, FL.)

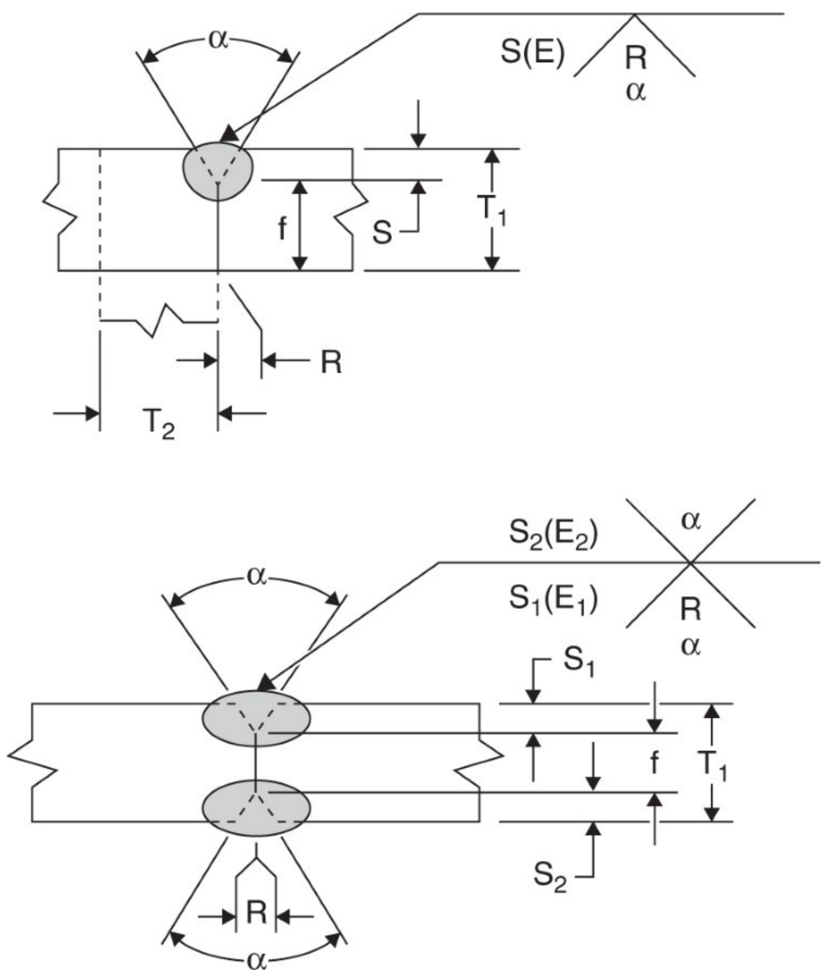

FIGURE 3.27 PJP single and double V-groove joint detail Figure 26 — PJP square groove joint details. (AWS D1.1/D1.1M:2015, Figure 3.2, adapted and reproduced with permission of the American Welding Society, Miami, FL.)

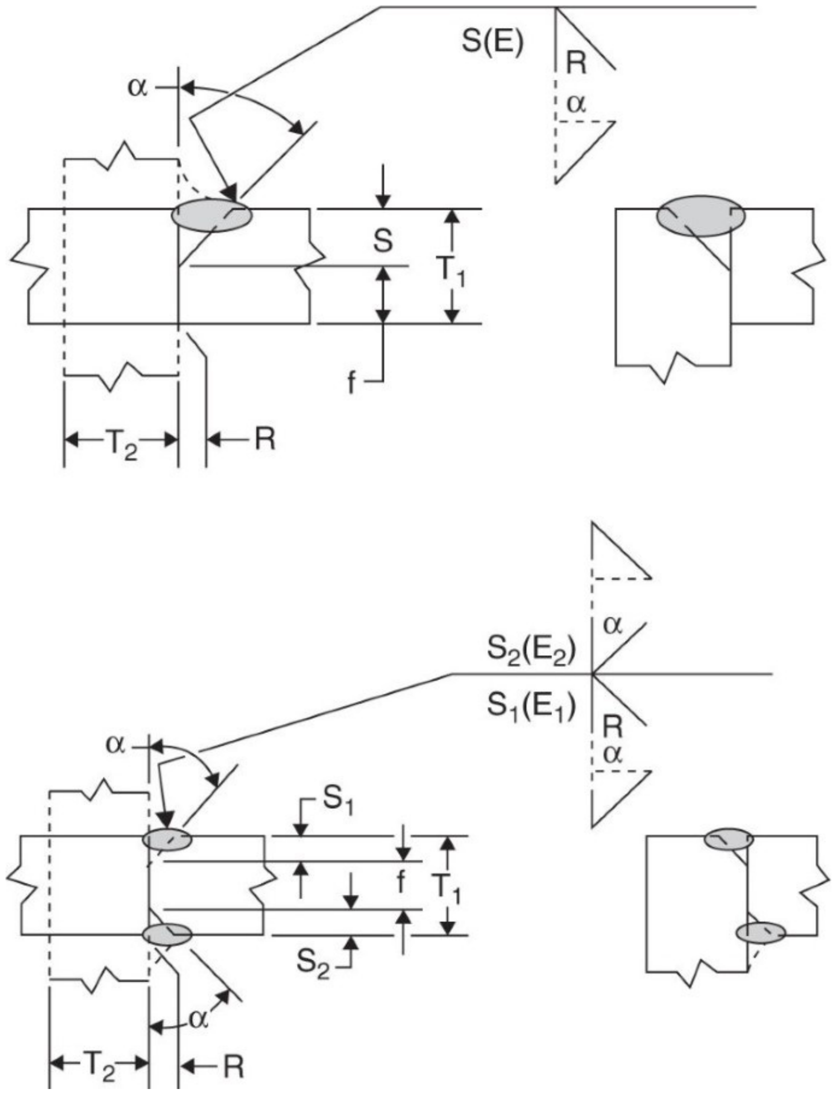

FIGURE 3.28 PJP single and double-bevel joint details. (AWS D1.1/D1.1M:2015, Figure 3.2, adapted and reproduced with permission of the American Welding Society, Miami, FL.)

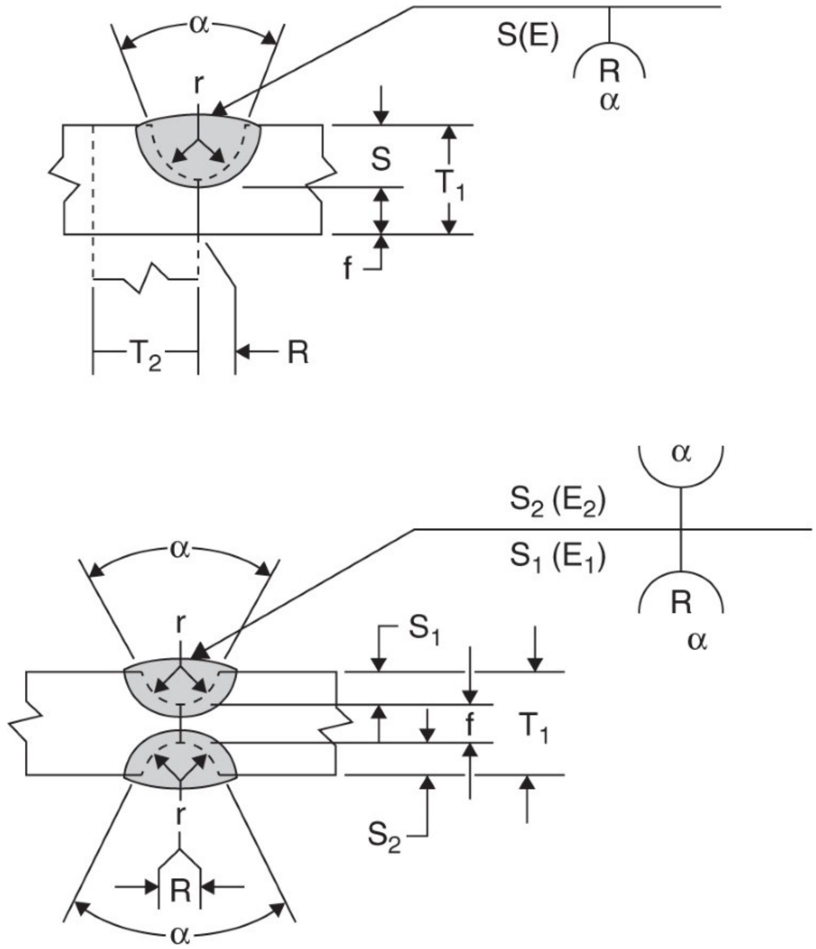

FIGURE 3.29 PJP single and double U-groove joint details. (AWS D1.1/D1.1M:2015, Figure 3.2, adapted and reproduced with permission of the American Welding Society, Miami, FL.)

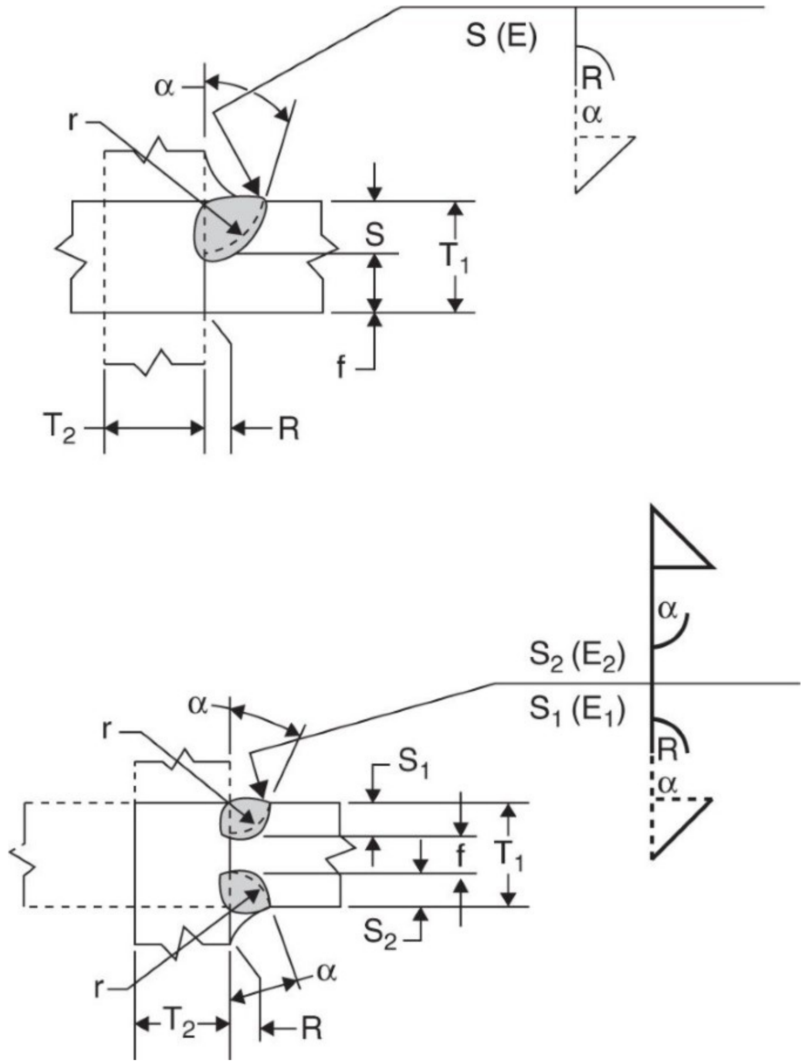

FIGURE 3.30 PJP single and double J-groove joint details. (AWS D1.1/D1.1M:2015, Figure 3.2, adapted and reproduced with permission of the American Welding Society, Miami, FL.)

The engineer or connection designer is required to specify the PJP throat size. By definition, the throat of a PJP is the least distance from the point in the root where there is fusion to the face of the weld, minus any reinforcement. This dimension is typically provided on the contract documents or design drawings, and more specifically on the weld symbol within parentheses. The throat dimension is abbreviated as the "E" dimension in D1.1. The depth of the bevel is known as the "S" dimension. Depending on the welding process, the position of welding and the included angle of the joint, "E" may be equal to "S" or may be less than "S."

To ensure the required "E" dimension is achieved, even if fusion is not achieved to root, the prequalified PJP details shown in D1.1 list the relationship between "E" and "S." If E = S, the situation is simple: the shop drawing would list the "S" dimension ahead of the "E" dimension which is shown within parentheses, and the same dimension would be shown for each.

If combination of welding variables is such that "E" is less than "S," then the depth of the bevel needs to be increased by the dimension shown on the prequalified joint detail [typically 3 mm (⅛ in)] so the required throat dimension is achieved. For example, if the drawings call for a throat dimension ("E") of 10 mm (⅜ in), but E = S – 3 mm [E = S – ⅛ in], then the depth of bevel (“S”) needs to be increased to 10 + 3 or 13 mm [⅜ + ⅛ or ½ in].

3.5.3 Fillet Welds

There are two specified dimensions associated with most fillet welds: the leg size and the length. The leg size is directly related to the throat dimension, which is the theoretical plane on which the weld will ultimately fail with sufficient load. The practice of both AISC and AWS on fillet welds is, in general, to reference the leg size when determining and specifying weld sizes.

An exception to the practice of specifying fillet weld leg sizes involve skewed T-joints. These nonorthogonal joints have an acute and an obtuse side. When the acute side is less than 80°, or when the obtuse side is greater than 100°, then AWS D1.1 clause 2.3.4(2) calls for the engineer to specify the throat size (not the leg size). For these welds, the shop drawings are required by AWS D1.1 clause 2.3.5.2(2) to “…show the detailed arrangement of welds and required leg size to account for effects of joint geometry and, where appropriate, the Z-loss reduction for the process to be used and the angle.” The Z-loss factor is used to account for incomplete fusion in the root of acute T-joints.

The reason for this deviation from the standard practice is due to the variety of issues involved (welding process, position, thickness of the skewed member, angle of skew) that are likely unknown to the engineer. Thus, the engineer specifies the throat dimension directly and the contractor works out the details necessary to deliver the required throat dimension.

As to the length, fillet welds may be the full length of the joint, partial length, or intermittent. Dimension for partial length and intermittent welds should be shown; when no weld length is shown, the default assumption is that the weld is full length.

According to AWS D1.1, clause 2.9.3.1 “Fillet weld terminations may extend to the ends or sides of parts or may be stopped short or may have end returns except as limited by the following cases.” The permissive language (“may”) permits any of the three conditions, except in four standard exceptions that follow. The engineer is to spell out where special terminations are required; otherwise, the contractor can select the type of termination that will be utilized.

3.6 WELDING PROCEDURES

Within the welding industry, the term welding procedure specification (or “WPS”) is used to signify the combination of variables that are to be used to make a certain weld. The terms welding procedure, or simply procedure, may be used. At a minimum, the WPS consists of the following:

Process (SMAW, FCAW, etc.)

Electrode specification (AWS A5.1, A5.20, etc.)

Electrode classification (E7018, E71T-1, etc.)

Electrode diameter

Electrical characteristics (DC+, DC–, AC)
Base metal specification (A36, A992, etc.)
Minimum preheat and interpass temperature
Welding current (amperage)/wire-feed speed
Arc voltage
Travel speed
Position of welding
Postweld heat treatment (when applicable)
Shielding gas type and flow rate (when applicable)
Joint design details

The welding procedure is somewhat analogous to a cook's recipe. The procedure outlines the steps required to make a quality weld under specific conditions.

3.6.1 Effects of Welding Variables

The effects of the variables are somewhat dependent on the welding process being employed, but general trends apply to all the processes. It is important to distinguish the difference between constant current (CC) and constant voltage (CV) electrical welding systems. Shielded metal arc welding is always done with a CC system. Flux-cored welding and gas metal arc welding generally are performed with CV systems. Submerged arc may utilize either.

Amperage is a measure of the amount of current flowing through the electrode and the work. It is a primary variable in determining heat input. Generally, an increase in amperage means higher deposition rates, deeper penetration, and more admixture. The amperage flowing through an electric circuit is the same, regardless of where it is measured. The role of amperage is best understood in the context of heat input and current density considerations. For CV welding, an increase in wire-feed speed will directly increase amperage.

For SMAW on CC systems, the machine setting determines the basic amperage, although changes in the arc length (controlled by the welder) will further change amperage. Longer arc lengths reduce amperage.

Arc voltage is directly related to arc length. As the voltage increases, the arc length increases, as does the demand for arc shielding. For CV welding, the voltage is determined primarily by the machine setting, so the arc length is relatively fixed in CV welding. For SMAW on CC systems, the arc voltage is determined by the arc length, which is manipulated by the welder. As arc lengths are increased with SMAW, the arc voltage will increase and the amperage will decrease. Arc voltage also controls the width of the weld bead, with higher voltages generating wider beads. Arc voltage has a direct effect on the heat input computation.

The voltage in a welding circuit is not constant, but is composed of a series of voltage drops. Consider the following example: Assume the power source delivers a total system voltage of 40 V. Between the power source and the welding head or gun, there is a voltage drop of perhaps 3 V associated with the input-cable resistance. From the point of attachment of the work lead to the power source work terminal, there is an additional voltage drop of, say, 7 V. Subtracting the 3 V and the 7 V from the original 40 V, this leaves 30 V for the arc. This example illustrates how important it is to ensure that the voltages used for monitoring welding procedures properly recognize any losses in the welding circuit.

The most accurate way to determine arc voltage is to measure the voltage drop between the contact tip and the workpiece. This may not be practical for semiautomatic welding, so voltage is typically read from a point on the wire feeder (where the gun and cable connection is made) to the workpiece. For SMAW, welding voltage is not usually monitored, since it is constantly changing and cannot be controlled except by the welder. Skilled workers hold short arc lengths to deliver the best weld quality.

Travel speed, measured in mm per minute [inches per minute], is the rate at which the electrode is moved relative to the joint. All other variables being equal, travel speed has an inverse effect on the size of the weld beads. As the travel speed increases, the weld size will decrease. Extremely low travel speeds may result in reduced penetration, as the arc impinges on a thick layer of molten metal and the weld puddle rolls ahead of the arc. Travel speed is a key variable used in computing heat input; reducing travel speed increases heat input.

Wire-feed speed is a measure of the rate at which the electrode is passed through the welding gun and delivered to the arc. Typically measured in meters per minute [inches per minute], the wire-feed speed is directly proportional to deposition rate and directly related to amperage. When all other welding conditions are maintained constant (for example, the same electrode type, diameter, electrode extension, arc voltage, and electrode extension), an increase in wire-feed speed will directly lead to an increase in amperage. For slower wire-feed speeds, the ratio of wire-feed speed to amperage is relatively constant and linear.

For higher levels of wire-feed speed, it is possible to increase the wire-feed speed at a disproportionately high rate compared to the increase in amperage. When these conditions exist, the deposition rate per amp increases but at the expense of penetration.

Wire-feed speed is the preferred method of maintaining welding procedures for constant-voltage wire-feed processes. The wire-feed speed can be independently adjusted, and measured directly, regardless of the other welding conditions. It is possible to utilize amperage as an alternative to wire-feed speed although the resultant amperage for a given wire-feed speed may vary, depending on the polarity, electrode diameter, electrode type, and electrode extension. Although equipment that monitors wire-feed speed has been available for many years, AWS D1.1 uses amperage as the primary method for procedure documentation. D1.1 does permit the use of wire-feed speed control instead of amperage, providing a wire-feed speed-amperage relationship chart is available for comparison.

The distance from the contact tip to the work is appropriately called the contact tip to work distance, or CTWD. The CTWD distance includes the electrode extension (also known as stickout) plus the arc length. The terms CTWD and stickout apply only to the wire-feed processes. As the electrode extension is increased in a constant-voltage system, the electrical resistance of the electrode increases, causing the electrode to be heated. This is known as resistance heating or I^2R heating. As the amount of heating increases, the arc energy required to melt the electrode decreases. Longer electrode extensions may be employed to gain higher deposition rates at a given amperage. When the electrode extension is increased without any change in wire-feed speed, the amperage will decrease. This results in less penetration and less admixture. With the increase in stickout, it is common to increase the machine voltage setting to compensate for the greater voltage drop across the electrode.

In constant-voltage systems, it is possible to simultaneously increase the electric stickout and wire-feed speed in a balanced manner so that the current remains constant. When this is done, higher deposition rates are attained. Other welding variables such as voltage and travel speed must be adjusted to maintain a stable arc and to ensure quality welding. The ESO variable should always be within the range recommended by the manufacturer.

Electrode diameter means larger electrodes can carry higher welding currents. For a fixed amperage, however, smaller electrodes result in higher deposition rates. This is because of the effect on current density discussed in the following.

Polarity is a definition of the direction of current flow. Positive polarity (DC+, reverse) is achieved when the electrode lead is connected to the positive terminal of the direct-current (DC) power supply. The work lead is connected to the negative terminal. Negative polarity (DC–, straight) occurs when the electrode is connected to the negative terminal and the work lead to the positive terminal. Alternating current (AC) is not a polarity, but a current type. With AC, the electrode is alternately positive and negative. Submerged arc is the only process that commonly uses either electrode positive or electrode negative polarity for the same type of electrode. AC may also be used. For a fixed wire-feed speed, a submerged arc electrode will require more amperage on positive polarity than on negative. For a fixed amperage, it is possible to utilize higher wire-feed speeds and deposition rates with negative polarity than with positive. AC exhibits a mix of both positive and negative polarity characteristics.

The magnetic field that surrounds any dc conductor can cause a phenomenon known as arc blow, where the arc is physically deflected by the field. The strength of the magnetic field is proportional to the square of the current value, so this is a more significant potential problem with higher currents. AC is less prone to arc blow, and can sometimes be used to overcome this phenomenon.

Heat input is proportional to the welding amperage, times the arc voltage, divided by the travel speed. Higher heat inputs relate to larger weld cross-sectional areas and larger heat-affected zones (HAZs), which may negatively affect mechanical properties in that region. Higher heat input generally results in slightly decreased yield and tensile strength in the weld metal, and generally lowers notch toughness because of the interaction of bead size and heat input.

Current density is determined by dividing the welding amperage by the cross-sectional area of the electrode. For solid electrodes, the current density is therefore proportional to I/d^2. For tubular electrodes where current is conducted by the sheath, the current density is related to the area of the metallic cross section. As the current density increases, there will be an increase in deposition rates, as well as penetration. The latter will increase the amount of admixture for a given joint. Notice that this may be accomplished by either the amperage or decreasing the electrode size. Because the electrode diameter is a squared function, a small decrease in diameter may have a significant effect on deposition rates and plate penetration.

Preheat and interpass temperature are used to control cracking tendencies, typically in the base materials. Regarding weld metal properties, for most carbon-manganese-silicon systems, a moderate interpass temperature promotes good notch toughness. Preheat and interpass temperatures greater than 290°C (550°F) may negatively affect notch toughness. Therefore, careful control of preheat and interpass temperatures is critical.

3.6.2 Purpose of Welding Procedure Specifications

The particular values for the variables discussed previously have a significant effect on weld soundness, mechanical properties, and productivity. It is therefore critical that those procedural values used in the actual fabrication and erection be appropriate for the specific requirements of the applicable code and job specifications. Welds that will be architecturally exposed, for example, should be made with procedures that minimize spatter, encourage exceptional surface finish, and have limited or no undercut. Welds that will be covered with fireproofing, in contrast, would naturally have less restrictive cosmetic requirements.

Many issues must be considered when selecting welding procedure values. While all welds must have fusion to ensure their strength, the required level of penetration is a function of the joint design and the weld type. All welds are required to deliver a certain yield and/or tensile strength, although the exact level required is a function of the connection design. Not all welds are required to deliver minimum specified levels of notch toughness. Acceptable levels of undercut and porosity are a function of the type of loading applied to the weld.

Determination of the most efficient means by which these conditions can be met cannot be left to the welders, but should be determined by knowledgeable welding technicians and engineers who create written welding procedure specifications and communicate those requirements to welders by the means of these documents. The WPS is the primary tool that is used to communicate to the welder, supervisor, and inspector how a specific weld is to be made. The suitability of a weld made by a skilled welder in conformance with the requirements of a WPS can only be as good as the WPS itself. The proper selection of procedure variable values must be achieved in order to have a WPS appropriate for the application. This is the job of the welding expert who generates or writes the WPS. The welder is generally expected to be able to follow the WPS, although the welder may not know how or why each particular variable was selected. Welders are expected to ensure welding is performed in accordance with the WPS. Inspectors do not develop WPSs, but should ensure that they are available and are followed.

AWS D1.1 requires written welding procedures for all fabrication performed. The inspector is obligated to review the WPSs and to make certain that production welding parameters conform to the requirements of the code. These WPSs are required to be written, regardless of whether they are prequalified or qualified by test. Each fabricator or erector is responsible for the development of WPSs. One prevalent misconception is that if the actual parameters under which welding will be performed meet all the conditions for “prequalified” status, written WPSs are not required. This is not true; all WPSs must be written.

The WPS is a communication tool, and it is the primary means of communication to all the parties involved regarding how the welding is to be performed. It must therefore be readily available to foremen, inspectors, and the welders.

It is in the contractor’s best interest to ensure that efficient communication is maintained with all parties involved. Not only can quality be compromised when WPSs are not available, but productivity can suffer as well. Regarding quality, the limits of suitable operation of the particular welding process and electrode for the steel, joint design, and position of welding must be understood. It is obvious that the particular electrode employed must be operated on the proper polarity, proper shielding gases must be used, and amperage levels must be appropriate for the diameter of electrode and for the thickness of material on which welding is performed. Other issues are not necessarily so obvious. The required preheat for a particular application is a function of the grade(s) of steel involved, the thickness(es) of material, and the type of electrode employed (whether low hydrogen or non-low hydrogen). The required preheat level can be communicated by means of the written WPS.

Lack of conformance with the parameters outlined in the WPS may result in the deposition of a weld that does not meet the quality requirements imposed by the code or the job specifications. When an unacceptable weld is made, the corrective measures to be taken may necessitate weld removal and replacement, an activity that routinely increases the cost of that particular weld 10-fold. Avoiding these types of unnecessary activities by clear communication has obvious ramifications in terms of quality and economics.

There are other economic issues to be considered as well. In a most general way, the cost of welding is inversely proportional to the deposition rate. The deposition rate, in turn, is directly tied to the wire-feed speed of the semiautomatic welding processes. If it is acceptable, for example, to make a given weld with a wire-feed speed of 5.0 m/min (200 in/min), then a weld made at 4.0 m/min (160 in/min) (which may meet all the quality requirements) would cost approximately 25 percent more than the weld made with the optimum procedure. Conformance with WPS values can help ensure that construction is performed at rates that are conducive to the required weld quality and are economical as well. Some wire feeders have the ability to preset welding parameters, coupled with the digital LED display or analog meters that indicate operational parameters, which can assist in maintaining and monitoring WPS parameters. The code imposes minimum requirements for a given project. Additional requirements may be imposed by contract specifications. The same would hold true regarding WPS values. Compliance with the minimum requirements of the code may not be adequate under all circumstances. Additional requirements can be communicated through the WPS, such as recommendations imposed by the steel producer, electrode manufacturer, or others can and should be documented in the WPS.

3.6.3 Prequalified Welding Procedure Specifications

The AWS D1.1 code provides for the use of prequalified WPSs. Prequalified WPSs are those that the AWS D1 Committee has determined to have a history of acceptable performance, and so does not subject them to the qualification testing imposed on all other welding procedures. The use of prequalified WPSs does not preclude their need to be in a written format. The use of prequalified WPSs still requires that the welders be appropriately qualified. All the workmanship provisions imposed in the fabrication section of the code apply to prequalified WPSs. The only code requirement exempted by prequalification is the nondestructive testing and mechanical testing required for qualification of welding procedures.

A host of restrictions and limitations imposed on prequalified welding procedures do not apply to welding procedures that are qualified by test. Prequalified welding procedures must conform with all the prequalified requirements in the code. Failure to comply with a single prequalified condition eliminates the opportunity for the welding procedure to be prequalified. The use of a prequalified welding procedure does not exempt the engineer from exercising engineering judgment to determine the suitability of the particular procedure for the specific application.

In order for a WPS to be prequalified, the following conditions must be met:

- The welding process must be prequalified. Only SMAW, SAW, GMAW (except GMAW-S), and FCAW WPSs may be prequalified.
- The base metal/filler metal combination must be prequalified.
- The minimum preheat and interpass temperatures prescribed in D1.1 must be employed.
- Specific welding requirements for the various weld types and welding processes must be maintained.
- A prequalified joint geometry must be used.

Even if prequalified joint details are employed, the welding procedure must be qualified by test if other prequalified conditions are not met. For example, if a prequalified detail is used on an unlisted steel, the welding procedures must be qualified by test.

The code does not imply that a WPS that is prequalified will automatically achieve the quality conditions required by the code. It is the contractor's responsibility to ensure that the particular parameters selected within the requirements of the prequalified WPS are suitable for the specific application.

Most contractors will determine preliminary values for a prequalified WPS based upon their experience, recommendations from publications such as the AWS Welding Handbooks, from AWS Welding Procedures Specifications (AWS B2.1), or other sources. It is the responsibility of the contractor to verify the suitability of the suggested parameters prior to the application of the actual procedure on a project, although the verification test need not be subject to the full range of procedure qualification tests imposed by the code. Typical tests will be made to determine soundness of the weld deposit (e.g., fusion, tie-in of weld beads, freedom from slag inclusions). The plate could be nondestructively tested or, as is more commonly done, cut, polished, and etched. The latter operations allow for examination of penetration patterns, bead shapes, and tie-in. Welds that are made with prequalified WPSs that meet the physical dimensional requirements (fillet weld size, maximum reinforcement levels, and surface profile requirements) and are sound (i.e., adequate fusion, tie-in, and freedom from excessive slag inclusions and porosity) should meet the strength and ductility requirements imposed by the code for welding procedures qualified by test. Weld soundness, however, cannot be automatically assumed just because the WPS is prequalified.

3.6.4 Guidelines for Preparing Prequalified WPSs

When developing prequalified WPSs, the starting point is a set of welding parameters appropriate for the general application being considered. Parameters for overhead welding will naturally vary from those required for down-hand welding. The thickness of material involved will dictate electrode sizes and corresponding current levels. The specific filler metals selected will reflect the strength requirements of the connection. Many other issues must be considered. Depending on the level of familiarity and comfort the contractor has with the particular values selected, welding a mock-up may be appropriate. Once the parameters that are desired for use in production are established, it is essential to check each of the applicable parameters for compliance with the D1.1.

3.6.5 Qualifying Welding Procedures by Test

There are two primary reasons why welding procedures may be qualified by test. First, it may be a contractual requirement. Secondly, one or more of the specific conditions to be used in the production may deviate from the prequalified requirements. In either case, a test weld must be made prior to the establishment of the final WPS. The first step in qualifying a welding procedure by test is to establish the procedure that is desired to be qualified. The same sources cited for the prequalified WPS starting points could be used for WPSs qualified by test. These will typically be the parameters used for fabrication of the test plate, although this is not always the case, as will be discussed later. In the simplest case, the exact conditions that will be encountered in production will be replicated in the procedure qualification test. This would include the welding process, filler metal, grade of steel, joint details, thickness of material, preheat values, minimum interpass temperature level, and the various welding parameters of amperage, voltage, and travel speed.

The initial parameters used to make the procedure qualification test plate beg for a name to define them, although there is no standard industry term. It has been suggested that “TWPS” be used, where the “T” could alternatively be used for temporary, test, or trial. In any case, it would define the parameters to be used for making the test plate since the validity of the particular parameters cannot be verified until successfully passing the required test. The parameters for the test weld are recorded on a procedure qualification record (PQR). The actual values used should be recorded on this document. The target voltage, for example, may be 30 V but, in actual fact, only 29 V were used for making the test plate. The 29 V would be recorded.

After the test plate has been welded, it is allowed to cool and the plate is subjected to the visual and nondestructive testing as prescribed by the code. The specific tests required are a function of the type of weld being made and the particular welding consumables. The types of qualification tests are described in D1.1, paragraph 4.5. In order to be acceptable, the test plates must first pass visual inspection followed by nondestructive testing (NDT). At the contractor’s option, either RT or UT can be used for NDT. The mechanical tests required involve bend tests (for soundness) macroetch tests (for soundness), and reduced section tensile tests (for strength). For qualification of procedures on steels with significantly different mechanical properties, a longitudinal bend specimen is possible. All weld metal tensile tests are required for unlisted filler metals. The nature of the bend specimens, whether side, face, or root, is a function of the thickness of the steel involved.

Once the number of tests has been determined, the test plate is sectioned and the specimens machined for testing. The results of the tests are recorded on the PQR. According to D1.1, if the test results meet all the prescribed requirements, the testing is successful and welding procedures can be established based upon the successful PQR. If the test results are unsuccessful, the PQR cannot be used to establish the WPS. If any one specimen of those tested fails to meet the test requirements, two retests of that particular type of test may be performed with specimens extracted from the same test plate. If both of the supplemental specimens meet the requirements, the D1.1 allows the tests to be deemed successful. If the test plate is over 11/2 in thick, failure of a specimen necessitates retesting of all the specimens at the same time from two additional locations in the test material.

It is wise to retain the PQRs from unsuccessful tests as they may be valuable in the future when another similar welding procedure is contemplated for testing.

The acceptance criteria for the various tests are prescribed in the code. The reduced section tensile tests are required to exceed the minimum specified tensile strength of the steel being joined.

Writing WPSs from Successful PQRs. When a PQR records the successful completion of the required tests, welding procedures may be written from that PQR. At a minimum, the values used for the test weld will constitute a valid WPS. The values recorded on the PQR are simply transcribed to a separate form, now known as a WPS rather than a PQR.

It is possible to write more than one WPS from a successful PQR. Welding procedures that are sufficiently similar to those tested can be supported by the same PQR. Significant deviations from those conditions, however, necessitate additional qualification testing. Changes that are considered significant enough to warrant additional testing are considered essential variables, and these are listed in D1.1, Tables 4.5, 4.6, and 4.7.

For example, consider an SMAW welding procedure that is qualified by test using an E8018-C3 electrode. From that test, it is acceptable to write a WPS that utilizes E7018 (since this is a decrease in electrode strength); however, writing a WPS that utilizes E9018-G electrode would not be permissible (because Table 4.5 lists an increase in filler metal classification strength as an essential variable). It is important to carefully review the essential variables in order to determine whether a previously conducted test may be used to substantiate the new procedure being contemplated.

D1.1, Table 4.1, defines the range of weld types and positions qualified by various tests. This table is best used, not as an after-the-fact evaluation of the extent of applicability of the test already conducted, but rather for planning qualification tests. For example, a test plate conducted in the 2G position qualifies the WPS for use in either the 1G or 2G position. Even though the first anticipated use of the WPS may be for the 1G position, qualifying in the 2G position may be advisable so that additional usage can be obtained from this test plate.

In a similar way, D1.1, Table 4.8, defines what changes can be made in the base metals used in production versus qualification testing. An alternative steel may be selected for the qualification testing simply because it affords additional flexibility for future applications.

If WPS qualification is performed on a non-prequalified joint geometry, and acceptable test results are obtained, WPSs may be written from that PQR utilizing any of the prequalified joint geometries (D1.1, Table 4.5, item 31).

3.6.6 Approval of WPSs

After a WPS is developed by the fabricator or erector, it is required to be reviewed in accordance to D1.1 requirements. For prequalified WPSs, the inspector is required to review the WPSs to ensure that they meet all the prequalified requirements.

The apparent logic behind the differences in approval procedures is that while prequalified WPSs are based upon well-established, time-proven, and documented welding practices, WPSs that have been qualified by test are not automatically subject to such restrictions. Even though the required qualification tests have demonstrated the adequacy of the particular procedure under test conditions, further scrutiny by the engineer is justified to ensure that it is applicable for the particular situation that will be encountered in production.

In practice, it is common for the engineer to delegate the approval activity of all WPSs to the inspector. There is a practical justification for such activity: the engineer may have a more limited understanding of welding engineering, and the inspector may be more qualified for this function. While this practice may be acceptable for typical projects that utilize common materials, more scrutiny is justified for unusual applications that utilize materials in ways that deviate significantly from normal practice. In such situations, it is advisable for the engineer to retain the services of a welding expert to evaluate the suitability of the WPSs for the specific application.

3.7 WELDING COST ANALYSIS

Welding is a labor-intensive technology. Electricity, equipment depreciation, electrodes, gases, and fluxes constitute a very small portion of the total welding cost. Therefore, the prime focus of cost control will be on reducing the amount of time required to make a weld.

The following example is given to illustrate the relative costs of material and labor, as well as to assess the effects of proper process selection. The example to be considered is the groove weld of beam flange-to-column connections. Since this is a multiple-pass weld, the most appropriate analysis method is to consider the welding cost per weight of weld metal deposited, such as dollars per pound. Other analysis methods include cost per piece, ideal for manufacturers associated with the production of identical parts on a repetitive basis, and cost per length, appropriate for single-pass welds of substantial length. The two welding processes to be considered are shielded metal arc welding and flux-cored arc welding. Either would generate high-quality welds when properly used.

To calculate the cost per weight of weld metal deposited, an equation taking the following format is used:

$$\text{Cost per mass} = \left(\frac{\text{labor and overhead rate}}{\text{Deposition rate (operating factor)}}\right) + \frac{\text{filler metal cost}}{\text{efficiency}}$$

The cost to deposit the weld metal is determined by dividing the applicable labor and overhead rate by the deposition rate, that is, the amount of weld metal deposited in a theoretical, continuous 1 hour of production. This cannot be maintained under actual conditions since welding will be interrupted by many factors, including slag removal, replacement of electrode, repositioning of the work or the welder with respect to the work, etc. To account for this time, an “operating factor” is utilized. The operating factor is defined as the arc-on time divided by the total time associated with welding activities. The following operating factors are typically used for the various processes:

Analysis of actual welding operations suggest the values shown in Table 3.2 are typically higher than actual measurements.

TABLE 3.2 Typical Operating Factors

SMAW	60%
FCAW	80%
GMAW	100%
SAW	100%

The cost of the filler metal is simply the purchase cost of the welding consumable used. Not all of this filler metal is converted directly to deposited weld metal. There are losses associated with slag, spatter, and in the case of SMAW, the stub loss (the end portion of the electrode that is discarded). To account for these differences, an electrode efficiency factor is applied. The efficiency factors shown in Table 3.3 are typically used for the various welding processes:

TABLE 3.3 Typical Electrode Efficiencies

SMAW	30%
FCAW	40%
GMAW	45%
SAW (automated)	50%

Operating factors for any given process can vary widely, depending on what a welder is required to do. In shop situations, a welder may receive tacked assemblies and be required only to weld and clean them. For field erection, the welder may “hang iron,” fit, tack, bolt, clean the joint, reposition scaffolding, and perform other activities in addition to welding. Obviously, operating factors will be significantly reduced under these conditions.

The following examples are the actual procedures used by a field erector. The labor and overhead costs do not necessarily represent actual practice. The operating factors are unrealistically high for a field erection site, but have been used to enable comparison of the relative cost of filler metals versus the labor required to deposit the weld metal, as well as the difference in cost for different processes. Once the cost per deposited pound is known, it is relatively simple to determine the quantity of weld metal required for a given project, and multiply it by the cost per weight to determine the cost of welding on the project.

In the SMAW example, the electrode cost is approximately 6 percent of the total cost. For the FCAW example, primarily due to a decrease in the labor content, the electrode cost is 25 percent of the total. By using FCAW, the total cost of welding was decreased approximately 65 percent. While the FCAW electrode costs 85 percent more than the SMAW electrode, the higher electrode efficiency reduces the increase in electrode cost to only 39 percent.

The first priority that must be maintained when selecting welding processes and procedures is the achievement of the required weld quality. For different welding methods which deliver the required quality, it is generally advantageous to utilize the method that results in higher deposition rates and higher operating factors. This will result in reduced welding time with a corresponding decrease in the total building erection cycle, which will generally translate to a direct savings for the final owner, not only lowering the cost of direct labor, but also reducing construction loan costs.

3.8 WELDING PROBLEMS: CRACKING AND TEARING DURING FABRICATION

A serious problem that may occur during fabrication or erection of steel structures is cracking. The rapid chemical and thermal changes that occur during welding may result in the formation of cracks in the weld, cracks in the base metal near the weld, or tears in the base metal near the weld. This section deals with cracking and tearing that occur during or near the time of welding.

Service failures may have similar characteristics to the cracks and tears described here, but the mechanisms and solutions are different. Service failures are caused by service loads; the cracking and tearing described in this section are caused by the shrinkage stresses that develop as the weld cools and shrinks. Service failures may occur after many years of successful operation; cracking and tearing associated with fabrication occurs within days of welding.

Cracking and tearing occur when hot, molten weld metal solidifies and cools, and volumetrically shrinks. Normally, the strains associated with shrinkage are accommodated by localized yielding, in the base metal or the weld metal. When this yielding is not possible, however, cracking can occur.

Weld cracking can take on several forms, each being caused by different phenomena. Centerline cracks occur in the center of a weld bead, parallel to the weld axis. Underbead cracks are also parallel to the weld axis, but occur in the heat affected zone, the region of the base metal adjacent to the weld that has been heated by welding and contains a microstructure that is different than the unheated base metal. Transverse cracks are in the weld metal and are oriented perpendicular to the weld axis. Finally, tearing in the base metal can occur; such tearing is parallel to the weld axis.

Cracks can be characterized as “hot cracks” or “cold cracks.” Hot cracks occur at elevated temperatures and are associated with the solidification and cooling of hot weld metal. Cold cracks occur at temperatures that are relatively cool, in metallurgical terms. Such cracking normally occurs at temperatures less than 200°C (400°F), which would normally be considered very hot; however, when dealing with electrical arcs that operate at 3300°C (6000°F) and materials that melt at 150°C (300°F), temperatures that are lower than 200°C (400°F) are relatively cool.

3.8.1 Centerline Cracking

Centerline cracking (Fig. 3.31) is a form of hot cracking that results from one of the following three phenomena: segregation-induced cracking, bead shape–induced cracking, and surface profile–induced cracking. Unfortunately, all three phenomena evidence themselves in the same type of crack, and it is often difficult to identify the cause. Moreover, experience has shown that often two or even all three of the phenomena will interact and contribute to the cracking problem. Understanding the fundamental mechanism of each of these types of centerline cracks will help in determining the corrective solutions.

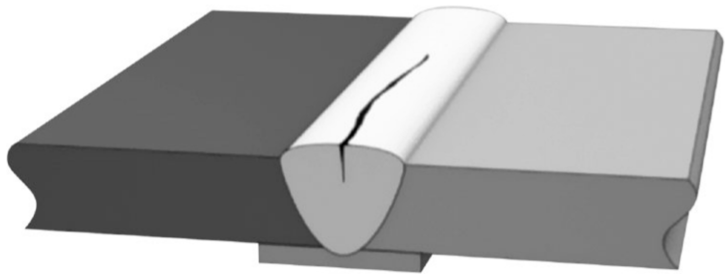

FIGURE 3.31 Centerline cracking example.

Segregation-induced cracking occurs when low melting point constituents in the admixture separate during the weld solidification process. If the steels contain higher levels of sulfur, phosphorus, lead, or copper, these elements will segregate into the center of the solidifying weld bead. Perhaps the most frequently encountered contaminant in steel is sulfur. In the presence of iron, the sulfur will combine to form iron sulfide (FeS) with a melting point of approximately 1200°C (2200°F). Steel, on the other hand, has a melting point of approximately 1540°C (2800°F). As the grains grow, FeS is forced into the center of the joint. Phosphorus, lead, and copper will act in a similar manner. The primary difference with these elements is that they do not form compounds, but are present in their basic form.

When centerline cracking induced by segregation occurs, the level of low melting point constituents must be reduced. Since the contaminant usually comes from the base material, the first consideration is to control the base metal composition. The second approach is to limit the amount of penetration into the base metal, which in turn limits the amount of low melting point material that is introduced into the weld metal. In the case of sulfur, it is possible to overcome the harmful effects of iron sulfides by preferentially forming manganese sulfide (MnS). Manganese sulfide is created when manganese is present in sufficient quantities to counteract the sulfur.

The second type of centerline cracking is known as bead shape–induced cracking. When a weld bead cross section is deeper than it is wide, the solidifying grains growing perpendicular to the steel surface intersect in the middle, but do not gain fusion across the joint. To correct for this condition, the individual weld beads must have at least as much width as depth. Recommendations vary from a 1:1 to a 1.4:1 width-to-depth ratio to remedy this condition. The total weld configuration, which may have many individual weld beads, can have an overall profile that constitutes more depth than width. Preferred weld cross sections are created when the weld joint is wide and weld beads are shallow.

The final mechanism that generates centerline cracks is surface profile conditions. When concave weld surfaces are created, internal shrinkage stresses will place the weld metal on the surface into tension and may cause the bead to crack. Conversely, when convex weld surfaces are created, the internal shrinkage forces pull the surface into compression. When concave beads crack, welding procedural changes are needed to create flat or slightly convex weld beads.

3.8.2 Underbead Cracks

Underbead cracking (Fig. 3.32) is a cold cracking phenomenon characterized by separation that occurs immediately adjacent to the weld bead, in the heat-affected zone (HAZ). Three factors contribute to this behavior: an excessive level of hydrogen, an applied or residual stress, and a sensitive HAZ. Underbead cracking occurs only at low temperatures, typically less than 200°C (400°F) and often only after the steel has returned to room temperature. Underbead cracks may be delayed, occurring 72 or more hours after welding. Typically, steel yield strengths of 480 MPa (70 ksi) are required for underbead cracks to develop.

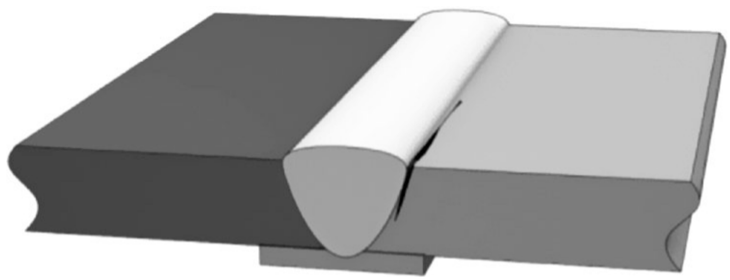

FIGURE 3.32 Underbead cracking example.

To overcome underbead cracking problems, one or more of the three contributing factors must be addressed. Hydrogen control involves the proper selection and storage of the welding electrodes and fluxes, as well as making certain that the base metals are clean and dry. The driving force behind underbead cracking is the transverse shrinkage of the weld metal. While residual stresses after welding cannot be eliminated, they can be controlled, starting with selecting filler metals of the proper strength for the application. Finally, and importantly, the HAZ sensitivity, or the hardness, of the HAZ, should be controlled. HAZ hardness is related to two factors: the base metal chemistry and the cooling rate that the HAZ experiences. Selection of base metals with lower levels of carbon and alloys reduces the hardenability of the HAZ, and mitigates underbead cracking tendencies. When higher carbon and higher alloy content steels must be welded, the HAZ hardness can be controlled by reducing the cooling rate experienced by this zone. Preheat is the major means of controlling HAZ hardness.

3.8.3 Transverse Cracks

Transverse cracking (Fig. 3.33) is another form of cold cracking, characterized by separation that occurs in the weld deposit, perpendicular to the weld axis. Three factors contribute to this behavior: an excessive level of hydrogen, an applied or residual stress, and a sensitive weld deposit. Like underbead cracking, transverse cracking occurs at lower temperatures and may be delayed. Transverse cracks may have very regular spacing, occurring at uniform intervals along the length of the weld. In general, transverse cracks are associated with weld metal tensile strengths greater than 620 MPa (90 ksi).

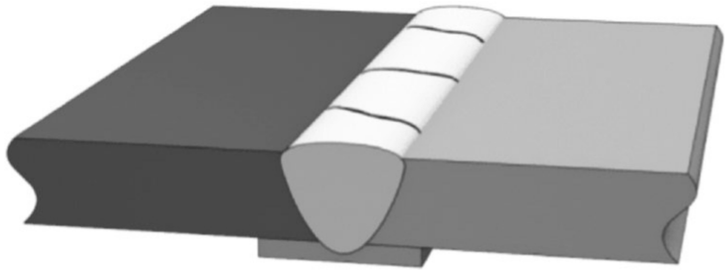

FIGURE 3.33 Transverse cracking example.

The solutions to transverse cracking are similar to those for underbead cracking: control the hydrogen content in the weld metal, control the residual stress, and control the sensitivity (hardness) of the weld deposit. In most cases of transverse cracking, the weld deposit has excessive strength, exceeding the capacity of the base metal. Thus, controlling the strength of the weld deposit is essential. Preheat is also helpful and hydrogen control is always important.

3.8.4 Lamellar Tearing

A lamellar tear (Fig. 3.34) is defined in AWS 3.0 as "a subsurface terrace and step-like crack in the base metal with a basic orientation parallel to the wrought surface caused by tensile stresses in the through-thickness direction of the base metals, weakened by the presence of small dispersed, planar-shaped, nonmetallic inclusions parallel to the weld surface." Like an underbead crack, lamellar tears do not occur in weld deposits but in the base metal. Unlike underbead cracks, lamellar tears typically occur outside the HAZ.

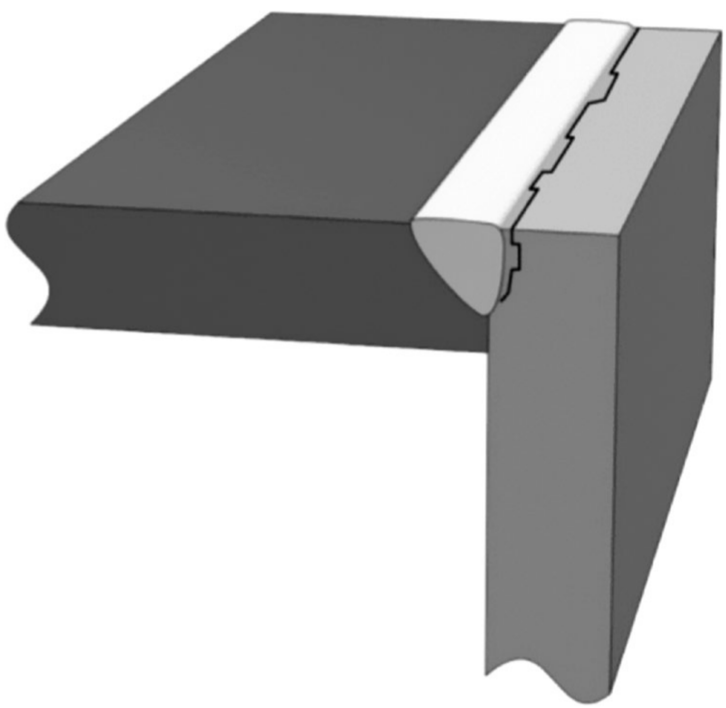

FIGURE 3.34 Lamellar tearing example.

Lamellar tearing is caused by the transverse shrinkage stresses from welding, combined with the inclusions in the through-thickness direction. Lamellar tearing tendencies are most pronounced in steels over 20 mm (¾ in) thick, and with weld deposits that are over 20 mm (¾ in) thick. Corner joints are the most susceptible to this type of tearing.

Minimizing the weld size is a first step toward overcoming lamellar tearing. The weld size must be consistent with design requirement, but larger than necessary welds introduce additional and unnecessary residual stresses. Preparing corner joints such that the bevel is applied to the material in which lamellar tearing might occur is a powerful step to mitigate this problem. Preheat and hydrogen control are also helpful. Steels with lower inclusion contents minimize lamellar tearing tendencies. Additionally, the shape of the inclusions can also be controlled (with spherical shapes being preferable to planar shapes).

3.9 WELDING PROBLEMS: DISTORTION

Distortion occurs due to the nonuniform expansion and contraction of weld metal and hot adjacent base metal during the heating and cooling cycles of the welding process. At elevated temperatures, hot, expanded weld and base metal occupies more physical space than it will at room temperatures. As the metal contracts, it induces strains resulting in stresses that are applied to the surrounding base materials. When the surrounding materials are free to move, these residual stresses cause distortion.

It should be emphasized that both the weld metal and the surrounding base material, are involved in this contraction process. For this reason, welding processes and procedures that introduce high amounts of energy into the surrounding base material will cause more distortion.

The shrinkage stresses from welding causes different forms of distortion, including angular distortion, longitudinal shrinkage, transverse shrinkage, longitudinal sweep or camber, panel distortion and rotational distortion, as illustrated in Fig. 3.35. Some forms are caused by transverse shrinkage, while others result from longitudinal shrinkage.

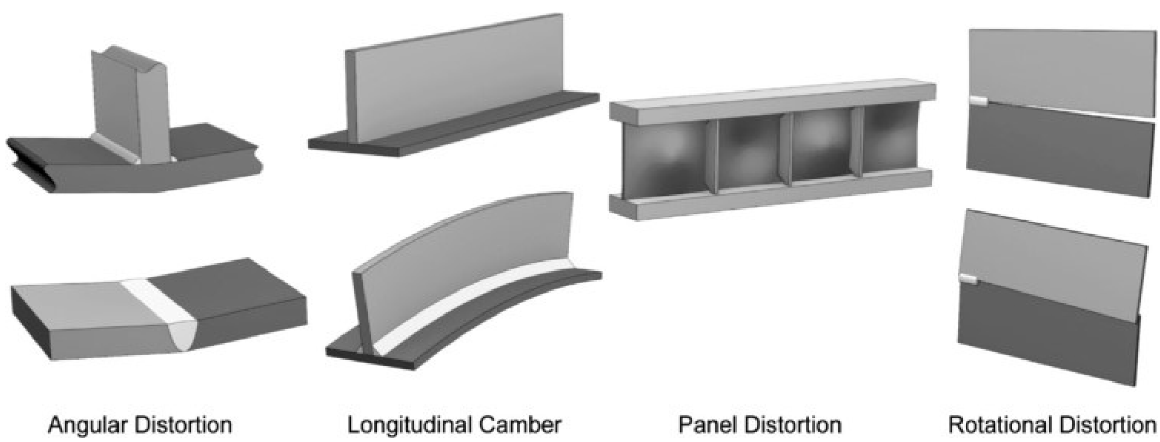

FIGURE 3.35 Distortion examples.

Stresses resulting from material shrinkage are inevitable in all welding that involves the application of heat. Distortion, however, can be minimized, compensated for, and predicted. Through efficient planning, design, and fabrication practices, distortion related problems can be minimized. Distortion control principles fit into two categories: those that should be considered in the design and detailing of the connection, and those associated with fabrication and erection.

The following principles should be incorporated into the connection design to minimize distortion:

- Use the smallest acceptable weld size (consistent with design requirements).
- Use weld details that minimize the amount of weld metal for a given weld size.
- To control longitudinal sweep or camber, place the weld on the neutral axis, or balance the shrinkage stresses about the neutral axis.

The following principles can be applied by the contractor to minimize distortion:

- Control over welding.
- Control fit up tolerances.
- For a given weld size, make the weld in the fewest number of passes.
- For a given weld size, make it with the least heat input.
- Use fixtures, clamps, strong-backs, and other restraints to resist the shrinkage forces.
- Use copper heat sinks to draw the heat away.

While distortion cannot be eliminated, the proper application of these principles will typically bring distortion into acceptable limits.

3.10 WELDING ON EXISTING STRUCTURES

Welding on existing structures may be necessary for a variety of reasons, but it generally falls into one of two categories: modifications to the structure to reconfigure the structure for different purposes and repairs to structures to correct for damage. Modifications may be simple additions to existing structures or may involve strengthening the structure to add load-carrying capacity. Repairs may be required due to the effects of overloading caused by natural events such as tornados, earthquakes, or extreme snow events. In other cases, repairs may be necessary to correct for fire damage or to replace corroded material.

AWS D1.1 clause 8 deals with the topic of strengthening and repair of existing structures and requires the engineer to "…prepare a comprehensive plan for the work. Such plans shall include, but are not limited to, design, workmanship, inspection, and documentation." The commentary provides helpful guidance. Welding-related concerns are addressed in a general way in this section. AWS D1.7 *Guide for Strengthening and Repair of Existing Structures* was developed to assist those associated with such projects by providing guidance, but not code requirements (AWS, 2010b).

Cooperative interaction between the engineer and contractor is critical on strengthening and repair projects. Unusual situations that are not anticipated during the planning stages are often encountered once the project begins. Budgets and schedules should anticipate that the unanticipated will likely arise. Historic drawings may be missing or inaccurate and the structure may not have been built as designed. Undocumented modifications may be discovered and other anomalies may arise. Recognitions of these realities and a cooperative interaction of all the parties involved are critical when such situation arise.

3.10.1 Safety Precautions

Welding on existing structures may involve potential welding hazards distinct from those associated with new construction projects. In addition to the routine measures needed to provide a safe workplace, the following issues should also be considered when working on existing structures.

Structural Stability. In a manner similar to the erection of a new building, and overall plan must be established when work is to be performed on existing structures. This is particularly important when existing members are cut or removed. For severely deformed structures, considerable stored elastic energy may be present in the existing members.

Fire and Explosions. Existing structures are often filled with combustible materials as well as pipes that may contain natural gas or other combustible fluids. The sparks from cutting torches and welding operations have created fires on many occasions. Preheating torches are another source of potential problems. Appropriate precautions must be taken to control the potential fire hazards.

The welding work lead circuit is another source of potential fire creation. It is simple and convenient for a welder to attach the work lead to a building frame member, perhaps hundreds of feet away from the point where welding is being performed. The welding current must pass through the structure and may take some unanticipated paths, such as through sheet metal duct work, electrical conduit, etc. At a point of high electrical resistance, localized heating of the portion of the welding circuit can cause a fire, one that is deeply hidden in the existing structure and away from the welding operations. To overcome this problem, the welding work lead should be attached as close as possible to the point where welding is to be performed.

Ventilation. The erection of a new steel frame is usually performed under conditions where the welding operations are exposed to the elements. Ventilation is rarely a concern for field welding operations. When welding on existing structures, the opposite condition is often experienced: ventilation may be inadequate. Special smoke removal equipment and air handling devices may be needed to provide a safe working environment.

Asbestos, Lead-Based Paint and Other Hazardous Material. Rehabilitation projects may involve fireproofing removal, and older fireproofing may contain asbestos. Older structures may have been painted with lead-based paint. Other hazardous material may be encountered during rehabilitation projects and special measures may be required to protect workers from exposure.

3.10.2 Existing Steel Composition and Condition

Before welding on existing structures, the steel should be investigated with respect to any potential welding problems, particularly when the structure involved is riveted.

A check of the chemical composition of one piece of steel in one location cannot be taken as representative of all the steel throughout the structure. Multiple heats of steel were probably used, and the steel might have come from multiple suppliers. Moreover, multiple grades of steel may have been used.

Steel that was welded in the past is equally weldable today. The primary weldability challenge comes when the existing steel was not welded upon. Steel that was originally joined by rivets or bolts should not be automatically considered to have poor weldability; rather, it should be viewed as material without established weldability. Many existing riveted structure have been successfully modified using normal welding practices, indicating that the steel had a chemical composition suitable for welding, even though the process was not used for the original fabrication or erection.

The condition of the steel also deserves special mention with a specific focus on corrosion. Severe section loss due to rusting can create a variety of welding challenges. Heavy rust will directly affect the weld quality and the ability to obtain fusion to the steel. Thin sections may encourage melt-though during welding. The pockets formed when rust is removed might result in excessive gaps and fit-up challenges.

For repairs to structures that have been damaged by fire, the steel should be examined to see if the heated steel was damaged in the fire. The damage may come from the heated steel being rapidly cooled by water used to extinguish the fire, hardening the steel. In other cases, steel that was heated may have cooled slowly, resulting in softening as compared to the original properties. Under fire conditions, steel members may buckle or deflect, creating serviceability problems.

3.10.3 Welding and Cutting on Members under Load

Before any work is performed, particularly if cutting is involved, the loading condition on the structure must be examined, considering both dead and live loads. Although it is often impractical, it is always desirable to remove as much load as possible before work begins. Shoring may be necessary. Thermal cutting on loaded members, particularly in tension regions must be done with great caution; steel members have fractured during such operations. Shoring as a precaution against the unexpected is advisable, particularly when redundant load paths are not certain.

As steel is heated, it loses strength and stiffness, and thus reasonable concerns have been raised regarding how welding will affect structures under load. Two important factors reduce the actual effect of such heating from welding. First, at temperatures up to approximately 340°C (650°F), the reduction in strength and stiffness is negligible (Blodgett, 1966). Secondly, at any given time, only a very small portion of the cross section of the structural element experiences the reduced properties (Tide, 1987).

The orientation of the weld with respect to the stress field is a factor, but rarely a controlling one. When welds are deposited parallel to the stress field, it is only the weld cross-sectional area and a small portion of the surrounding steel that experiences the reduced strength due to the elevated temperature. When welds are perpendicular to the stress field, the area of reduced strength and stiffness is the height of the weld bead plus a small portion of the surrounding steel times the length of the weld that is hot. This length includes the weld pool (which is typically 1.5 to 3 times as long as it is wide), and some length beyond the weld pool. The greater amount of heated metal in this case has prompted the general rule-of-thumb preference for longitudinal welds versus transverse welds when welding on members under load. However, the actual impact of such differences is typically inconsequential, but this should be checked for the application involved (Ricker, 1987; Tide, 1987).

3.10.4 Modifications and Additions to Undamaged Steel

For this section, the base assumption is that the steel is undamaged: it is free of cracks and has not been plastically deformed. In some ways, modification and additions are similar to adding an additional tier on top of a partially erected building, but in other ways, the work is quite different: the frame is normally more rigid, under load, and has undergone the natural settling that occurs during the life of a structure. Bolted connections, if used, may have slipped and loads have caused beams to deflect. Concrete slabs are likely in place. For all these reasons, an existing frame typically is more restrained than a similar frame being constructed. The extra rigidity may result in more cracking tendencies.

3.10.5 Repair of Plastically Deformed Steel

Steel that has been previously subjected to inelastic deformations (such as due to overload conditions or seismic events) may require welding. When steel is strained in the 5 to 18 percent range, yields can increase 70 percent, tensile strengths increase 35 percent, elongation decrease 30 percent and the CVN transition temperature increase by 65°F (Pense, 2004).

In addition to effects of cold working, welding will compound the effects through a phenomenon called strain aging. When the deformed steel is heated into the range of 260 to 430°C (500 to 800°F) (as will happen during welding), the CVN toughness decrease further, typically increasing the CVN transition temperature by an additional 35°F (Pense, 2004; Stout, 1987). This same region of locally reduced notch toughness is also the region that will be strained as the weld shrinks leaving residual stresses at the yield point. Any small notch-like discontinuity in this area can serve to initiate fracture. Fortunately, this zone is in the base metal, away from the weld, and weld discontinuities will not form in this area. However, discontinuities in the fusion zone are adjacent to the cold worked region. Since any cracking from strain aging occurs adjacent to the weld, it has many of the same characteristics of underbead cracking, which is hydrogen driven. Despite appearance similarities, the mechanisms seem to be different. The nitrogen content of the steel, and specifically the “free nitrogen” (i.e., the nitrogen that is not chemically combined with other elements, such as aluminum) is a chief contributor to strain aging. Steels that are not fully killed with aluminum are particularly sensitive to strain aging. This would include rimmed steels, semi-killed steels, and silicon-killed steels, which are not commonly produced today but are present in many existing structures. Continuous casting of steel made with electric furnaces requires the use of aluminum in steel, and thus, much of today’s steel production is more resistant to strain aging (Bailey, 1994).

Thermal stress relief can help the steel recover from some of the harmful effects of strain, provided that the steel does not experience reheat cracking during stress relief. Reheat cracking is a form of cracking that occurs when steel contains at least two of the following elements: Cr, Mo, V, and B (Bailey, 1994). Unfortunately, these elements are present in many structural steels used today. As an alternative to traditional stress relief, a full normalizing heat treatment that completely reverses the effects of cold working may be applied and eliminate strain aging concerns altogether.

From a practical perspective, existing structures with plastically deformed steel will rarely be stress relieved or normalized. When welding on severely deformed steel results in repeated underbead cracking, and provided hydrogen has been dismissed as a contributing factor, such material may need to be removed and a new piece of steel inserted and welded in place.

3.11 WELDING ON SEISMICALLY RESISTANT STRUCTURES

In high-seismic applications (when the seismic response modification factor R is taken greater than 3), the requirements in the building code differ from other loading conditions in that it is assumed that portions of the building's seismic force resisting system (SFRS) will undergo controlled inelastic response when subjected to major seismic events. Welds and welded connections that are part of the SFRS connect members that are subject to yield-level stresses and plastic deformations during such events. In order to resist the imposed loads, welded connections must be designed, detailed, fabricated, and inspected to more rigorous standards than are required for statically loaded buildings. The weld metal property requirements are also different.

This section provides a general overview of typical requirements, but is not intended to be a comprehensive summary of all the provisions of various seismic standards, nor should it be used as a replacement for these other documents. As is the case elsewhere in this chapter, the focus is primarily devoted to welding-related provisions. This handbook contains a chapter devoted to the design-related aspects of structures designed to resist seismically applied loads.

3.11.1 High Connection Demands

High-seismic framing systems generally have the highest demands concentrated at the ends of beams and braces, right near the point of the connections. Thus, connections are often in or near the most severely stressed portions of a structure. Inelastic deformations are not typically expected to be concentrated in the welds themselves, but welds are often near the base metal in which such strains are located. In order for the expected inelastic deformations to occur, the welded connections must be strong enough to resist the applied stresses without fracture, and the base metal must be capable of deforming to accommodate the straining.

The welded connections in high-seismic applications must be strong, ductile, and fracture-resistant. Strength and ductility are primarily addressed through the selection of the welding filler metals and control of the procedures used to deposit the metal. Such criteria are not significantly different than the requirements for low-seismic applications.

Three factors determine the ability of a connection to resist brittle fracture: the applied stresses, the presence (or lack) of cracks, notches and other stress concentrations, and the fracture toughness of the material. The applied stresses in the connection are inherently linked to the configuration of the connection. In general terms, two approaches have been used in seismic design to reduce the applied stresses in the connection: the connection can be strengthened (by the use of reinforcing ribs, gussets, coverplates, etc.), or the demand on the connection can be reduced (such as through the use of reduced beam sections, often called “dogbones”). These factors are not directly weld related, but have a direct effect on the localized stresses in the weld and ductility demands on the weld.

The other two factors (stress concentrations and material fracture toughness) are specifically welding related. The first variable consists of two different issues: cracks and stress concentrations. For connection fracture resistance, welds and HAZs must be free of cracks and crack-like discontinuities; that is, planar and near-planar flaws. To avoid cracks, specifications like the AWS D1.8 Structural Welding Code—Seismic Supplement emphasize hydrogen control. The AISC Seismic Provisions call for specific postwelding nondestructive testing (NDT) to detect any cracking that might have occurred during or after welding. Lamellar tearing can be similarly detected. Incomplete fusion, some slag inclusions, and planar discontinuities, may have a crack-like effect on fracture resistance. Good welding procedures and welder workmanship limit the production of such discontinuities, and effective NDT is used to detect remaining planar flaws.

3.11.2 Stress Concentrations

Stress concentrations occur in a variety of forms, including notches and gouges from flame cutting, weld toes, left-in-place weld tabs, and weld discontinuities such as undercut, underfill, and porosity. Since the demand on seismically loaded connection is so high, it is important to minimize the number and severity of stress concentrations. The AISC 341 Seismic Provisions and the AISC 358 Prequalified Connection Standard, as well as AWS D1.8, prescribe limits for such stress concentrations in the connections of structures subject to seismic loading.

Steel backing left in-place in T joints of moment connections can create a crack-like planar discontinuity that constitutes a major stress concentration. Illustrated in Fig. 3.36, the stress concentration is created by the naturally occurring lack-of-fusion plane between the vertical edge of the steel backing and the column flange. Additionally, this is a likely site of various welding discontinuities, such as incomplete fusion and slag inclusions.

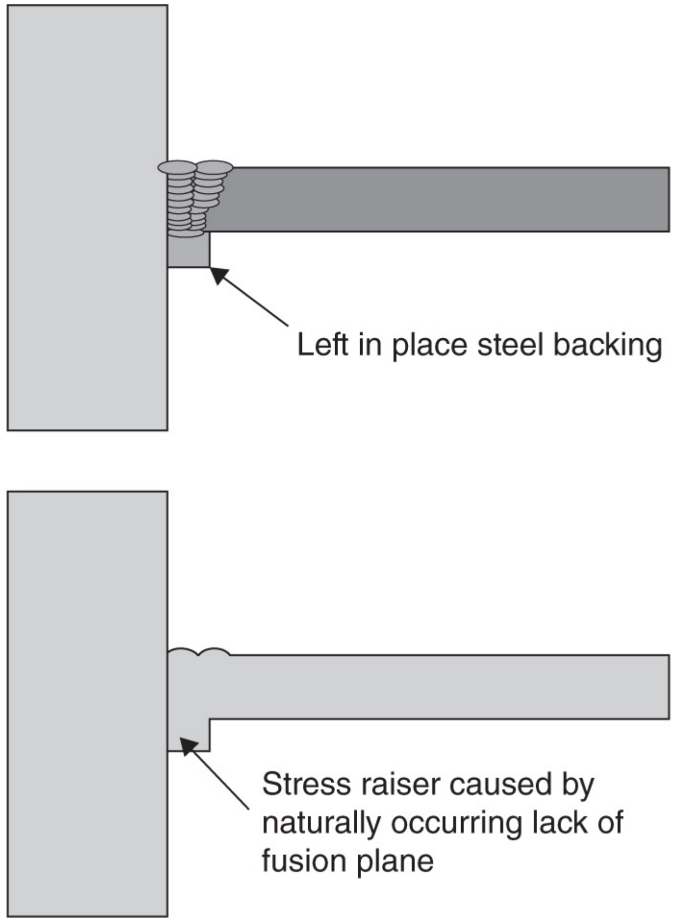

FIGURE 3.36 Left in place steel backing.

3.11.3 Fracture Resistance

In addition to the stress level and the presence of cracks and stress raisers, the fracture toughness of the materials involved affects the fracture resistance of the connection. The materials of interest include the base metal, weld metal, and the HAZ. Tests performed on base metals suggest that commercially supplied rolled shapes routinely exhibit sufficient fracture toughness to avoid the specification of special requirements (Frank, 1997), except for heavier rolled shapes and thicker plates. Similarly, when welding heat input is constrained within normal fabrication limits, no special controls have been found necessary for HAZ fracture toughness control (Johnson, 1997). For weld metal, fracture toughness requirements (in the form of minimum CVN toughness values) have been developed, and specifics are discussed below (Barsom, 2003).

3.11.4 Demand Critical Connections and Protected Zones

Connections in the SFRS that are subject to such severe loading conditions and those joints whose failure would result in significant degradation in the strength and stiffness of the SFRS have been identified in the AISC and AWS standards as “demand critical.” Welds in demand-critical connections are called “demand-critical welds” and are subject to additional detailing provisions, material requirements, workmanship and fabrication standards, and inspection provisions.

The material in the area wherein plastic hinges are intended to form must be relatively smooth and free of notches, gouges, tack welds, shear studs and other geometric changes that might concentrate stress or inhibit ductile behavior. To ensure that ductility is not impaired in this region by inadvertent attachments, for example, the term "protected zone" has been created and defined. In this region, restrictions on attachments and fabrication practices apply. The AISC Seismic Provisions and AISC Prequalified Connection Standard define the region, and AWS D1.8 further specifies operations that are prohibited in this zone (AISC, 2016a; AISC, 2011; AWS, 2009).

3.11.5 Seismic Welded Connection Details

The permissible welded connection details for seismic applications will depend on the particular connection type and the contract documents. For moment frames, the engineer will either select a prequalified connection detail from AISC 358 (the typical case), or will need to have connection tested in accordance with the protocol as outlined in AISC 341. Welded details can vary, but general practices for welded connection details are describe in the next sections.

Steel Weld Backing. Steel backing may create notch effects in the weld root, depending on the joint type and loading conditions (Fig. 3.36). When used in tee joints typical of beam-to-column connections in moment frame buildings, and particularly for the bottom beam flange connection, lateral forces will cause bending moments, which impose tensile stresses on these connections. The notch-like condition created by the left-in-place backing in T-joints can serve as a stress concentrator and crack initiator.

To eliminate this condition, the steel backing can be removed, the root of the weld gouged to sound metal, and a reinforcing fillet weld applied (Fig. 3.37). This is an expensive operation that is typically applied only to the bottom beam-flange to column-flange connection in special moment resisting frames.

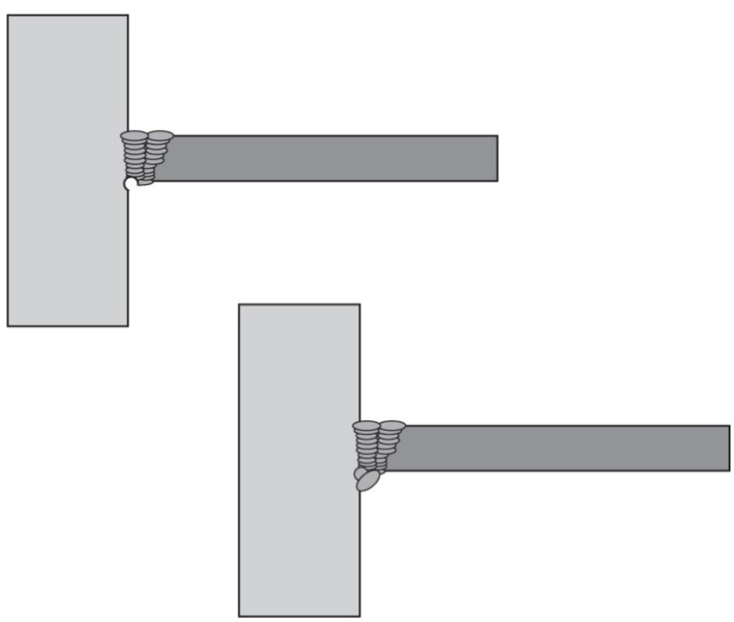

FIGURE 3.37 Backgouged steel backing with a rewelded fillet joint.

For top beam-flange to column-flange welds, a simple reinforcing fillet weld, added between the backing and column, sufficiently reduces the stress concentration, so as to permit steel backing to remain in place (Fig. 3.38). Welds should not be placed between the backing and the beam flange, as such connections actually increase the amount of stress transferred into the backing, and increase the notch effect of the unfused backing.

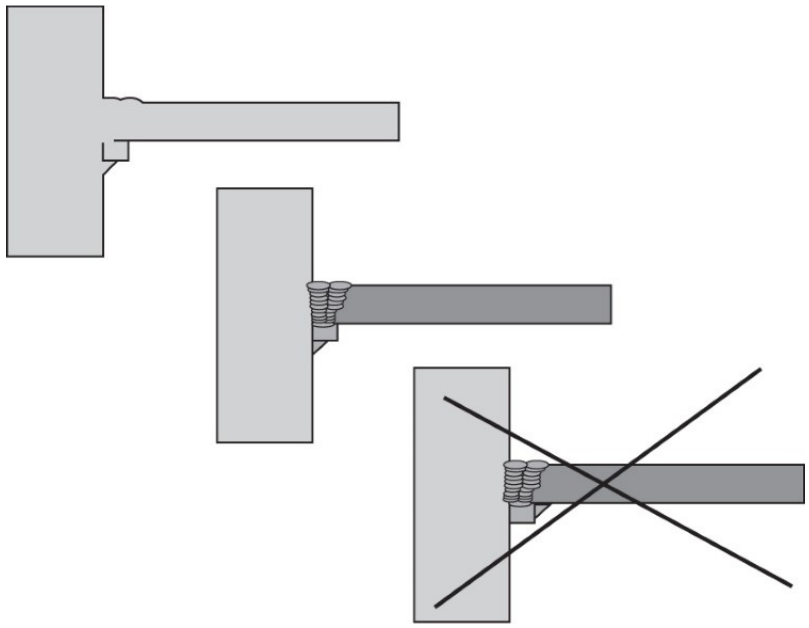

FIGURE 3.38 Proper and improper backing bar reinforcing weld.

While left-in-place backing on connections, such as that described above, create undesirable stress concentrations, this is not automatically the case whenever and wherever backing is left in place. In a butt joint, for example, the unfused region between the base metal and the backing lies parallel to the direction of loading, and does not constitute the same type of stress concentration as previously discussed (Fig. 3.39). Accordingly, steel backing may be left in place in certain locations on members in the SFRS.

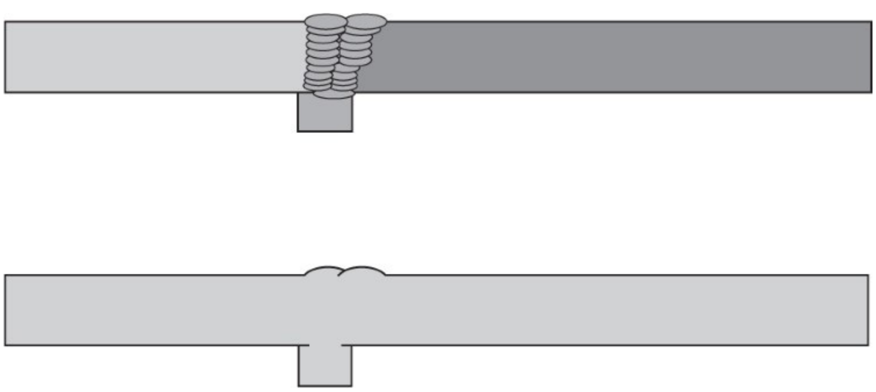

FIGURE 3.39 Backing bar left in place on but welds loaded parallel with the direction of welding.

Weld Access Holes. In moment frames in high-seismic applications, the distribution of stresses through the end of a beam into the connection is affected by the size and the nature of the weld access hole. The AISC Prequalified Connection Standard has special requirements for weld access hole geometries in some situations. In addition, weld access holes are required to be fabricated free of unacceptable notches and gouges that may serve as stress concentrators. Specific workmanship standards for weld access holes are given in AWS D1.8.

Weld Tabs. Weld tabs are normally left in place for building construction, but for buildings subject to high-seismic loading, weld tab removal may be required. The portions of a weld that are located on tabs are typically not inspected and may contain a host of discontinuities. Removing the weld tabs after the weld has solidified and cooled eliminates any potential harmful effects such discontinuities may have on connection behavior. Removal is even more important in situations where stresses are attracted toward the weld tabs, such as when wide-flange shapes are used for beams and are connected to box columns.

3.11.6 Filler Metal Requirements

For demand-critical welds, the deposited weld metal is required to demonstrate a minimum Charpy V-notch (CVN) toughness of 54 J (40 ft-lb) at +21°C (+70°F), and additionally, show 27 J (20 ft-lb) at –18°C (0°F), depending on the standard. These criteria were developed based upon loading conditions, connection details, workmanship standards and inspection requirements (Barsom, 2003). The lower temperature acceptance criterion is based upon the AWS A5 filler metal classification tests. The +21°C (+70°F), CVN toughness must be demonstrated on two test plates, one welded with the highest heat input to be used in fabrication or erection, and one welded with the lowest heat input. The high heat input test replicates slow cooling conditions, while the low heat input generates high cooling rates. For CVN toughness, optimal results are obtained with a moderate cooling rate, and values decrease as cooling rates both increase and decrease as compared to the moderate rate.

These criteria are applicable to structures with enclosed structural elements, assumed to be maintained at a temperature above +10°C (+50°F), despite external ambient conditions. For situations where this is not the case, alternate criteria must be employed, requiring testing of welds at lower temperatures.

The required weld metal fracture toughness was based upon connection details that were free of large crack-like discontinuities such as those created by left-in-place steel backing, fabrication induced cracks, and workmanship defects. High toughness values will not ensure adequate structural performance when stresses are too high, when members are highly constrained, or when severe geometric stress raisers exist (Barsom, 2003). The CVN toughness criteria outlined above, and as contained in the AISC and AWS standards, presume that other portions of these standards are being applied to the design, detailing, fabrication, and inspection of connections.

3.11.7 Welder Qualification Tests

When joining wide-flange beams to columns with groove welds, typically the welder makes the bottom-flange weld by welding through a weld-access hole. This difficult welding situation requires that welds be interrupted along their length since the web precludes a full-length weld pass. Thus, a series of weld pass starts and stops will be contained near the mid-length of the weld, under the web.

To ensure that welders are capable of making such welds, specific welder qualification tests are contained in AWS D1.8, generally replicating the geometry of bottom beam-to-column connections, and specifically designed to test the integrity of the weld in the region of the simulated web. The welder is required to weld through a minimum sized weld-access hole, using the maximum welding deposition rate, and the type of backing that will be used on the actual application (including the option of using no backing). The test simulates a T joint, and restricts access similar to actual bottom beam flange-to-column flange connections. Requiring welders to demonstrate their skills on such connection mock-ups helps to ensure that workmanship on the final structure will meet the special demands of welding on structures subject to seismic loading.

3.11.8 Nondestructive Testing

Nondestructive testing (NDT) of the completed connection serves as a final validation that the required weld integrity has been achieved. There are a variety of NDT processes, each with unique capabilities and limitations. The project quality assurance plan (QAP) specifies the details of NDT. Included are the definitions of who performs what testing, by what process, and the applicable acceptance criteria. AWS D1.8 details the NDT technician qualifications, testing protocols, and other inspection techniques.

3.12 ACKNOWLEDGMENTS

Substantial portions of this chapter have been taken from other publications by the same author(s). Portions of this chapter were taken from AISC Design Guide 21, *Welded Connections—A Primer for Engineers*, first edition, as well as from the draft of the yet-to-be-printed second edition. AISC's contributions are recognized and appreciated. Also, portions of this chapter were taken from the McGraw-Hill publication *Manufacturing Engineer's Handbook*, second edition. McGraw-Hill's contributions are also recognized and appreciated. The American Welding Society provided permission for reproduction of various prequalified joint details as contained in D1.1; AWS's contributions are recognized and appreciated. The authors' employer, The Lincoln Electric Company, is also recognized for its support in the preparation of this chapter.

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CHAPTER 4

PARTIALLY RESTRAINED CONNECTIONS

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4.1 INTRODUCTION

The AISC Specification has recognized semirigid (Type 3) or partially restrained (PR) construction since the 1940s (AISC, 1947, 2016a). Because the design of Type 3/PR connections is predicated on a set of forces obtained from an advanced structural analysis that includes the connection deformation characteristic and because few if any design texts address this issue, this chapter will begin with an introductory discussion of PR connection and its effect on frame behavior. Once these issues are understood, the connection design can proceed as for any other steel connection. For more detailed discussions of modeling and analysis issues for PR frames, the reader is referred to several excellent publications (Chan and Chui, 2000; Faella et al., 2000; Chen et al., 2011).

After the discussions on PR frame design, examples for several types of PR connections, including T stubs and flange plate connections, are presented. Design of these connections for wind loads is straightforward, as this is only a matter of strength, and Examples 4.1 and 4.3 cover this case. Design for seismic loads is more complex, as both the ductility and energy dissipation of the connection also need to be considered. A large amount of research on PR bolted connections has been carried out after the 1994 Northridge earthquake, leading to the development of detailed design procedures for the use of these connections in areas of high seismicity (FEMA, 1997, 2000). When properly designed, these connections exhibit excellent ductility and energy dissipation capacity, distributing the deformation between ductile mechanisms in both the beam and the connection (Fig. 4.1). The seismic design examples presented in this chapter have been updated to reflect the most recent AISC 341 provisions (AISC, 2016b), as well as AISC 358 (AISC, 2016c), which now includes three types of prequalified PR connections (T stub, Simpson Strong-Tie, and bolted flange moment connections).

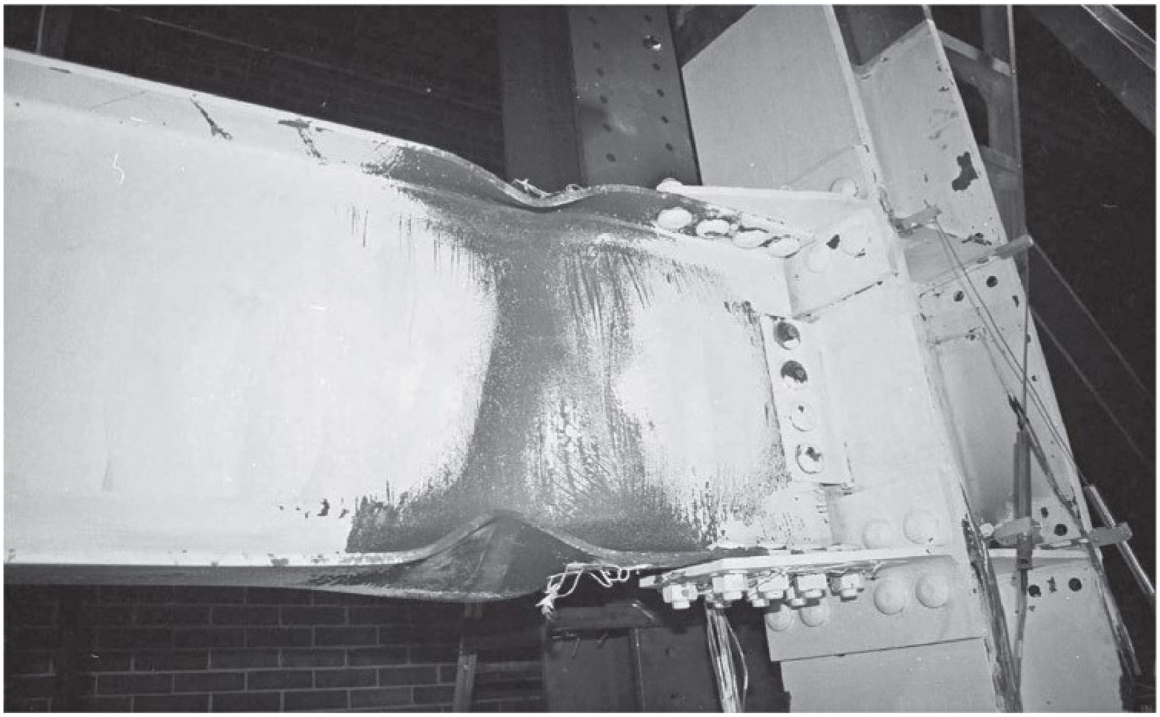

FIGURE 4.1 Cyclic performance of T-stub connection.

Three significant changes that have occurred since the previous edition of this chapter was published (2010) must be highlighted. The first change is the widespread availability of robust software tools that allow engineers to more rationally and efficiently model PR connections, as most advanced structural analysis programs now contain nonlinear rotational springs capable of modeling connection behavior according to ASCE 7 and 41 (ASCE, 2016a, b) or similar principles. The second change is the publication (Chen et al., 2011) of a very complete catalogue of moment-rotation curves based on both experimental data and advanced analyses. Although extreme care must still be taken when utilizing those curves in design, they provide a strong starting point for designers unfamiliar with PR connection modeling. The third change is that the use of bolted connections will be bolstered by the recognition in AISC 360-16 of a new higher strength type of bolts. In addition to the usual bolts, now renamed Grades A (F3125 A325M, F_{nt} = 620 MPa) and B (F3125 A490M, F_{nt} = 780 MPa), a new Grade C (F3043/F3111, F_{nt} = 1035 MPa) has been added. The additional strength of Grade C bolts means that the same bolted connections can carry 33% more tension and shear than if Grade B bolts are used. These changes have made the use of PR an attractive alternative for the design of low-rise frames.

It should be noted that the examples shown in this chapter deal with connection behavior without explicitly treating the effect of the floor diaphragm. PR connections may benefit significantly from using reinforcing bars in the floor slabs to carry negative moments over the supports and to redistribute forces in the connection region. The design of this type of composite PR connection has been covered in detail in several publications (Leon, 1996, 1997), and a short summary of the topic is given at the end of the chapter.

4.2 CONNECTION CLASSIFICATION

Moment-rotation (M-θ) curves are generally assumed to be the best characterization of connection behavior for design purposes. These M-θ curves are generally derived from experiments on cantilever-type specimens (Fig. 4.2*a*). The moments (M) are calculated directly from the statics of the specimen, while the rotations (θ) are measured over a distance typically equal to the beam depth. The rotation reported thus includes all elastic and inelastic deformation components occurring in the joint region, including a portion of the beam; this fact is important when modelling the actual beam and connection lengths. For the case of a top-and-seat angles shown in Fig. 4.2, these components include, among others deformation mechanisms for the top flange, the elastic deformations due to the pullout of the angle, the rotation due to yield line formation in the leg bolted to the column due to bending, yielding of the angle leg attached to the beam in tension, slip of the bolts, and hole elongation due to bearing (Fig. 4.2*b*).

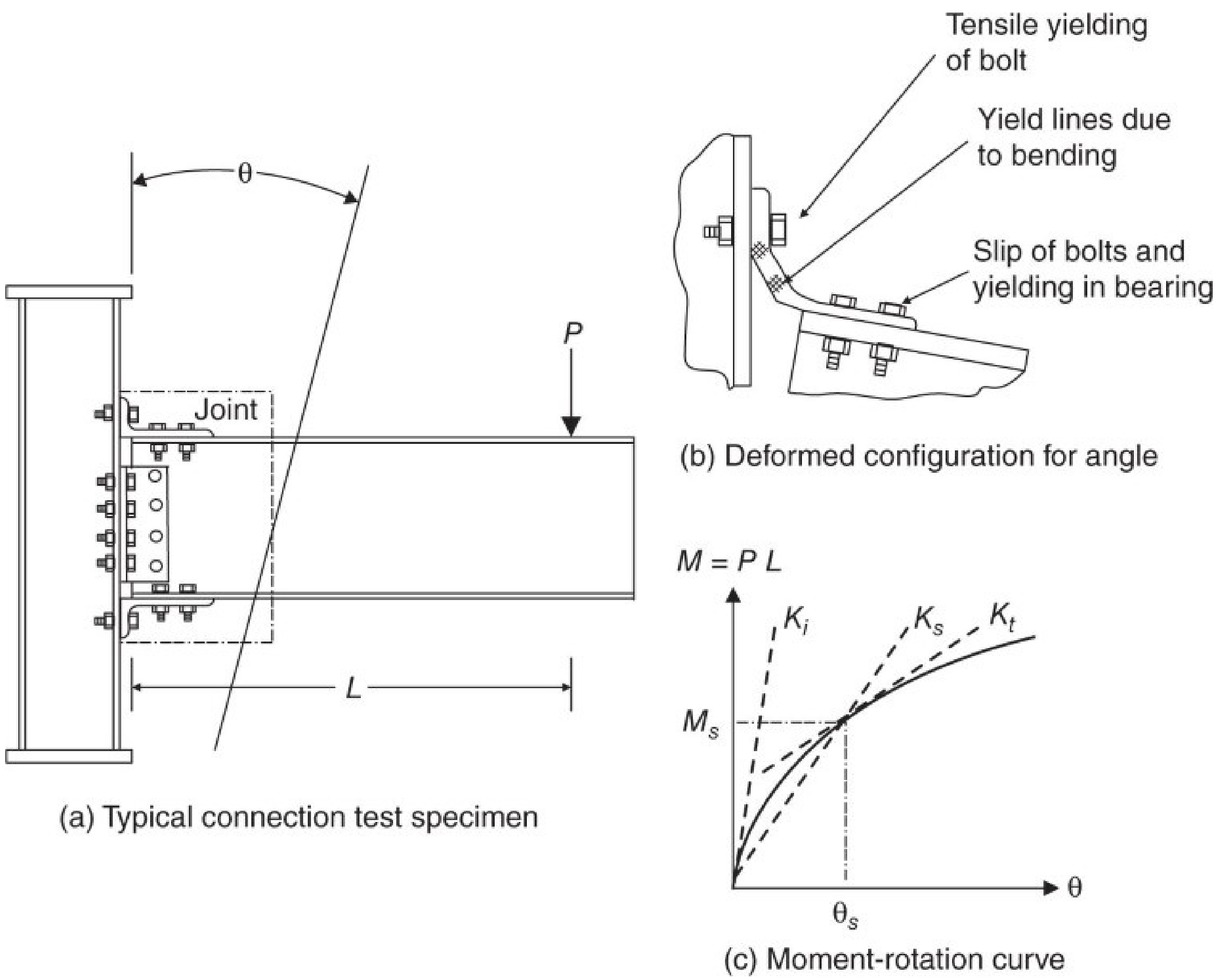

FIGURE 4.2 Derivation of M-θ curves from experiments.

From the fifth to the eighth edition of the AISC allowable stress specification (AISC, 1947, 1978), PR connections were categorized as Type 3 construction. Type 3 design was predicated on the assumption that “connections of beams and girders possess a dependable and known moment capacity intermediate in degree between the rigidity of Type 1 (rigid) and the flexibility of Type 2 (simple).” This definition was confusing since it mixed strength and stiffness concepts, and was generally interpreted as referring to the initial stiffness (K_i) of the connection as characterized by the slope of its moment-rotation curve (Fig. 4.2*c*), as opposed to either a secant (K_s, taken to a prescribed θ_s or M_s) or tangent (K_t) stiffness as would be more commonly used today. Moreover, these specifications allowed the use of PR connections in "wind frames" under the Type 2 (simple framing) classification, where the connections were assumed as simple for gravity loads and rigid for lateral loads. Until the early 1980s, many steel frames were designed using PR connections through this artifice, which has disappeared from the most recent specifications. While extensive research (Ackroyd and Gerstle, 1982) has shown this procedure to be generally safe, the final forces and deformations computed from this simplified analysis can be significantly different from those using an advanced analysis program that incorporates the entire nonlinear M-θ relationship shown in Fig. 4.2*c*.

The description of Type 3 construction used in previous versions of the steel specification could not properly account for the effect of connection flexibility at the serviceability, ultimate strength, or stability limit states. The first LRFD specification (AISC, 1986) recognized these limitations and changed the types of construction to fully restrained (FR) and partially restrained (PR) to more realistically recognize the effects of the connection flexibility on frame performance. The definition of PR connections in the first two LRFD versions of the specification (AISC, 1986), however, conformed to that used for Type 3 in previous ASD versions. Research PR connection behavior has led to more comprehensive proposals for connection classification (Gerstle, 1985; Nethercot, 1985; Bjorhovde et al., 1990; Eurocode 3, 1992, to name but a few of the earlier ones) that clarify the combined importance of stiffness, strength and ductility in connection design. Currently, AISC defines PR moment connections as connections that transfer moments, but where "the rotation between connected members is not negligible. In the analysis of the structure, the force-deformation response characteristics of the connection shall be included. The response characteristics of a PR connection shall be documented in the technical literature or established by analytical or experimental means. The component elements of a PR connection shall have sufficient strength, stiffness and deformation capacity at the strength limit states." The commentary to the three most recent editions of the unified AISC Specifications contains more detailed discussion on connection classification schemes. The discussion here, which remains consistent with that in the previous edition of this book, is in substantial agreement with the main concepts that appear in those commentaries but remains primarily the author's view. In that approach three characteristics must be separately recognized: stiffness (FR, PR, or simple), strength (full or partial strength) and ductility (brittle or ductile) as shown schematically in Fig. 4.3.

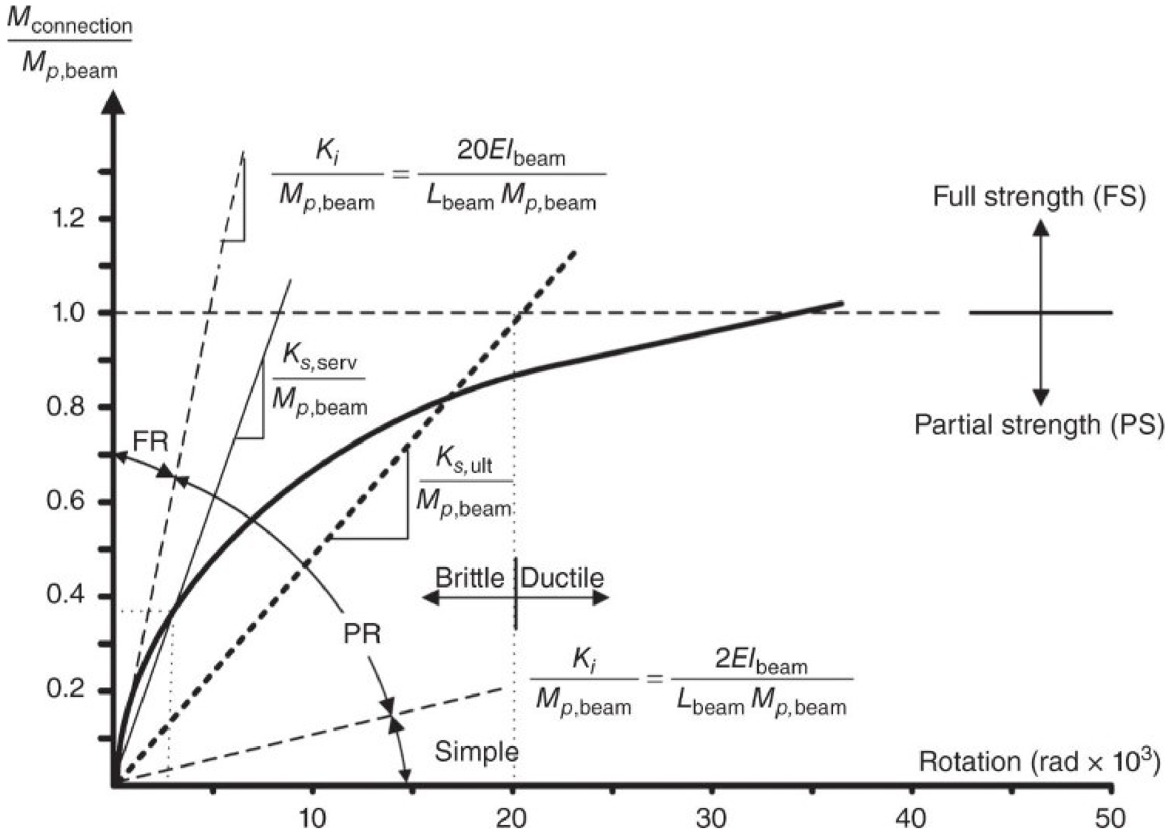

FIGURE 4.3 Classification by stiffness, strength, and ductility for nonseismic.

4.2.1 Connection Stiffness

As noted earlier, the connection stiffness is generally taken as the slope of the M-θ curve. Since the curves are nonlinear from the start, it is possible to define this stiffness based on the tangent approach (such as for K_i or K_t in Fig. 4.2*c*) or on a secant approach (such as $K_{s,\text{serv}}$ or $K_{s,\text{ult}}$ in Fig. 4.3). A tangent approach is viable only if the analysis programs available can handle a continuous nonlinear or multilinear rotational spring. Even in this case, however, the computational overhead can be large and this option is recommended only for verification of performance, rather than initial design. In most designs for regular frames, a secant approach will probably yield a reasonable initial solution at a fraction of the calculation effort required by the tangent approach. In this case, the analysis can be carried out in two steps by using linear springs. For deflections and drift checks, a service secant stiffness ($K_{s,\text{serv}}$) can be taken at 0.0025 rad; for ultimate strength checks, an ultimate secant stiffness ($K_{s,\text{ult}}$) can be taken at 0.02 rad. Clearly, the deformations computed for the service load level will be fairly accurate, since the deviation of $K_{s,\text{serv}}$ from the true curve is typically small. On the other hand, the deformations computed for the ultimate strength case will probably not be very accurate, since there can be very large deviations and the linear spring $K_{s,\text{ult}}$ can only be interpreted as an average. However, this approximation is probably sufficient for design purposes. Designers should be conscious that there is no theoretical proof that a secant stiffness such as $K_{s,\text{ult}}$ will provide a conservative result.

The stiffness of the connection is meaningful only when compared to the stiffness of the connected members. For example, a connection can be classified as rigid (Type FR) if the ratio (α) of the connection secant stiffness at service level loads (K_{serv}) to the beam stiffness (EI/L), is greater than about 18 for unbraced frames (Fig. 4.2). Generally, connections with α less than 2 are regarded as pinned connections. Limits on the ranges of a cannot be established uniquely because they will vary depending on the limit state used to derive them. For regular frames, for example, one commonly used criterion to establish an upper limit is that the reduction in elastic buckling capacity of the frame due to connection flexibility should not exceed 5% from that given by an analysis assuming rigid connections (Eurocode 3, 1992). Because this reduction in buckling capacity is tied to whether the frame is braced or unbraced, the value of 20 is suggested for unbraced frames, while a value of 8 is sufficient for braced frames. For continuous beams in braced frames, on the other hand, limits based on achieving a certain percentage of the fixed-end moment, or reaching a deflection limit, seem more reasonable (Leon, 1994).

4.2.2 Connection Strength

A connection can also be classified in terms of strength as either a full strength (FS) connection or a partial strength connection (PS). An FS connection develops the full plastic moment capacity of the beam framing into it, while a PS connection can only develop a portion of it. As shown in Fig. 4.3, for classifying connections according to strength, it is common to nondimensionalize the vertical axis of the M-θ curve by the beam plastic moment capacity ($M_{p,\text{beam}}$). Connections not capable of transmitting at least $0.2M_p$ at a rotation of 0.02 rad are considered to have no flexural strength. Because many PR connections do not exhibit a plateau in their strength even at large rotations, an arbitrary rotation value must be established to compare connection strength ($M_{p,\text{conn}}$) to the capacity of the beam. For this purpose, a rotation of 0.02 rad is recommended by the author.

4.2.3 Connection Ductility

Connection ductility is a key parameter either when the deformations are concentrated in the connection elements, as is the typical case in PR connections, or when large rotations are expected in the areas adjacent to the connections, as in the case of ductile moment frames with welded connections. The ductility required will depend on the flexibility of the connections and the particular application (i.e., braced frame in a nonseismic area versus an unbraced frame in a high seismic area).

A connection can be classified as ductile based on both its absolute and its relative rotation capacity (Fig. 4.4). The horizontal axes in Fig. 4.4 show both total connection rotation and connection ductility. Three connection curves are shown: (a) two for connections in special moment frames (SMFs), one with hardening or nondegrading behavior (ND) and one with moderate degradation (D), and (b) one for a degrading connection in an intermediate moment frame (IMF). The total rotation (in terms of milliradians or radians $\times 10^3$) is how typical moment-rotation curves for connection tests are reported. In general, only the envelopes of the cyclic results are shown, and a very coarse relative limit between ductile and nonductile connections for seismic design can be set a total rotation of 0.02 rad for nonseismic and 0.04 rad for seismic design.

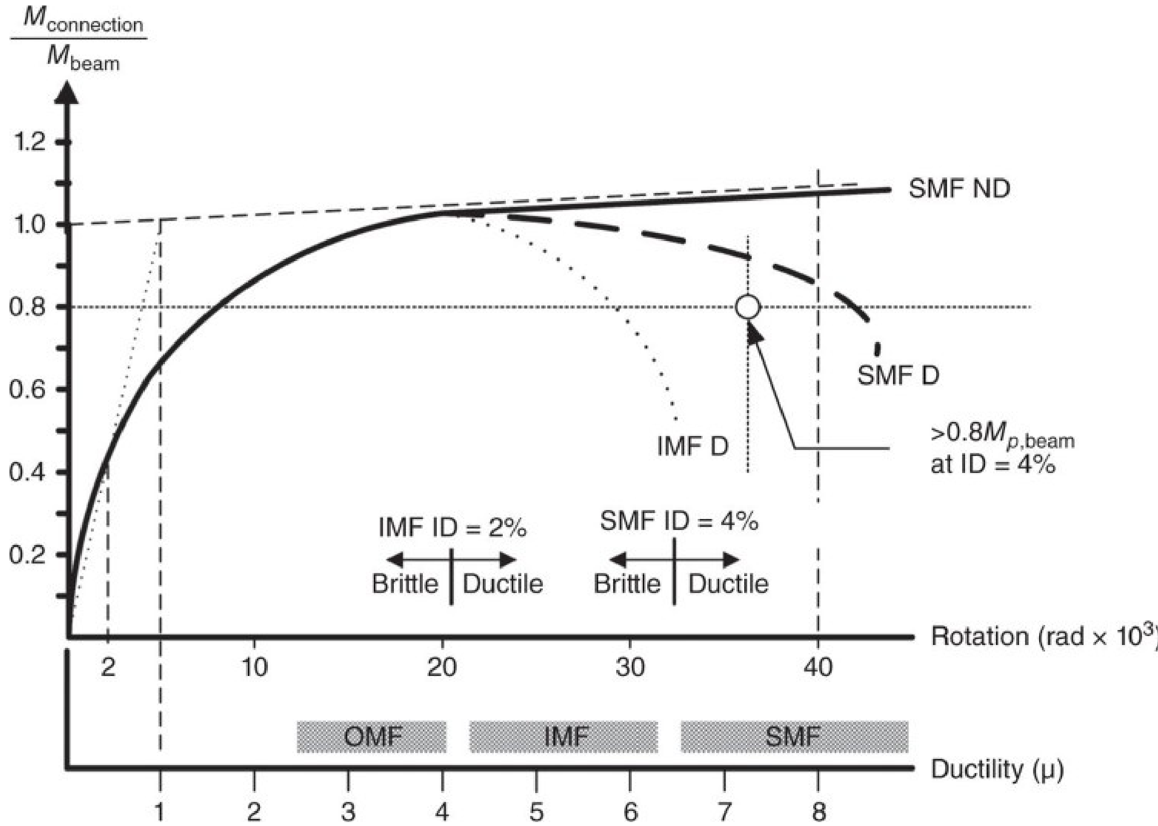

FIGURE 4.4 Possible ductility classification for seismic connections.

The relative ductility index ($\mu = \theta_u/\theta_y$) that can be used for comparing the rotation capacity of connections with similar moment-rotation characteristics. In order to compute a relative ductility (μ), a yield rotation (θ_y) must be defined. For PR connections, such as the one shown in Fig. 4.3, this definition is troublesome since a yield moment is difficult to determine. In this case, for the case of the connection in a special moment frame with no degradation (SMF, solid line) and for illustrative purposes only, the yield rotation is defined as the rotation at the intersection of the service and hardening stiffnesses of the connection. In general, a mimimum relative ductility in the range of 2.5 to 3 is associated with well-detailed, nonseismic connections (OMF or ordinary moment frame). Relative ductilities of 6 or more are associated with well-detailed, seismic connections (SMF or special moment frames), while relative ductilities of in the range of 4 to 6 are associated with the intermediate seismic category (IMF or intermediate moment frame).

Since the end of the work on the SAC projects, the qualifications for connection performance has undergone two significant changes. First, the performance criteria for special and intermediate moment frames (0.04 rad of total connection rotation for SMF and 0.03 rad for IMF, both including an assumed 0.01 rad of elastic deformation) have been changed to the total interstory drifts (ID = 4% and 2%, respectively, for SMF and IMF). These are shown also in Fig. 4.4, but their location in this figure is arbitrary with respect to the axes as they do not refer to connection but story deformation. In addition, the original requirement that the connection capacity at 0.04 rad does not decrease by more than 20% from its maximum has been changed to a requirement that at 4% drift SMF connections do not have less than 80% of the nominal flexural capacity of the beam. These two are significant changes, as direct conversions between both interstory drift and connection rotation and connection and beam strength are not possible.

Both the absolute and relative rotation capacities need to take into account any strength degradation that may occur as a result of local buckling or slip, particularly under cyclic loads. The behavior of the connections shown by the solid (SMF ND), dashed (SMF D), and dotted (IMF D) lines in Fig. 4.4 can lead to significant differences in frame behavior, especially with respect to strength and stability. Finally, it should be emphasized that the limits discussed above, with the exception of the interstory drift and 0.8 $M_{p,beam}$ limits appearing in AISC 341, are based purely on the opinions of the author.

4.2.4 Derivation of *M*-θ Curves

As noted earlier, *M*-θ curves were typically derived from experiments, and, more recently, from finite element and similar analyses. Many of the tests have been collected into databases (Ang and Morris, 1984; Goverdhan, 1984; Nethercot, 1985; Kishi and Chen, 1986; Chan and Chui, 2000; for example). Based on these databases, equations for the complete *M*-θ curves for different types of connections have been proposed. However, numerous important variables, such as the actual yield strength of the materials and the torque in the bolts, are generally poorly documented or missing for many of these tests. Thus many of the *M*-θ curves and equations available from these databases cannot be considered as reliable. Some, but not all of these reservations, have been addressed by more recent databases (Chen et al., 2011). Finally, care should be exercised when utilizing tabulated moment-rotation curves not to extrapolate to sizes or conditions beyond those used to develop the database since other failure modes, such as frame stability, may control the design (ASCE, 1997).

Two analytical approaches have recently become practical alternatives and/or complements to experimental testing in developing *M*-θ curves. The first alternative is a detailed, nonlinear finite element analysis of the connection. While time consuming, because of the extensive parametric studies required to derive reliable *M*-θ curves, this approach has gone from a pure research tool to an advanced design office tool in just a few years, thanks to the tremendous gains in computational power available in new desktop workstations.

The second approach is the one proposed by the Eurocodes and commonly labeled the component approach. In this case each deformation mechanism in a joint is identified and individually quantified through a series of small component tests. These tests are carefully designed to measure one deformation component at the time. Each of these components is then represented by a spring with either linear or nonlinear characteristics. These springs are arranged in series or in parallel and the overall *M*-θ curve is derived with the aid of simple computer programs that conduct the analysis of the spring system.

Figure 4.5 shows a typical component model for a T-stub connection. In this example the K1 and K2 springs model the panel zone deformation due to shear, while springs K3 and K4 model the bending deformations of the T stubs. Springs K3 and K4 are made up of the contributions of several other springs that model different deformation components (Fig. 4.5*b*). For simplicity, in Fig. 4.5, the most relevant springs are shown in series, which is probably sufficient to determine a monotonic *M*-θ curve for design. In reality, there are complex interactions between these springs that are not shown here, which will lead to the need for other springs, gap and dashpot elements in parallel and in series to properly model cyclic behavior.

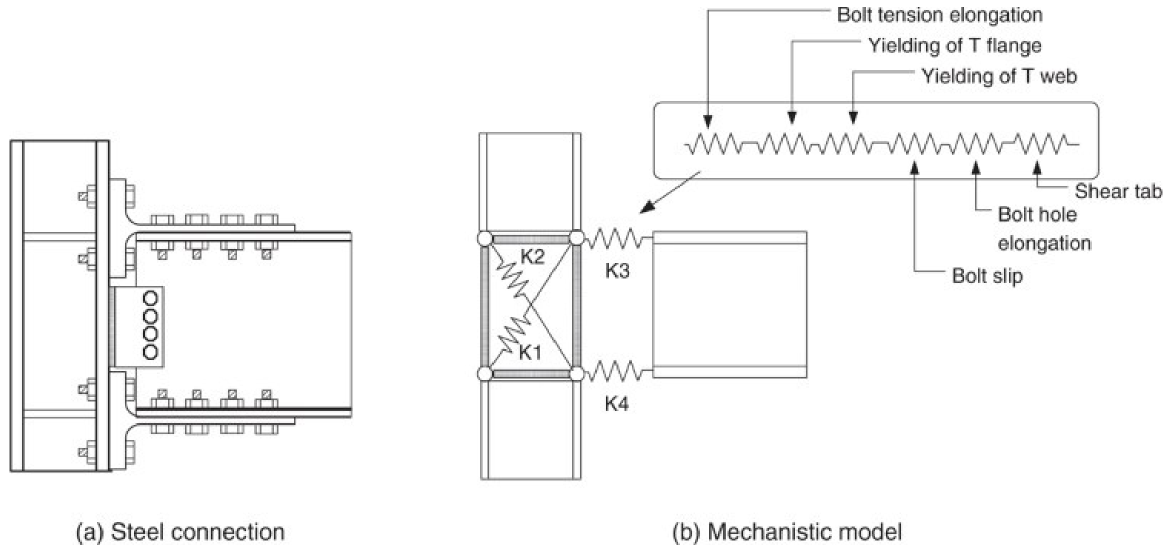

FIGURE 4.5 Component model for a T-stub connection.

4.2.5 Analysis

For many types of connections, the stiffness at the service load level falls somewhere in between the fully restrained and simple limits, and thus designers need to account for the PR behavior. The *M*-θ characteristic can be obtained from experiments or models as described in the previous section. The effect of PR connections on both force distribution and deformations in simple systems will be illustrated with two short examples.

Figure 4.6 shows the moments and deflections in a beam subjected to a uniformly distributed load. The horizontal axis is logarithmic and shows the ratio of the connection to beam stiffness ($\alpha = K_{serv}L/EI$). The deformations range from that of a simply supported beam ($\Delta = 5\,wL^4/384EI$) for a very flexible connection ($\alpha \rightarrow 0$) to that of a fixed beam ($\Delta = wL^4/384EI$) for a very stiff connection ($\alpha \rightarrow \infty$). From both the deflection and force distribution standpoints, for a range of $15 < \alpha < \infty$, the behavior of the connection is essentially that of a fixed beam. Similarly, for a range of $0 < \alpha < 0.3$, the beam is essentially simply-supported. Note that the ranges given here were selected arbitrarily, and that they will vary somewhat with the loading condition. This is why, as was noted earlier in the discussion of connection stiffness, the selection of limits for α to separate FR, PR, and simple behavior is not straightforward. It is important to note, however, that the horizontal axis of Fig. 4.6 is logarithmic. This means that apparently large changes in connection stiffness actually result in much smaller changes in forces or deformations. This lack of sensitivity is actually what allows us to design PR connections by simplified methods, since it means that the connection stiffness does not need to be known with great precision.

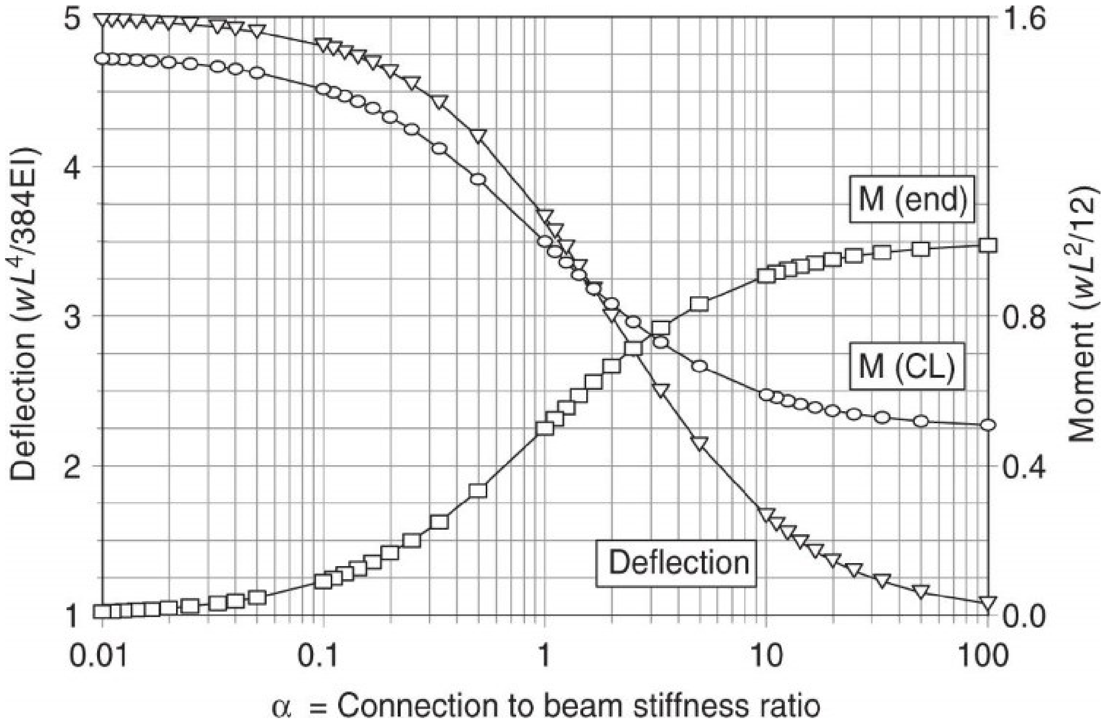

FIGURE 4.6 Moments and deflections for a beam under a uniformly distributed load with PR connections at its ends.

Figure 4.7 shows the results of an analysis for the general case of a one-story, one-bay frame with springs both at the connections to the beam (K_{conn}) and at the base of the structure (K_{base}). A simple formula for the drift cannot be written for this general case. Figure 4.7 shows the drifts for five levels of base fixity ($\alpha_{base} = K_{bas} H_e/EI_{col}$ = 0, 1, 2.5, 5, 10, and 4) versus a varying $\alpha_{beam} = (K_{conn}\, L/EI)$. The calculations are for a frame with an I_{beam} = 2000 in^4, L = 288 in, I_{colunm} = 500 in^4, H = 144 in, a concentrated horizontal load at the top of P = 2.4 kips, and a distributed load on the beam of w = 0.08333 kip/in. The vertical axis gives the deflection as a multiplier (τ) of the fully rigid case, where $K_{conn} = K_{base} = \infty$. The drift value for the latter is 0.025 in. For the case of K_{base} = 4, as the connection stiffness decreases, the deflection reduces to that of a cantilever subjected to $P/2$ (τ = 3.25). For the other extreme (K_{base} = 0), the deflections increase rapidly from τ = 4.06 as the stiffness of the connection is decreased, since we are approaching the unstable case of a frame with pins at all connections as $\alpha \rightarrow 0$. Figures such as this indicate the wide range of behavior that PR connections can provide, and the ability of the designer to use the connection stiffness to tailor the behavior of the structure to its performance requirements.

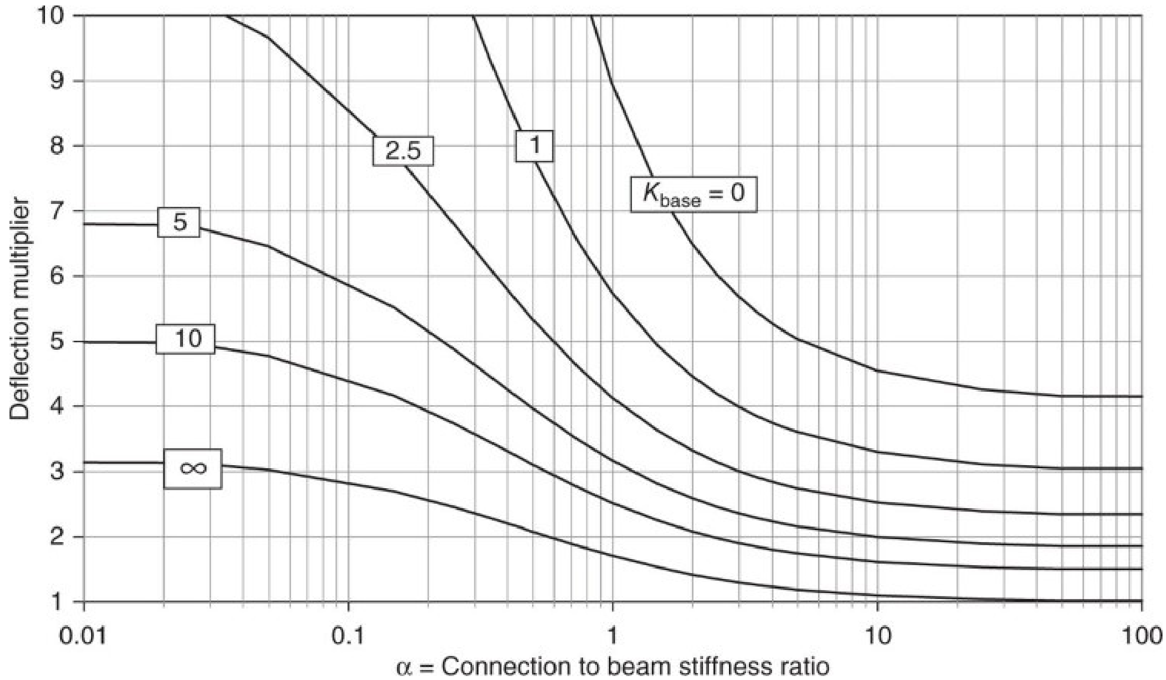

FIGURE 4.7 Drifts of a simple frame with various degrees of base fixity and connection stiffness.

Another very important lesson to be drawn from Fig. 4.7 is the large effect of the base fixity on frame drift. While it is common to assume in the analysis that the column bases are fixed, such degree of fixity is difficult to achieve in practice, even if the column is embedded into a large concrete footing. Most footings are not perfectly rigid or pinned, with the practical range probably being $1 < K_{base} < 10$. As can be seen from Fig. 4.6, the difference in drift between the assumption of $K_{base} = \infty$ (perfect base fixity) and a realistic assumption ($K_{base} = 10$) ranges from about 50% when K_{conn} is ∞ to about 300%, when α is 0.

Figure 4.7 indicates that there are infinite combinations of K_{base} and K_{conn} for a given deflection multiplier. Consider the case of a one-story, one-bay frame with the properties given for Fig. 4.6. For a target deflection multiplier of, say, 3, one can design the frame with a pinned base and a K_{conn} approaching infinity (α = 0), or one can design a rigid footing with a connection having an α = 2 (pinned). This flexibility in design is what makes PR connection design both attractive and somewhat disconcerting. It is attractive because it provides the designer with a wide spectrum of possibilities in selecting the structural members and their connections. It is disconcerting because most designers do not have extensive experience with PR analysis and PR frame behavior.

There are currently numerous good texts that address the analysis and design of PR frames (CTBUH, 1993; Leon et al., 1996; Chan and Chui, 2000; Faella et al., 2000; Chen et al., 2011). There is a considerable range in the complexity of the analysis approaches proposed in the literature. The appropriate degree of sophistication of the analysis depends on the problem at hand. When incorporating connection restraint into the design, the designer should take into account the effect of reduced connection stiffness on the stability of the structure, and the effect of connection deformations on the magnitude of second order effects (ASCE, 1997). Usually design for PR construction requires separate analysis to determine the serviceability limit state and the ultimate limit state because of the nonlinear nature of the M-θ curves.

4.3 DESIGN OF BOLTED PR CONNECTIONS

The design of a connection must start from a careful assessment of its intended performance. This requires the designer to determine the performance criteria with respect to stiffness (FR, PR, or simple), strength (FS or PS), and ductility. The stiffness is critical with respect to serviceability, while strength and ductility are critical with respect to life safety issues. These criteria must be consistent with the model assumed for analysis. From Fig. 4.7, if an assumption of a rigid connection was made in the analysis, the resulting connection will typically be fully welded, welded-bolted or a stiffened thick end plate type. Similarly, if the connection was assumed as simple, then a shear plate welded to the column and bolted to the beam or angles bolted to both column and beam are appropriate.

If explicit use of PR behavior was made in the analysis, in the form of a rotational spring with a given K_{serv}, then a wide variety of connections can be chosen, ranging from an end plate (close to FR/FS performance) to top-and seat angles (close to simple performance). The key here is to match the K_{serv} of the connection as designed to that assumed in the analysis. The matching should be done at the service level because drift and deflection criteria will probably govern the design in modern steel frames. The stiffness of the connection should be checked with at least the component model approach (Fig. 4.5). Since the stiffness of the connection will be dependent on the actual configuration of the connecting elements and the size of the framing members, it is possible to adjust the stiffness to match that assumed in design.

The ultimate strength and ductility of the connection as designed must also be compatible with that assumed in design. In this case it is imperative to identify all possible failure modes for the connection as designed. Moreover, it is necessary to understand the hierarchy of failure modes so that modes are excluded. Table 4.1 (FEMA, 1997) shows a proposed hierarchy for seismic design of a variety of connections: column welded-beam bolted (CW-BB), column- and beam-bolted or T-stub (CB-BB), end plates (EP), top-and seat connections (TS), and partially restrained composite connections (PR-CC). The table indicates the type of failure associated with each mechanism (ductile, semiductile, or brittle), and lists the ductile and semiductile mechanisms in descending order of desirability. This table is arbitrary and reflects the biases of the author. As an example of how this hierarchy can be achieved for a T stub, Fig. 4.8 shows the possible yielding (mechanisms 1 to 9 in likely order of occurrence) and fracture mechanisms (mechanism 10, any of which will lead to connection failure). For many PR connections, the numerous sources of deformations provide considerable ductility but complicate the design. Designers are encouraged to develop their own lists and rankings based on their experience and regional preferences of fabricators and erectors.

TABLE 4.1 Failure Modes for Bolted Connections

Connection Type	CW-FB	CB-FB	EP	TS
Strength (FS or PS)	FS	FS	FS/PS	PS
Stiffness (FR or PR)	FR	FR/PR	PR	PR
DUCTILE:				
Slippage of slip-critical (friction) bolts	1	1		
Flexural beam yielding adjacent to nodal zone	2	2	1	
Yielding of connecting elements in tension	3	3		2
Formation of yield lines in connecting elements		4	2	1
Yielding of slab reinforcement in tension				
Panel zone yielding	4	5	3	3
Limited local buckling	5	6	4	4
SEMI-DUCTILE:				
Elongation of bolt holes due to bearing	6	7		5
Yielding of bolts to column flange in tension	8	9	5	6
Shear yielding of bolts to beam flange	7	8		7
Severe local buckling of beam flange	9	10	6	8
BRITTLE:				
Fracture of welds between column and plate	A			
Fracture/failure of shear connection to web	A	A	A	A
Bearing/crushing failure of concrete				
Fracture of shear studs and rebar				
Fracture of beam flange due to local buckling	A	A	A	
Shear failure of bolts	A	A	A	A
Tensile failure of bolts (including prying action)		A	A	A
Fracture of beam through net section		A	A	A
Fracture of connecting element through net section		A	A	A
Column web failure (yielding, crippling, buckling)	A	A	A	A
Edge distance or spacing failure of bolts	A	A	A	A
Block shear	A	A	A	A

A indicates a brittle failure mode that should be carefully checked in design.
CW-FB = column welded, flange bolted connections
CB-FB = column bolted, flange bolted
EP = end plate
TS = top-and-seat angles with double web angles
PR-CC = partially restrained composite connection

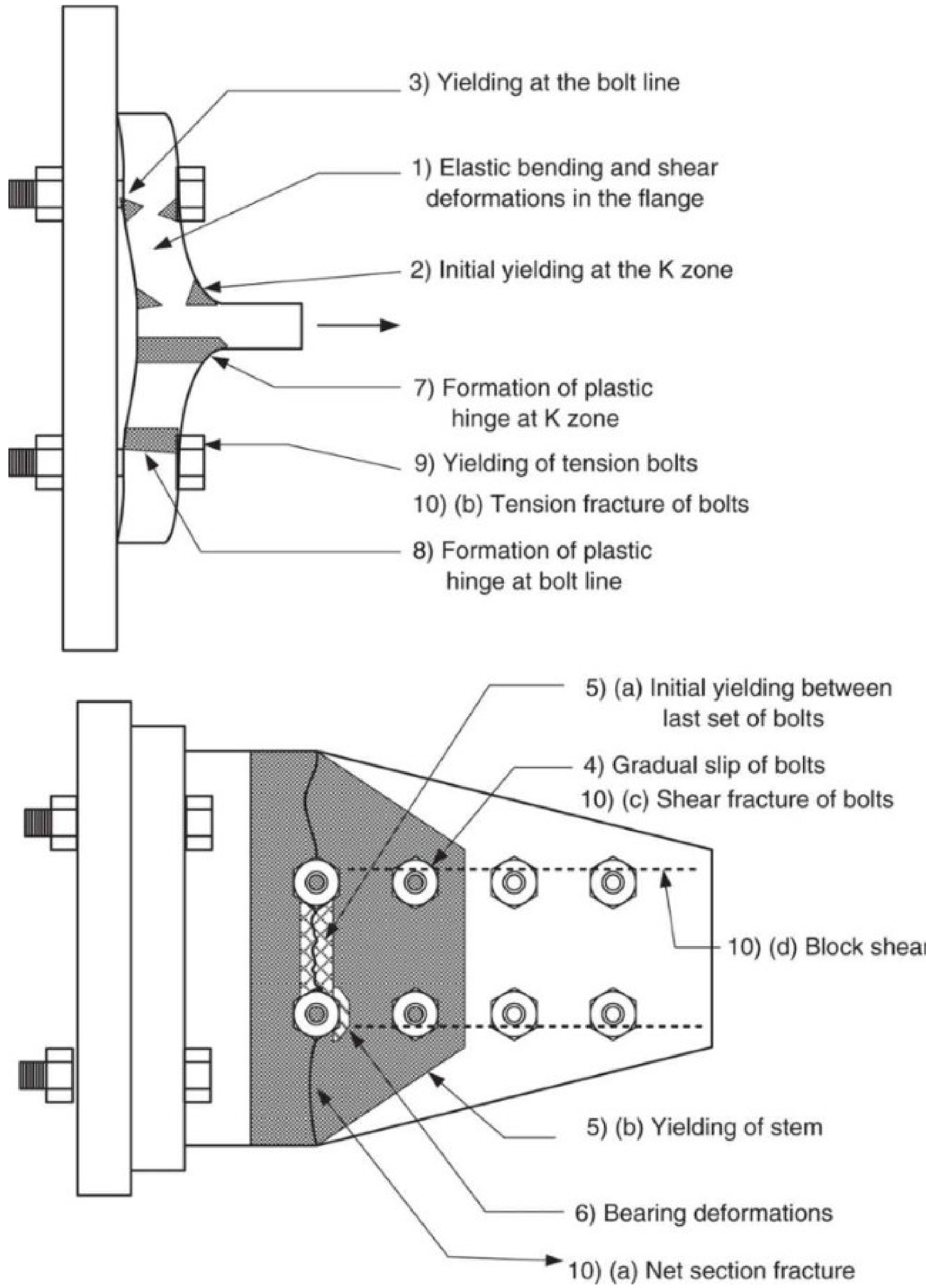

FIGURE 4.8 Yielding and fracture mechanisms in a T-stub connection.

Special note should be made of the fact that the material properties play an important role in connection performance. In particular, the separation between the expected yield (R_yF_y) and expected ultimate strength (R_tF_u) of the material is a key factor. As our understanding of the failures in steel frames during the 1994 Northridge earthquake improves, it is clear that material performance played an important role in some of the failures encountered. Issues related to the ductility and toughness of the base materials for both welds and bolts, installation procedures, QA/QC in the field, and need for new, tighter material specifications have received considerable attention (FEMA, 1997). Designers should strive to obtain the latest information in this area so that future failures are avoided.

The design process outlined places a heavy additional burden on designers, both in terms of professional responsibility and continuing education, not to mention substantial additional design time. Two important points need to be made with respect to these issues. First, as our designs become more optimal with respect to both strength and stiffness, many of the traditional assumptions made in design need to be carefully reexamined. These include, for example, serviceability criteria based on substantially different partition and cladding systems from those used today. Second, these optimized systems are far more sensitive to the assumptions about connection behavior, since typically far fewer moment-resisting connections are used in steel frames today than twenty years ago.

In this section, first the fundamentals of design for full strength, partially restrained (FS/PR) bolted connections will be discussed, followed by that for partial strength, partially restrained (PS/PR) ones. The design for both seismic and nonseismic cases will be discussed. The emphasis will be on understanding the basic steps in connection design and developing an understanding of the crucial mechanisms governing their behavior.

4.3.1 Column Welded-Beam Bolted Connections

The design of column welded-beam bolted (CW-BB) connections (Fig. 4.9) has been discussed extensively by Astaneh-Asl (1995) and Schneider and Teeraparbwong (2002). The mechanistic model for this type of connection, labeled column bolted-beam bolted (CW-BB), is essentially the same as that shown in Fig. 4.5 for a T-stub connection. The main differences are that the springs representing the tension elongation of the bolts and the yielding in the flange have to be replaced by a spring that represents the behavior of the weld between the column flange and the beam flange.

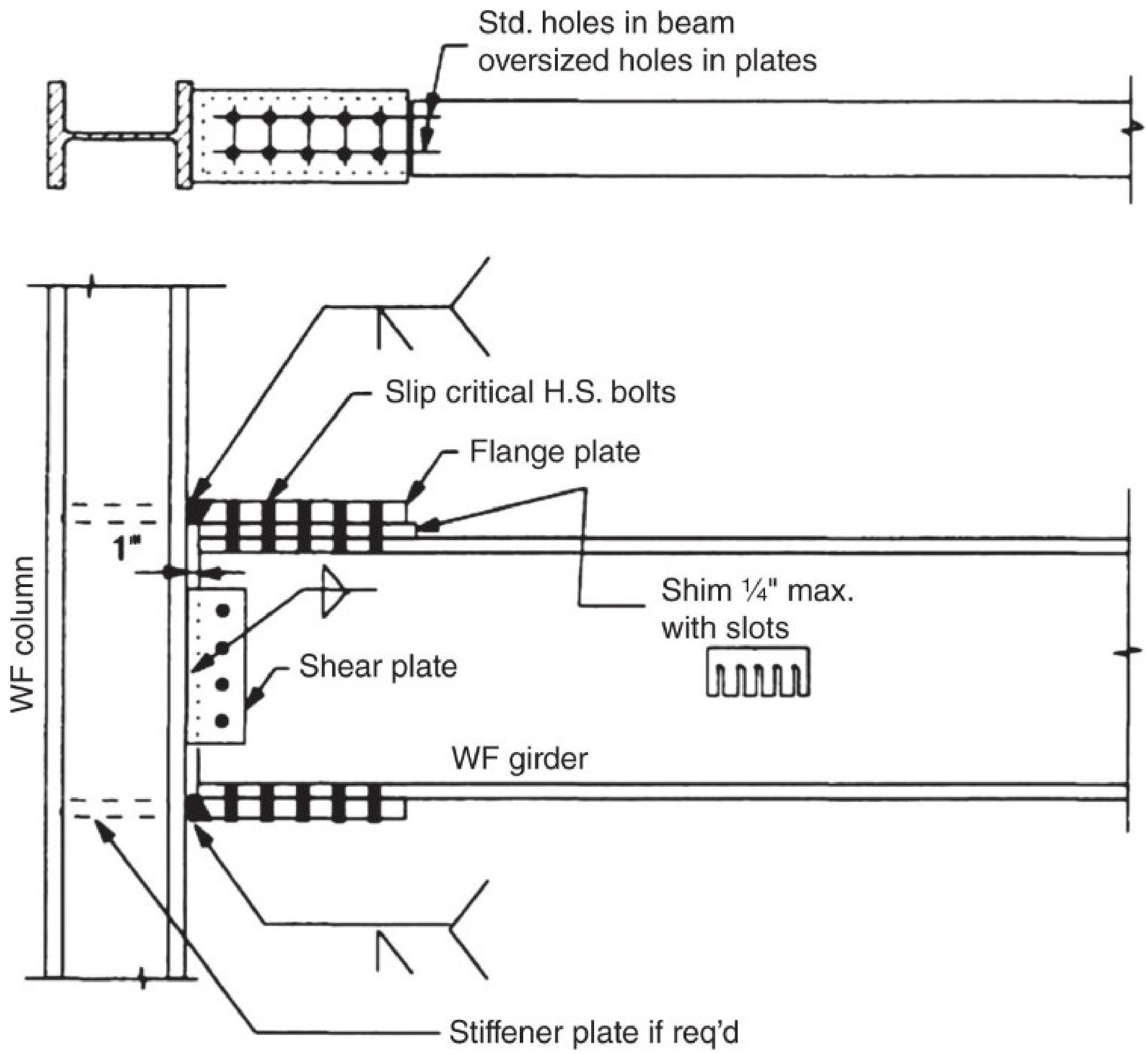

FIGURE 4.9 Typical CW-BB connection (Astaneh-Asl, 1995).

Table 4.1 lists the main failure modes for this type of connection. In general, the desired failure mechanisms will be slip of the bolts followed by yielding of the beam and the connection plate. The main failure modes to avoid are brittle failure of the welds, shear failure of the bolts, and a net section failure in the connecting plate or beam. With this hierarchy established, it is possible to develop a design strategy, as outlined in the steps shown below, for the design of these connections under monotonic loads.

The design of any connection subjected to seismic loads is similar in principle to the static design, except that a capacity design approach must be followed. In this context, capacity design implies that the connection must be designed to behave in a ductile manner under the maximum expected forces that can be introduced by the framing members. Thus, for WC-BB connections, the welds need to be strong and tough enough, such that the weld strength does not control, and fracture problems related to the welding procedures and materials are eliminated. For WC-BB connections, yielding should be limited to the connection plate or the beam flange. This requires a careful assessment of the minimum and maximum capacities associated with each of the springs in Fig. 4.4, since the forces are inertial rather than gravity type. For seismic design of connections, both AISC 341-05 and AISC 358-05 require that the expected (or mean) strength of the beam be used rather than its nominal (or 5% fractile) strength. To accomplish this, nominal yield and ultimate strength values are multiplied by either a R_y or R_t factor, which varies with the material type. In addition, to account for peak connection strength, strain hardening, local restraint, additional reinforcement and other connection conditions, an additional factor (C_{pr}) is used, where C_{pr} is taken as the average of the yield plus ultimate strength divided by the yield strength. C_{pr} need not be taken as greater than 1.2. This capacity design approach is different from the static (i.e., nonseismic) case, where the connection can be designed for forces derived from the structural analysis, and without regard to the actual ultimate capacity and failure mode of each of the connection components.

Before looking at examples of CW-BB connections for both static and seismic loading cases, a number of important design issues need to be understood. These issues, discussed in detail below, are of particularly significance for CW-BB connections, but the principles involved are applicable to most strong PR connections:

1. Proportioning of flange connection: Whenever possible the yield strength of the connection elements (top and bottom plates) should be matched to that of the beam flange. This will ensure that distributed yielding takes place and that severe local buckling will not ensue. Severe local buckling can result in an early fracture of the beam flanges if cyclic loads are present. Astaneh-Asl (1995) recommends that for yielding on the gross section:

$$b_p t_p R_y F_{yp} \cong b_f t_f R_y F_{yf} \tag{4.1a}$$

   where b and t are the width and thickness and the subscripts p and f refer to the plate and beam flange, respectively. Usually, the expected yield strength of the materials is not known when the design is done. For designs not involving seismic forces, the nominal material properties, as opposed to the nominal ones, can be used throughout. For the case of seismic forces, the same assumptions can be made with regards to sizing the plate, but the C_{pr} factor (typically, $C_{pr} = 1.1\,[(F_y + F_u)/2]$) must be applied to allow for overstrength and strain-hardening. To avoid a tensile rupture of the flange, by AISC 360, Section F13:

$$d_b \leq \frac{1}{2} b_f \left(1 - \frac{Y_t F_y}{F_u}\right) - 3 \tag{4.1b}$$

   where $Y_t = 1$ if $(F_y/F_u) < 0.8$ or 1.1 otherwise. In order to ensure a ductile failure, the ratio of the effective area (A_e) to the gross area (A_g) of the plate should be at least:

$$\frac{A_e}{A_g} \geq \frac{R_y F_y}{R_t F_u} \tag{4.2}$$

2. For the case of seismic loads another key issue is the design of the welds to the column flange. In this area there are recent detailed guidelines proposed by SAC (FEMA, 1997, 2000) and incorporated into AISC 341 and 358 (AISC, 2016b, 2016c). The AISC 341 provisions require that a welding procedure specification (WPS) be prepared as required by AWS D1.1 (AWS, 2015). AWS D1.1 provides detailed procedures for welding (see

Chap. 3) and this standard should become familiar to all structural engineers. In addition, a minimum Charpy N-Notch test (CVN) toughness of 20 ft-lb at –20°F is required of all filler metal by the seismic AISC Specification.

3. Local buckling criteria: The current limits suggested by AISC (0.38 $^{(0.38}\sqrt{(E/F_y)}$ for b/t in beam flanges in compression and 3.76/$^{3.76/}\sqrt{(E/F_y)}$ for webs in flexural compression seem to provide a reasonable limit to ensure that the nominal plastic moment capacity of the section is reached. For seismic applications, these limits have been tightened somewhat, to 0.30$^{0.30}\sqrt{(E/F_y)}$ for b/t in beam flanges and something less than 3.14 $^{3.14}\sqrt{(E/F_y)}$ for webs in flexural compression to ensure not only that the capacity can be reached, but also that sufficient rotational ductility is available. The typical buckle that forms when these criteria are met is a smooth, small local buckle. This precludes the development of a sharp buckle that may lead to fracture under reversed inelastic loading. The current limits on web slenderness also seem to provide reasonable limits, although the actual performance will be tied to the detailing of the web connections and whether composite action is expected. The slenderness of the connection plates, measured between the weld to the column flange and the centerline of the first row of bolts, should also be kept as low as practicable to prevent the formation of a local or global buckle in this area. Current criteria for unsupported compression elements are applicable in this case.
4. Bolts: The bolt group should be designed not only to prevent a shear failure of the connectors, but also to provide adequate performance during the slipping phase of the moment-rotation behavior. Since slip provides a good energy dissipation mechanism, it is prudent to design the connection such that the slip occurs well above the service load but also below the ultimate strength of the connection. To meet this criterion, Astaneh-Asl (1995) recommends that the nominal slip resistance (F_{slippage}) be such that

$$1.25F_{\text{service}} > F_{\text{slippage}} < 0.8F_{\text{ultimate}}$$

where F_{service} corresponds to the nominal slip strength of the bolt group and F_{ultimate} corresponds to the nominal shear strength of the bolts.
5. Web connection design: The design of the web connection is usually made without much regard to the contribution of this part of the connection to the flexural strength of the joint, unless the flange connections carry less than 70% of the total moment (AISC, 2016b). It is clear from the performance of MRFs during the Northridge earthquake that careful attention should be paid to ensure that the web connection is detailed to provide rotational ductility and strength that are compatible with the action of the flanges. Astaneh-Asl (1995) suggests that the shear plates be designed to develop the plastic moment strength of the web:

$$h_p t_p (0.6F_{yp}) > h_{gw} t_{gw} (0.6F_{yw})$$

$$h_p^2 t_p F_{yp} > h_{gw} t_{gw} F_{yw}$$

where h and t are the depth and thickness, F_y is the yield strength, and the subscripts p and gw refer to the shear plate and the beam web, respectively. Here again, allowances should be made for the steel overstrength (say R_y = 1.1 to 15). Failure modes to be avoided include bolt shear, block shear, net area fractures, and weld fractures.

Design Example 4.1. Design a full strength connection between a W530 × 92 girder and a W360 × 179 column. Both sections are A572M Grade 345. The beam clear span is 7.5 m. Design for wind loads assuming the analysis shows a maximum moment (M_u) of 600 kN-m and a maximum shear (V_u) of 350 kN at the column face. The service moment (M_{serv}) is 250 kN-m. Use Grade A M22 bolts with threads excluded from the shear plane.

1. Check beam local buckling:

$$\text{Flange:}\quad \left(\frac{b}{t}\right)=6.70\leq 0.38\sqrt{\frac{E}{F_y}}=9.2,\text{ ok}$$

$$\text{Web:}\quad \left(\frac{h}{t_w}\right)=46.9\leq 3.76\sqrt{\frac{E}{F_y}}=90.6,\text{ ok}$$

2. Check net area fracture versus gross section yielding of the girder flange by AISC 360 B3, Section F13. For the W530 × 92, b_f = 209 mm, t_f = 15.6 mm, d = 533, and $S_x = 2070 \times 10^3$ mm^3; holes for M22 bolts assumed as standard holes with 24 mm diameter or 48 mm total):

$Y_t = (F_y/F_u) = 450/345 = 0.76 < 0.8 \rightarrow Y_t = 1.0$

$F_u A_{fn} = (450\text{ MPa})[(209-48)(15.6)\text{ mm}^2] = 1130\text{ kN}$

$Y_t F_y A_{fg} = (1.0)(345)(209)(15.6) = 1125\text{ kN}$

Since the net section governs ($F_u A_{fn} < Y_t F_y A_{fg}$), the girder moment capacity from AISC 360, F13, Eq. (F13-1), is

$$M_{n,g}\approx\frac{F_uA_{fn}}{A_{fg}}S_x=\frac{(450)(209-48)(15.6)}{(209)(15.6)}\left(2070\times10^3\right)=717.6\text{ kN-m}$$

$$\phi M_{n,g}=(0.9)(717.6)=645.8\text{ kN-M}>M_u=600\text{ kN-m, ok}$$

$$F_{\text{flange}}=F_{\text{plate}}=\frac{717.6\text{ kN-m}}{533\text{ mm}}=1212\text{ kN}$$

For designing the connection, it is important not to underestimate the girder moment. The $M_{n,g}$ calculated above is about equal to that given by $F_y Z_{x,n}$, where $Z_{x,n}$ is the plastic net area. Since this is a wind design case, the F_{plate} computed appears reasonable for design as it will ensure that the beam will yield first, but that there is not a substantial connection overcapacity as will be the case for a seismic design.

3. Determine the size of the flange plate, assuming that the plate thickness (t_p) will be 18 mm and that the plate is 345M. Balancing the plastic capacity of the plate against that of the beam, gives a plate width (b_p) for the gross and net area cases of

$$b_{p,g} \geq \frac{F_{\text{beam}}}{\phi t_p F_{yp}} = \frac{(1212\text{ kN})}{(0.9)(18\text{ mm})(345\text{ MPa})} = 244\text{ mm}$$

$$b_{p,n} = b_{p,g} - 48 = \frac{F_{\text{beam}}}{\phi t_p F_u} = \frac{(1212\text{ kN})}{(0.75)(18\text{ mm})(450\text{ MPa})} = 248\text{ mm} \rightarrow \text{Try } b_f = 250\text{ mm}$$

4. Check gross (A_{pg}) and net (A_{pn}) area forces for the plate:

Gross section: $F_{pg} = \phi A_{pg} F_{vp} = (0.9)[(250 \times 18)\text{ mm}^2](345\text{ MPa}) = 1397\text{ kN}$

Net section: $F_{pn} = \phi A_{pn} F_{up} = (0.75)[(250 - 48) \times 16)\text{ mm}^2](450\text{ MPa}) = 1227\text{ kN}$

F_{pn} governs and is slightly higher than $F_{\text{plate}} \rightarrow$ ok

5. Determine number (N_s) of F3125 M22 bolts required for shear in the flanges using AISC360 J3, Eq. (J3-1b):

$$R_u = \phi R_{nv} = \phi F_{nv} n_{sp} A_b$$

$$\phi = 0.75$$

$$F_{nv} = 457\text{ MPa (F3125, threads excluded, AISC Table J3.2)}$$

$$n_{sp} = 1\text{ (number of shear planes)}$$

$$A_b = \left(\frac{\pi d^2}{4}\right) = \left(\frac{3.14(22)^2}{4}\right) = 380\text{ mm}^2$$

$$R_u = \phi R_{nv} = (0.75)(457)(1)(380) = 130.3\text{ kN}$$

$$N_s = \left(\frac{F_{\text{flange}}}{R_u}\right) = \left(\frac{1212}{130.3}\right) = 9.3,\ \text{use } N = 10\text{ bolts}$$

Assuming a gage of 125 mm, this means the edge distance for a 250 mm-wide plate is equal to the minimum required (28 mm by AISAC Table J3.4M). Assuming (1) a bolt spacing of 3d, (2) a distance between the last bolt and the weld at the column flange of equal to 100 mm, and (3) a distance of 50 mm between the centerline of the last bolt and the end of the plate, the minimum length of the plate is 414 mm, say 425 mm.

6. Check bolt bearing on the beam flange by AISC 360 J3, Eq. (J3-6a):

$$F_{\text{bearing}} = \phi N R_n = \phi N(2.4 d t F_u)$$

$$F_{\text{bearing}} = (0.75)(10)(2.4)(22)(15.6)(450) = 2780 \text{ kN} \geq F_{\text{flange}}, \text{ ok}$$

7. Check bolt service load slip capacity by AISC 360 J3, Eq. (3-4):

$$R_{u,\text{slip}} = \phi\mu\, D_u h_f T_b n_s$$

$\phi = 1.00$ (standard size hole)

$\mu = 0.3$ (class A, unpainted clean mill scale)

$D_u = 1.13$ (multiplier for actual bolt pretension)

$h_f = 1.0$ (factor for fillers)

$T_b = 176$ kN (minimum bolt pretension, AISC Table J3.1M)

$n_s = 1$ (number of slip planes)

$$R_{u,\text{slip}} = \phi\mu\, D_u h_f T_b n_s = (1.00)(0.3)(1.13)(1)(176)(1) = 59.7 \text{ kN}$$

$$M_{\text{slip}} = (10 \text{ bolts})(59.7 \text{ kN/bolt})(533 \text{ mm}) = 318 \text{ kN-m}$$

$$M_{\text{slip}} = 318 \text{ kN-m} > 1.25\, M_{\text{serv}} = 1.25\,(250.0 \text{ kN-m}) = 312.5 \text{ kN-m, ok}$$

8. Check block shear by AISC 360 J4, Eq. (J4-5). Assume shear failure along the bolts and tensile failure across bolt gage and $U_{bs} = 1.0$:

Gross area in shear = A_{gv} = 2 (425 – 100 – 11)(18) = 11306 mm^2

Gross area in tension = A_{gt} = (125)(18) = 2250 mm^2

Net area in shear = A_{nv} = 11306–[2(3.5 × 24)(18)] = 8282 mm^2

Net area in tension = A_{nt} = 2250–[2(24 × 18)] = 1386 mm^2

$$R_n = 0.6\, F_u\, A_{nv} + U_{bs}\, F_u\, A_{nt} \leq 0.6\, F_y\, A_{gn} + U_{bs}\, F_u\, A_{nt}$$

$$R_n = 0.6(450)(8282) + (1.0)(450)(1386) \leq 0.6(345)(11306) + (1.0)(450)(1386)$$

$$R_n = 2560 \text{ kN} \leq 2964 \text{ kN}$$

$$\phi R_n = (0.75)(2560) = 2145 \text{ kN} >> F_{\text{flange}} = 1212 \text{ kN, ok}$$

9. Determine weld size: The weld thickness, based on a 70-ksi electrode, is

$$t_{\text{weld}} = F_{\text{plate}}/(0.6 F_{\text{EXX}} \times b_p)$$

$$= 1201 \text{ kN}/[(0.6)(480 \text{ MPa})(240 \text{ mm}) = 17.4 \text{ mm, say } 19 \text{ mm}$$

(use full penetration weld)

10. Detail the shear connection to the web: The design of the shear connection for this case will not be carried out in detail here (see Chap. 2 for design of shear connections). From the AISC Manual, a pair of L102 × 102 × 7.9 angles with four M22 Grade A bolts provide adequate shear resistance. The final design is shown in Fig. 4.10.

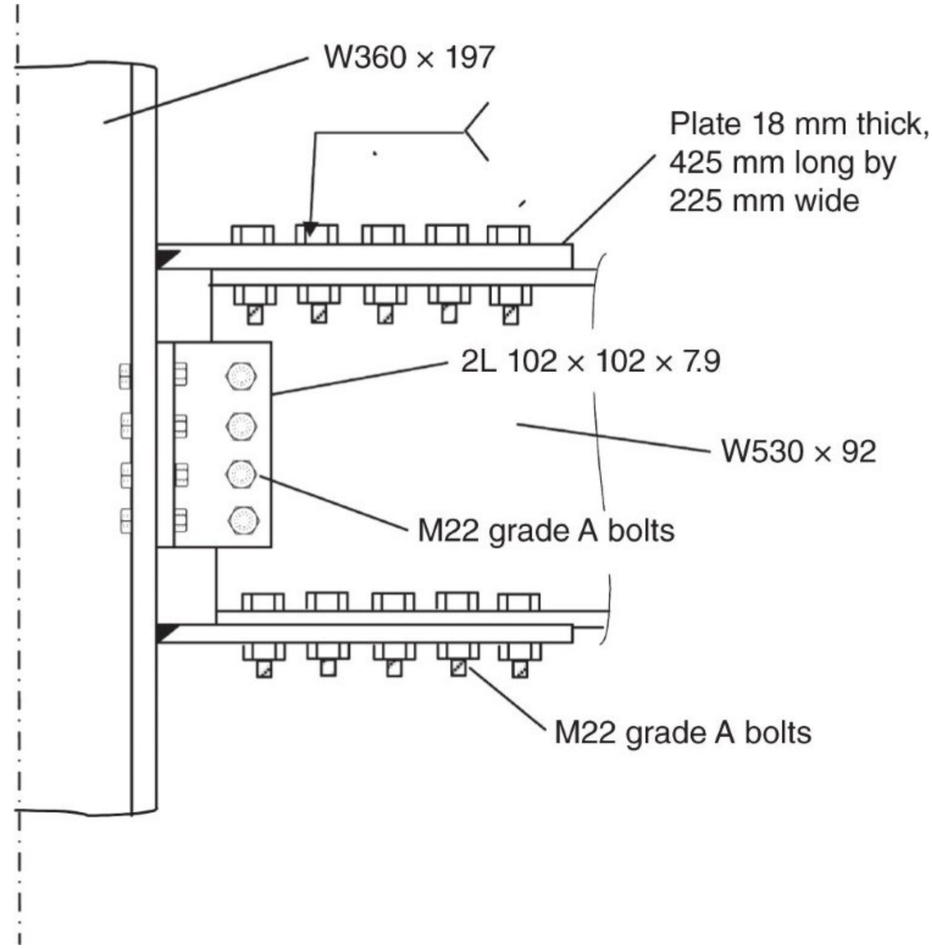

FIGURE 4.10 Final configuration for Example 4.1 connection.

11. Moment and rotation at service: The connection will not slip until the frictional capacity of the bolts (M_{slip}) is reached when the force in the plate reaches 597 kN (slip capacity of the 10 bolts) or a moment of about 318.2 kN-m.

The elastic contributions of the plate and the beam flange up to a distance equal to the beam depth (0.533 m) from the column face will be considered to calculate the rotation. A linear distribution of strains will be assumed, with a moment diagram consistent with a slip stress at the middle row of bolts of 597 kN times $d_{beam}/2$ or 318.2 kN-m. under a uniformly distributed load. The calculations are tedious and are best carried out with the aid of a spreadsheet, resulting in an overall elastic deformation of 0.395 mm, with the plate contributing about 70%.

Assuming that the connection rotates about the center of the beam, the connection rotation is

$$\theta_{conn} = \frac{\Delta_{conn}}{(d/2)} = \frac{0.395 \text{ mm}}{(533 \text{ mm}/2)} = 0.00148 \text{ rad} = 1.48 \text{ mrad}$$

The connection stiffness is

$$k_{\text{conn}} = \frac{M_{\text{slip}}}{\theta_{\text{conn}}} = \frac{(318.2 \text{ kN-m})}{0.00148 \text{ rad}} = 2.15 \times 10^5 \text{ kN-m/rad}$$

The relative stiffness, assuming the beam is 7.5 m long, is

$$\alpha = \frac{k_{\text{conn}} L_{\text{beam}}}{EI_{\text{beam}}} = \frac{(2.15 \times 10^8 \text{ kN-mm})(7500 \text{ mm})}{(200 \text{ GPa})(552 \times 10^6 \text{ mm}^4)} = 14.6 \rightarrow \alpha \leq 20, \text{ therefore PR}$$

At this point the connection will begin to slip. It is reasonable to assume that about 0.5 mm of slip will occur at this point, increasing the rotation to about 3.36 mrad. From there on the connection will reload with a stiffness slightly less than the initial stiffness, as additional slip and bearing deformations will begin to occur.

12. Moment and rotation at yield and ultimate:

$$M_{y,\text{beam}} = F_y S_x = (345 \text{ MPa})(2070 \times 10^3 \text{ mm}^3) = 714.2 \text{ kN-m} \rightarrow \text{gross area}$$

$$M_{u,\text{beam}} = F_y Z_{x,n} = (345 \text{ MPa})(1962 \times 10^3 \text{ mm}^3) = 676.9 \text{ N-m} \rightarrow \text{net area}$$

From before, $M_{u,\text{beam}}$ = 708.6 kN-m → net area

$P_{y,\text{plate}} = F_y A_{n,\text{ plate}}$ = (345 MPa)(250 − 48)(18) = 1254 kN

$M_{y,\text{plate}} = P_{y,\text{plate}} (d + t_{\text{plate}})$ = (1254 kN)(533 + 18)/1000 = 691.2 kN-m

All these values are close to one another so one could select 700 kN-m as a compromise.

$$M_{u,\text{plate}} \approx M_{y,\text{plate}} \left(\frac{F_u}{F_y}\right) = 700 \text{ N-m} \left(\frac{450 \text{ MPa}}{345 \text{ MPa}}\right) \approx 900 \text{ kN-m}$$

Thus the moment is capped by yielding of the net section of the beam flange at the first row of bolts. Yield will begin around (345/450)(700) = 537 kN-m. At this stage, the elastic deformations of the plate and beam are about 0.67 mm and the slip will be, at a minimum, 1 mm. At this point

$$\theta_{\text{conn}} = \frac{\Delta_{\text{conn}}}{(d/2)} = \frac{1.67 \text{ mm}}{(533 \text{ mm}/2)} = 0.0063 \text{ rad}$$

After this point it becomes difficult to compute deformations, but at about 700 kN-m there will be about 0.81 mm of elastic deformation of the plate, 0.42 mm of elastic deformation in the beam flange and probably about 1.5 mm of slip and bearing deformations, totaling about 2.73 mm of total deformation for a rotation of 10.2 mrad. There will be, of course, substantial plastic deformation, so it is likely that this rotation will be at least doubled before the beam reaches its ultimate strength.

The connection itself is actually stronger than the beam, with an ultimate capacity of about 900 kN-m, controlled by the net section but also close to the shear capacity of the bolts. An approximate moment-rotation curve for this joint is shown in Fig. 4.11; this is probably accurate to ± 10%.

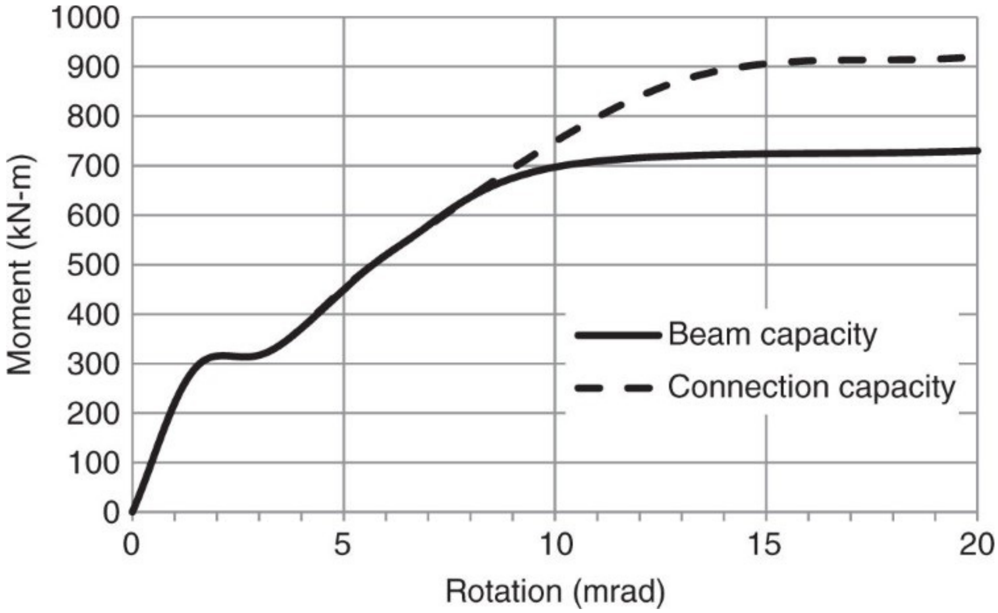

FIGURE 4.11 Approximate moment-rotation curve for joint in Example 4.1.

Figures 4.12 and 4.13 show some typical details and variations proposed by Astaneh-Asl for this type of connections. Figure 4.12 shows a variation where the bottom flange is welded rather than bolted, while Fig. 4.13 shows a connection to the weak axis of the column.

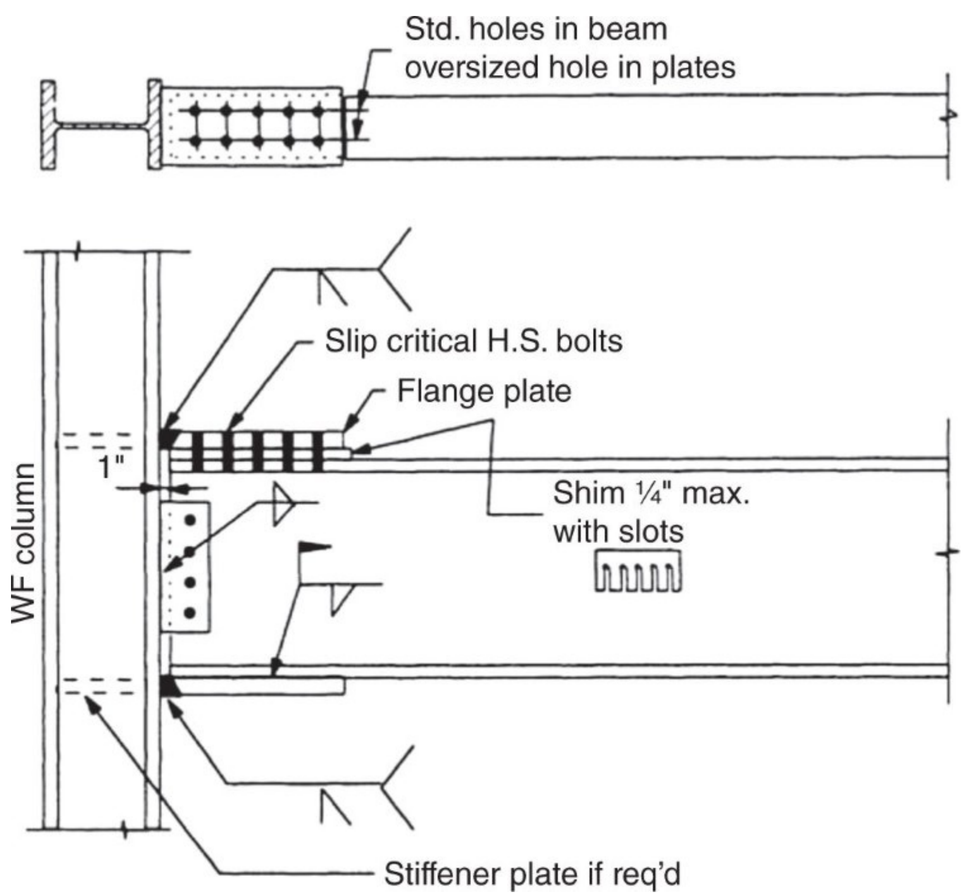

FIGURE 4.12 Typical CW-BB connection at the top and CW-BW connection at bottom (Astaneh-Asl, 1995).

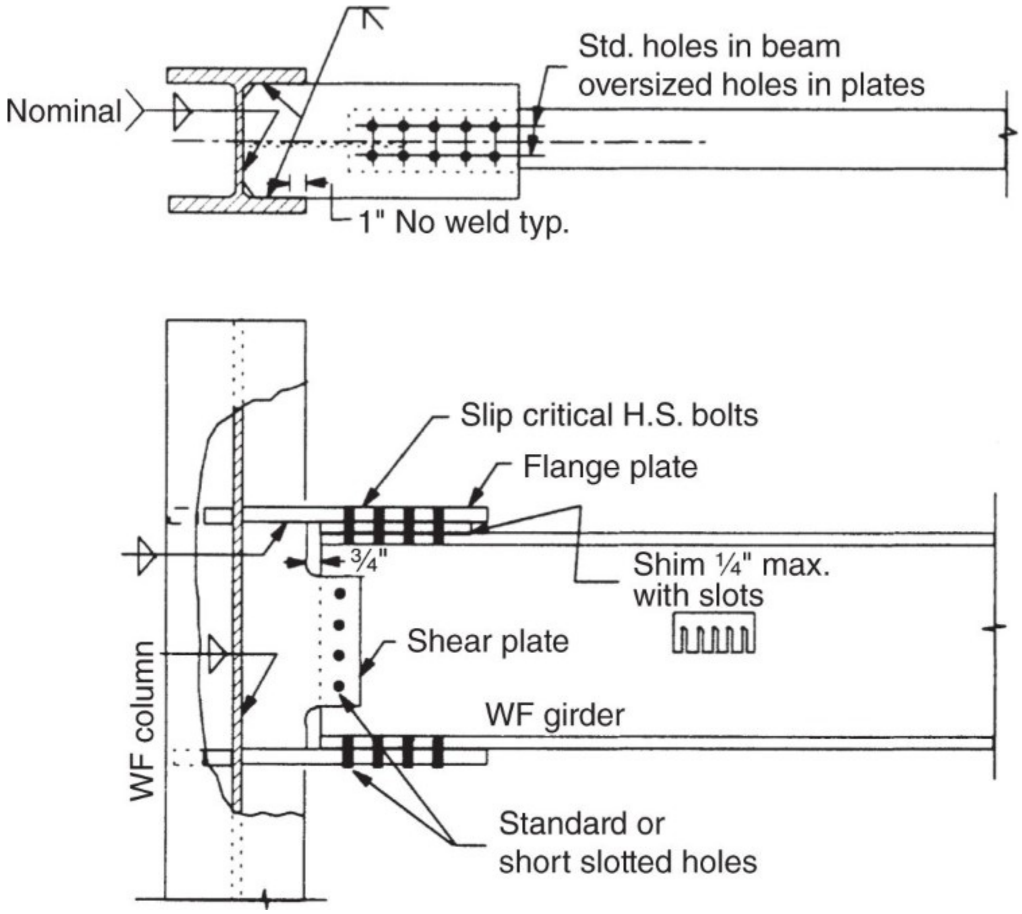

FIGURE 4.13 Typical CW-BB connection to weak axis of the column (Astaneh-Asl, 1995).

It is important to note that in the example above it was assumed that the loads were well known. In Example 4.2, below, it will be shown that while a connection can be designed to connect similar size members in a frame located in a high seismic zone, the requirements can be very different.

Design Example 4.2. Design a full strength connection between a W690 × 170 girder and a W360 × 463 column. Clear span is 8.5 m. The sections satisfy the requirements of AISC A358 for bolted flange plate (BFP) connections. Assume the dead and live load as 11 kN/m each. Both sections are A572 Grade 350. Use Grade B M24 bolts with threads excluded from the shear plane. Design for seismic design category (SDC) D.

1. Determine maximum moment capacity required for the connection design (AISC 358, Eqs. (2.4-1) and (2.4-2)):

$$M_{pr} = C_{pr} Z_x R_y F_y = \left(\frac{(350+450}{2 \times 345}\right)(4550 \times 10^3 \text{ mm}^3)(1.1 \times 350 \text{ MPa}) = 1989 \text{ kN-m}$$

If we assume that all bending forces are transmitted through the beam flange, the force in the beam flange consistent with this moment would be

$$F_{\text{plate}} = \frac{1989 \text{ kN-m}}{0.684 \text{ m}} = 2908 \text{ kN}$$

2. The maximum bolt diameter will be taken as [AISC 358, Eq. (7.6-2M)]:

$$d_b \le \frac{1}{2} b_f \left(1 - \frac{Y_t F_y}{F_u}\right) - 3 = \frac{1}{2}(254)\left(1 - \frac{345}{450}\right) - 3 = 26.6 \text{ mm} \rightarrow \text{Use M24 bolts}$$

Check net area fracture versus gross section yielding of the girder flange by AISC 360, Section F13. Recall that $Y_t = (F_y/F_u) = 0.76 < 0.8$ so $Y_t = 1.0$. For the WW690 × 170, b_f = 254 mm, t_f = 18.9 mm, and d = 684 mm; holes for M24 bolts assumed as 27 mm in diameter per AISC Table J3.3M:

$$F_u A_{fn} = (450)(254 - 2(27))(18.9) = 1701 \text{ kN}$$

$$Y_t F_y A_{fg} = (1.0)(345)(254)(18.9) = 1656 \text{ kN}$$

Thus the gross section governs ($F_u A_{fn} > Y_t F_y A_{fg}$) by these computations, but in reality either of them could control as the two values are very close to one another.

3. Check local buckling, assuming highly ductile members (AISC 341, Table D1.1):

$$\text{Flange:} \quad \lambda_{hd} = \left(\frac{b}{t}\right) = 6.70 \le 0.32\sqrt{\frac{E}{R_y F_y}} = 7.35, \text{ ok}$$

$$\text{Web:} \quad \lambda_{hd} = \left(\frac{h}{t_w}\right) = 49.5 \le 2.57\sqrt{\frac{E}{R_y F_y}} = 59.0, \text{ ok}$$

4. The maximum unbraced length (L_b) for seismic design is (AISC 341, Table D1.1):

$$L_b \le \frac{0.095\, r_y E}{R_y F_y} \approx 50 r_y = 50(53.9 \text{ mm}) = 2.7 \text{ m}$$

5. Estimate number of A490 × M24 bolts required for shear. Note that a 1.25 factor is used here to increase the number of bolts, as the design moment at the column face will be increased by the shear acting at the critical section [AISC 358, Eq. (7.6-4)]:

$$N \ge \frac{1.25 M_{pr}}{f_v (d)(F_v A_b)} = \frac{1.25(1989 \text{ kN-m})(10^6)}{(1.00)(684 \text{ mm})(579 \text{ MPa})(452 \text{ mm})} = 13.9, \text{ say 14 bolts}$$

6. Determine the beam hinge location (S_h). The hinge will be located below the last row of bolts away from the column face. Assuming a bolt spacing(s) of 75 mm, and end distance of 50 mm, and a distance between the first row of holes and the column (S1) of 100 mm, the plate length will be 600 mm and the hinge will be located at S_h by AISC 358, Eq. (7.6.5):

$$S_h = 3S_1 + s\left(\frac{n}{2} - 1\right) = 100 + 75\left(\frac{14}{2} - 1\right) = 5503 \text{ mm}$$

7. Compute the shear in the beam (V_h) at the location of the plastic hinges (Fig. 4.14). The actual distance between the hinges (L_v) is the total centerline distance minus the column depth minus two times the distance to the plastic hinge:

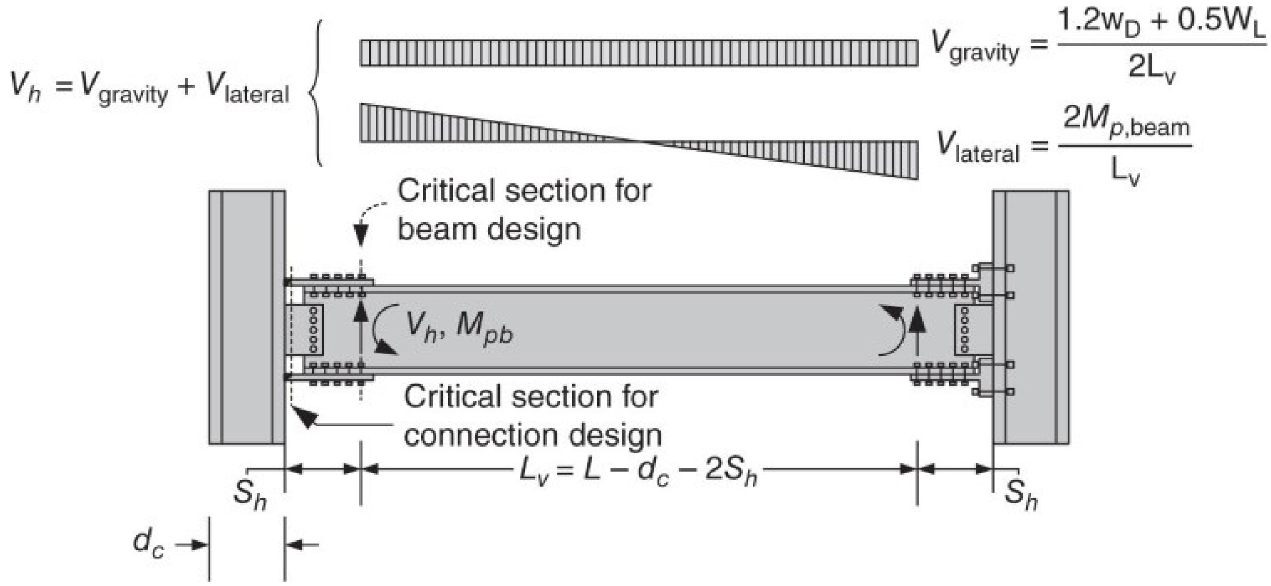

FIGURE 4.14 Increase in moment at connection critical section.

$$L_v = L - d_c - 2S_h = 8500 - 435 - 2(550) = 6965 \text{ mm}$$

The shear will be computed based on assuming a $w_u = 1.2D + 0.5L = 1.2(11) + 0.5(11) = 18.7$ kN/m. Thus, from AISC 358, Eq. (7.6–13):

$$V_h = \frac{2M_{pr}}{L_v} + \frac{w_u L_v}{2} = \frac{(2)(1989)}{6.965} + \frac{(18.7)(6.965)}{2} = 636.3 \text{ kN}$$

8. The actual moment at the face of the column (M_f) is

$$M_f = M_{pr} + V_h S_h = 1989 + (636.3)(0.55) = 2339 \text{ kN-m}$$

9. The actual force on the plate is [AISC, Eq. (7.6–7)]:

$$F_{\text{plate}} = \frac{M_f}{d + t_p} = \frac{2339 \text{ kN-m}}{(0.690 + 0.028)\text{m}} = 3258 \text{ kN}$$

10. Recheck the number of bolts:

$$n \geq \frac{F_{\text{plate}}}{\phi(F_v A_b)} = \frac{3258}{(1.00)(579)(452)} = 12.4 \rightarrow \text{So 14 bolts is ok}$$

11. Determine the size of the flange plate, assuming that the plate width will be somewhere between the widths of the beam flange (254 mm) and the column flange (412 mm). Assume distance between bolts is 150 mm and edge distances are 105 mm. Try a 360 mm plate:

$$t_f = \frac{F_{\text{plate}}}{\phi_d F_y b} = \frac{3419}{(1.0)(345)(360)} = 27.5 \text{ mm} \rightarrow \text{Use 28 mm plate}$$

12. Check neat and gross areas for the plate:

$F_{\text{gross area}} = R_y\, F_y\, A_{\text{gross}} = (1.1)(345)(360 \times 28) = 3825 \text{ kN} > F_{\text{plate}} \rightarrow$ ok

$F_{\text{net area}} = F_u\, A_{\text{net}} = (450)(360 - 2(27)) \times 28) = 3855 \text{ kN} > F_{\text{plate}} \rightarrow$ ok

13. Check block shear on the plate. Assume shear failure along the bolts and tensile failure across bolt gage and $U_{bs} = 1.0$:

$$A_{gv} = 2\,(450 + 50 - 12)(28) = 27328 \text{ mm}^2$$

$$A_{gt} = (125)(28) = 3500 \text{ mm}^2$$

$$A_{nv} = 27328 - 2(6.5 \times 27)(28) = 17500 \text{ mm}^2$$

$$A_{nt} = 3500 - (1 \times 28)(27) = 2744 \text{ mm}^2$$

$$R_n = 0.6\, F_u\, A_{nv} + U_{bs}\, F_u\, A_{nt} \leq 0.6\, F_y\, A_{gv} + U_{bs}\, F_u\, A_{nt}$$

$$R_n = 0.6(450)(17500) + (1.0)(450)(2744) \leq 0.6(345)(27328) + (1.0)(450)(2744)$$

$$R_n = 5961 \text{ kN} \leq 6892 \text{ kN}$$

$$\phi R_n = (0.75)(5961 \text{ kips}) = 4470 \text{ kN} > F_{\text{plate}} = 3419 \text{ kN} \rightarrow \text{ok}$$

$$R_{u,\text{slip}} = f\,\mu\, D_u\, h_f\, T_b n_s \text{ [AISC 360, Eq. (J3-4)]}$$

$$T_b = 257 \text{(minimum bolt pretension, AISC Table J3.1M)}$$

$$R_{u,\text{slip}} = f\,\mu\, D_u\, h_f\, T_b \text{n}_s = (1.00)(0.3)(1.13)(1)(257)(1) = 87.1 \text{ kN}$$

$$M_{\text{slip}} = (14 \text{ bolts})(87.1 \text{ kN/bolt})(690 \text{ mm}) = 841.6 \text{ kN-m}$$

14. Detail the shear connection to the web: The design of the shear connection for this case will not be carried out in detail here (see Chap. 2 for design of shear connections). From the AISC Manual, a pair of 10-mm angles with five M27.F3125 bolts provide adequate shear resistance.

15. Check connection stiffness: For an initial stiffness, check at the slip level. Following the same approach as for Example 4.1, the elastic elongation at the slip level is 0.768 mm, leading to a rotation at the beginning of slip of

$$\theta_{\text{conn}} = \frac{\Delta_{\text{conn}}}{(d/2)} = \frac{0.768 \text{ mm}}{(690 \text{ mm}/2)} = 0.00223 \text{ rad } = 2.23 \text{ mrad}$$

The connection stiffness is

$$k_{\text{conn}} = \frac{M_{\text{slip}}}{\theta_{\text{conn}}} = \frac{(841.6 \text{ kN-m})}{0.00223 \text{ rad}} = 3.78\times10^5 \text{ kN-m/rad}$$

The relative stiffness, assuming the beam is 8.0 m long, is

$$\alpha = \frac{K_{\text{conn}} L_{\text{beam}}}{EI_{\text{beam}}} = \frac{(3.78\times10^8 \text{ kN-mm})(8000 \text{ mm})}{(200 \text{ GPa})(1700\times10^6 \text{ mm}^4)} = 8.9 \rightarrow \alpha \le 20, \text{ therefore PR}$$

Note that this will put this connection in the middle or lower portion of the FR range. This may appear to contradict the assumption used in AISC358 that consider this to be a FR connection. There are at least two reasons that this is not alarming. First, the calculations above can only be considered as an estimate as it is clear that slip is the main contributor to the deformation. Second, AISC358 is more focused on strength and ductility rather than stiffness, so it is likely that under a typical ground motion, frame stability, which is where the PR classification comes in, will not be an issue.

The rest of the checks should proceed as for Example 4.1, with additional checks for the column for (1) continuity plates (likely to be needed), (2) doubler plates (unlikely), and beam-to-column moment ratio. For the beam, additional checks for block shear and shear connection demand should be performed.

4.3.2 Column Bolted-Beam Bolted (T Stubs)

Bolted T-stub connections were a popular connection in moment-resisting frames before field welded connections became economical, and along with end-plate connections, still represent the most efficient kind of column-bolted-beam bolted (CB-BB) connection. The mechanistic model for this type of connection is shown in Fig. 4.5, while the possible yield and failure modes are shown in Fig. 4.8. The important conceptual difference between a CW-BB and a CB-BB is that for T stubs the springs that represent the connection to the column flange have lower strength and stiffness. This is because they represent the flexural deformations that can take place in the flanges of the tee as well as any axial deformation of the bolts to the column flange. Both of these are flexible when compared to the axial stiffness of a weld, which can be considered to be an almost rigid element. In addition, for the CB-BB connections, the spring representing the bolts needs to include the prying action, which can significantly increase the force in the bolts at ultimate. Figure 4.15 shows prying action in a very flexible T stub. In this case the flexibility of the flange of the stub results in an additional prying force (Q) at the tip of the stub flange. This force increases the nominal force in the bolts above its nominal pretension value (T).

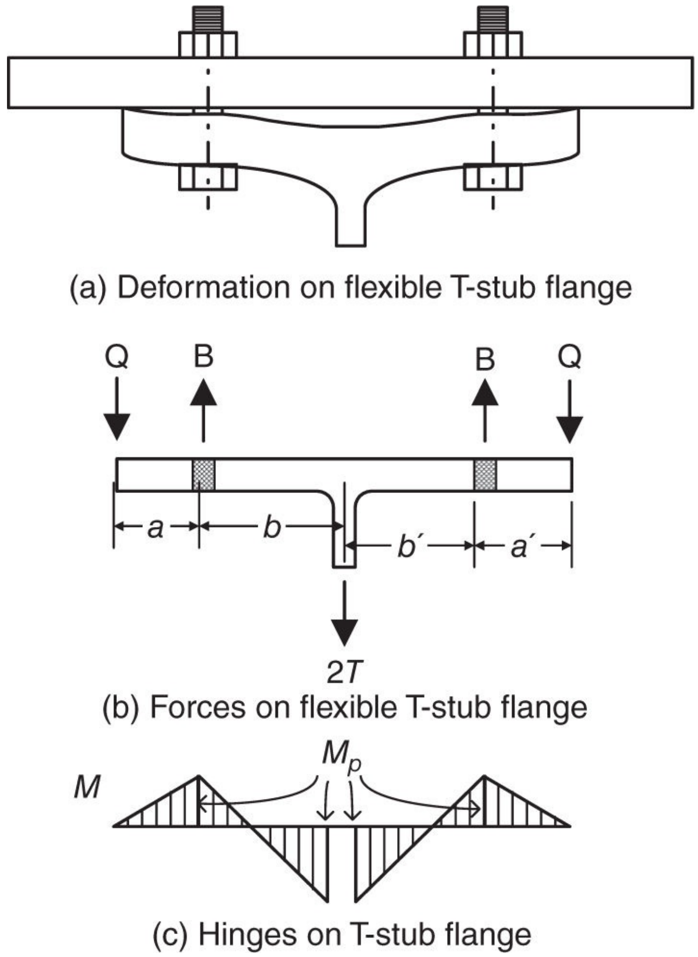

FIGURE 4.15 Prying action in T stub, showing the case of a flexible flange.

For the case of the T stub, the springs shown in Fig. 4.5 can have a wide range of strength and stiffnesses, depending primarily on the thickness of the flanges and the location and size of the bolts to the column. The big advantage of this type of connection over a CW-BB one is that these springs can provide a much larger deformation capacity than a weld would. A T-stub connection can thus provide a good balance between strength, stiffness, and ductility.

The design of a T-stub connection essentially follows the same steps as for the CW-BB connections described above for the stem portion of the connection, with important additional design provisions for prying action, bolt tensile elongation capacity, local effects on the column flange, and bolt shear strength. The strength of the connection to the column, taking into account prying action, is limited by

- *The bending strength of the flanges of the T:* This depends primarily on the thickness of the flanges and the exact location of the bolt holes.
- *The ultimate tensile strength of the stem of the T:* The net area generally governs over the gross area criteria because the width of the stem at the critical section for net area is not too different from that of the critical section for gross area.
- *The tensile strength of the bolts:* This is influenced primarily by the prying action.
- *The shear strength of the bolts:* It is difficult to fit more than 8 to 10 bolts in the stem of a conventional T (cut form a W shape) and thus large bolts may be needed.

Each of these failure modes must be checked individually and the lowest strength taken as the controlling value. Guidelines for these calculations are given in the AISC Specification (AISC, 2016), textbooks (Salmon et al., 2009), and the standard references (Kulak et al., 1987). An excellent review of the design, including some of the numerical problems that can be encountered, is given by Thorton (1985). In this chapter, Example 4.3 is based on the work of Swanson and Leon (2000, 2001), while Example 4.4 is based on unpublished work by Swanson, Rassati, and Leon for AISC 358.

For seismic design, the effect of reversed cyclic loading on these connections is to progressively decrease the tension in the bolts to the column flange. Because of prying action, the stress range in these bolts is probably significantly larger than that calculated based on the simplified models used for design. This can result in either low cycle fatigue failures or in fracture of the bolt due to excessive elongation.

Currently, AISC only allows the use of T stubs cut from rolled sections. However, there has recently been quite a bit of research into using built-up (or welded) T sections; the preliminary results show very promising behavior (Hantouche et al., 2012, 2013). Use of welded T stubs will considerably simplify the design as the design results in proportions that sometimes are difficult to obtain from T stubs cut from rolled sections.

Design Example 4.3. A PR connection is to be designed to transfer a factored moment of 315 kN-m and a factored shear of 450 kN kip from a W530 × 85 beam to the flange of a W36 × 122 column. The connection consists of tee sections for moment transfer and web angles for shear transfer. All materials are 250 MPa steel, with an ultimate strength of 400 MPa. Bolts are to be M24 Grade B bolts with threads excluded from the shear planes. Seismic design is not required.

1. If all bending moment is carried by the tees, the axial force on the stem (P_T) is

$$P_T = \frac{M_u}{\theta d} = \frac{315 \text{ kN-m}}{(0.9)(0.536)} = 621.7 \text{ kN} = 2T \rightarrow T = 310.8 \text{ kN}$$

2. Determine the minimum number of bolts (N) required to carry the tensile force to the column flange. Ignore the prying forces for now and check later.

$$R_u = \phi R_{nt} = \phi F_{nt} A_b \text{[AISC 360, Eq. (J3-1b)]}$$

$$R_u = (0.75)(620)(452) = 210.2 \text{ kN}$$

$$N = \left(\frac{P_T}{R_u}\right) = \left(\frac{621.7 \text{ kN}}{210.2 \text{ kN}}\right) = 2.96 \rightarrow \text{use } N = 4 \text{ bolts}$$

Note that because prying forces can be large in this type of connection, it is best to have a very conservative number of bolts to the column flange. This check is used here mostly to ensure that a reasonable number of bolts are needed (i.e., 4 or 8 bolts rather than more which would be hard to accommodate).

Determine the number of bolts (M) required to transmit the forces from the stem to the beam flanges through shear (bolts are in single shear):

3. Check minimum stem thickness (t_s) so that bearing does not govern:

$$R_u = \phi R_{nt} = \phi F_{nt} A_b \text{ [AISC 360, Eq. (J3-1b)]}$$

$$R_u = (0.75)(457)(452) = 154.9 \text{ kN}$$

$$N = \left(\frac{P_T}{R_u}\right) = \left(\frac{621.7 \text{ kN}}{154.9 \text{ kN}}\right) = 4.01 \rightarrow \text{use } N = 6 \text{ bolts}$$

$$F_{\text{bearing}} \geq 6217 \text{ kN} = \phi N(2.4\ dt_p F_u) \text{ [AISC 36, Eq. (J3-6a)]}$$

$$t_s \geq \frac{F_{\text{bearing}}}{\phi N(2.4\ dF_u)} = \frac{621.7 \text{ kN}}{(0.75)(6)(2.4)(24)(450)} = 5.33 \text{ mm}$$

This thickness is small and not likely to govern.

4. Determine the stem thickness (t_p) required to transmit tension on the stem of the tee. Assume plate width (w) at critical section is about 225 mm (total column flange width is 257 mm.):

Gross area:

$$t_{s,g} \geq \frac{P_T}{\phi b_p F_y} = \frac{621.7 \text{ kN}}{(0.9)(225 \text{ mm})(250 \text{ MPa})} = 12.2 \text{ mm}$$

Net area (assuming holes are 26 mm):

$$t_{s,g} \geq \frac{P_T}{\phi b_p F_y} = \frac{621.7 \text{ kN}}{(0.75)(225 - (2 \times 26)\text{mm})(400 \text{ MPa})} = 12.0 \text{ mm}$$

Therefore a web rolled section with a web thickness greater than 12.2 mm is needed.

Determine the flange thickness (t_f) for the tee section. This needs to take prying action into account. A simplified mechanism for computing the additional forces due to prying action is shown in Fig. 4.15*b*. The prying forces (Q) arise from the additional forces developed at the end of the T flanges as the T stub is pulled. Assuming that each side of the flange can be modelled as a two-span beam with one end fixed (at the web) and one end free to rotate (edge of T stub), the maximum forces can be calculated based on the formation of plastic hinges at both the web and the edge of the bolt. For details see Salmon et al. (2009), pp. 709–717. From this type of model, an equation for the required plate thickness can be derived. One such equation is that proposed by (Thornton, 1994):

$$t_t \geq \sqrt{\frac{4Tb'}{\phi w F_y (1 + \alpha\delta)}}$$

where T = force in each bolt line or half the force in T stem

b' = distance from the web centerline to the inside edge of the bolt

a' = distance from inside edge of the bolt to edge of T stub

w = tributary width of the flange

F_y = yield strength of the T stub

$\phi = 0.9$

α, δ, β = constants as defined below

To minimize prying action, the edge distance (a) should be minimized.

For a M24 this is 30 mm, so

$$a' = a + (d_{\text{hole}}/2) = 30 + 13 = 43 \text{ mm} \rightarrow \text{assume about 50 mm}$$

$$B_b = R_{u,t} = 210.2 \text{ kN}$$

$$T_b = \frac{P_T}{N} = \frac{621.7}{4} = 155.4 \text{ kN}$$

$$\beta = \left(\frac{B}{T} - 1\right)\frac{a'}{b'} = \left(\frac{210.2 \text{ kN}}{155.4 \text{ kN}} - 1\right)\left(\frac{50 \text{ mm}}{60 \text{ mm}}\right) = 0.294$$

$\delta = 0.75$ (an estimate, since the section has not been chosen)

$$\beta < 1 \rightarrow \text{use } \alpha = \min\left(\left[\left(\frac{1}{0.75}\right)\left(\frac{0.294}{1-0.294}\right) = 0.555\right] 1\right) \rightarrow \alpha = 1.0$$

$w = 225$ mm (assumed)

Salmon and Johnson, following Thorton, recommend to compute β as a function of α and δ, where

$$\beta = \left(\frac{B}{T_b} - 1\right)\frac{a'}{b'}$$

If $\beta \geq 1 \rightarrow$ use $\alpha = 1 \rightarrow$ indicates large prying forces exist

If $\beta < 1 \rightarrow \text{use } \alpha = \min\left(\left[\left(\frac{1}{\delta}\right)\left(\frac{\beta}{1-\beta}\right)\right] 1\right) \rightarrow$ indicates moderate prying forces exist

$$\alpha = \frac{M_1}{\delta M_2}$$

where δ = ratio of net area at bolt line to gross area at M_1

B = maximum bolt resistance, usually taken as the bolt tensile capacity

T_b = tensile force in each bolt

For our example, assuming a gage (g) is about 150 mm and noticing that the distances a and b are unknown at this point:

$$b=\left(\frac{g}{2}-\frac{t_w}{2}\right)=\left(\frac{150}{2}-\frac{14}{2}\right)=68\text{ mm}$$

$$b'=b-\frac{d}{2}=68-\frac{27}{2}=54.5\text{ mm}\rightarrow\text{assume } b=60\text{ m}$$

$$t_t\geq\sqrt{\frac{4Tb'}{\phi wF_y(1+\alpha\delta)}}=\sqrt{\frac{4(310.8)(30)(1000)}{(0.9)(225)(250)(1+(0.555)(0.75))}}=22.8\text{ mm}$$

In summary, we want a *WT* with a $t_w > 12.2$ mm and a $t_f > 22.8$ mm. In addition we want to minimize b_f so as to control prying and the stem must be able to accommodate three lines of bolts. If we assume that the section will have a k_{des} of about 35 mm, a bolt spacing of 80 mm, and end distances of 30 mm, a minimum stem depth of about 250 mm will be required.

Try a WT 265 × 69, with $t_f = 23.6$ mm, $b_f = 214$ mm, $t_w = 14.7$ mm, $b_f = 214$ mm, and $d = 274$ mm.

5. Check the prying force using the formula proposed by Salmon and Johnson:

$$Q\geq T\left(\frac{\alpha\delta}{1+\alpha\delta}\right)\left(\frac{a'}{b'}\right)$$

$$b_f=2(a'+b')=214\text{ mm}\rightarrow\text{keep } a<1.25b,\text{ so use } b=48\text{ mm and } a=59\text{ mm}$$

$$\delta=\frac{w-2d_h}{w}=\frac{225-(2\times 27)}{225}=0.76$$

$$\beta=\left(\frac{B}{T}-1\right)\frac{a'}{b'}=\left(\frac{210.2\text{ kN}}{155.4\text{ kN}}-1\right)\left(\frac{59\text{ mm}}{48\text{ mm}}\right)=0.433$$

$$\beta<1\rightarrow\text{use } \alpha=\min\left(\left[\left(\frac{1}{0.76}\right)\left(\frac{0.433}{1-0.433}\right)=1.01\right]1\right)\rightarrow\alpha=1.0$$

$$Q\geq T\left(\frac{(0.76)(1.0)}{1+[(1.0)(0.76)]}\right)\left(\frac{48\text{ mm}}{59\text{ mm}}\right)=0.35T$$

$$T+Q=(1+0.35)T=1.35(155.4)=209.9\text{ kN}\leq\phi R_u=210.2\text{ kN},\rightarrow\text{ok}$$

Design is satisfactory; use WT12 × 47 to carry tensile and compression forces.

6. Design an angle for shear transfer:

Check shear capacity:

$$N=\frac{V_u}{\phi R_u}=\frac{500\text{ kN}}{154.9\text{ kN}}=3.2\rightarrow\text{Use 4 bolts}$$

Check bearing on angles:

$$t_a \geq \frac{(V_u/2)}{\phi N(2.4 d_b F_u)} = \frac{250\text{ kN}(1000)}{(0.75)(4)(2.4)(22\text{ mm})(400\text{ MPa})} = 7.89\text{ mm} \rightarrow \text{Use 8 mm}$$

Check angle length for net section:

$$(1_a - N d_h) \geq \frac{V_u}{\phi t_a F_u} \rightarrow 1_a \geq \frac{V_u}{\phi t_a F_u} + N d_h$$

$$1_a = \frac{500\text{ kN}(1000)}{(0.75)(8\text{ mm})(400\text{ MPa})} + (3)(24\text{ mm}) = 280\text{ mm} \rightarrow \text{Use 300 mm}$$

Check angle length for gross section capacity:

$$\phi V_n = \phi t_a \mathrm{I}_a F_y = (0.9)(8\text{ mm})(300\text{ mm})(250\text{ MPa})/1000 = 540\text{ kN} > V_u = 500\text{ kN, ok}$$

Check bearing on beam web:

$$\phi R_u = \phi N(2.4 t_w d_b F_u = (0.75)(4)(2.4)(10.3)(22)(400) = 652.6\text{ kN} > V_u = 500\text{ kN, ok}$$

Use a 102 × 102 × 9.5 angle 300 mm long with four A22 Grade A bolts for the shear connection.

7. Check if stiffeners in column are required:

To avoid stiffeners, the column web must be checked for

a. Compression zone:

(1) Local web yielding:

$$\phi R_n = F_{yw} t_w (5k + 1_b)\ \text{[AISC 360, Eq. (J10-2)]}$$

$$\phi R_n = (1.00)(250\text{ MPa})(13\text{ mm})[5(36\text{ mm}) + 16.5\text{ mm}/1000 = 638.6\text{ kN} > P_T\text{, ok}$$

(2) Web crippling:

$$\phi R_n = \phi 0.80 t_w^2 \left[1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E F_{yw} t_f}{t_w}} \quad \text{[AISC 360, Eq. (J10-4)]}$$

$$\phi R_n = \frac{(0.75)\,0.80(13)^2}{1000}\left[1 + 3\left(\frac{16.5}{363}\right)\left(\frac{13}{21.7}\right)^{1.5}\right]\sqrt{\frac{(200{,}000)(250)(21.7)}{(13)}}$$

$$\phi R_n = 981.9\text{ kN} \gg P_T\text{, ok}$$

(3) Compression buckling of the web:

$$\phi R_n = \phi\left(\frac{24t_w^3\sqrt{EF_{yw}}}{h}\right) \text{ [AISC 360, Eq. (J10-8)]}$$

$$\phi R_n = (0.9)\left(\frac{24(13)^3\sqrt{200{,}000(250)}}{280(1000)}\right) = 1198 \text{ kN} \gg P_T\text{, ok}$$

b. Tension zone:

(1) K1.2—Local flange bending:

$$\phi R_n = \phi 6.25 F_{yf} t_f^2 \text{ [AISC 360, Eq. (J10-1)]}$$

$$\phi R_n = (0.9)(6.25)(250)(21.7)^2/1000 = 662.2 \text{ kN} > P_T\text{, ok}$$

$$P_{bf} = \phi_b 6.25 t_{fc}^2 F_{yc} = 0.9 \times 6.25(0.855)^2 \times 36 = 148.0 \text{ kips}$$

Thus no stiffener are required. The final design is shown in Fig. 4.16.

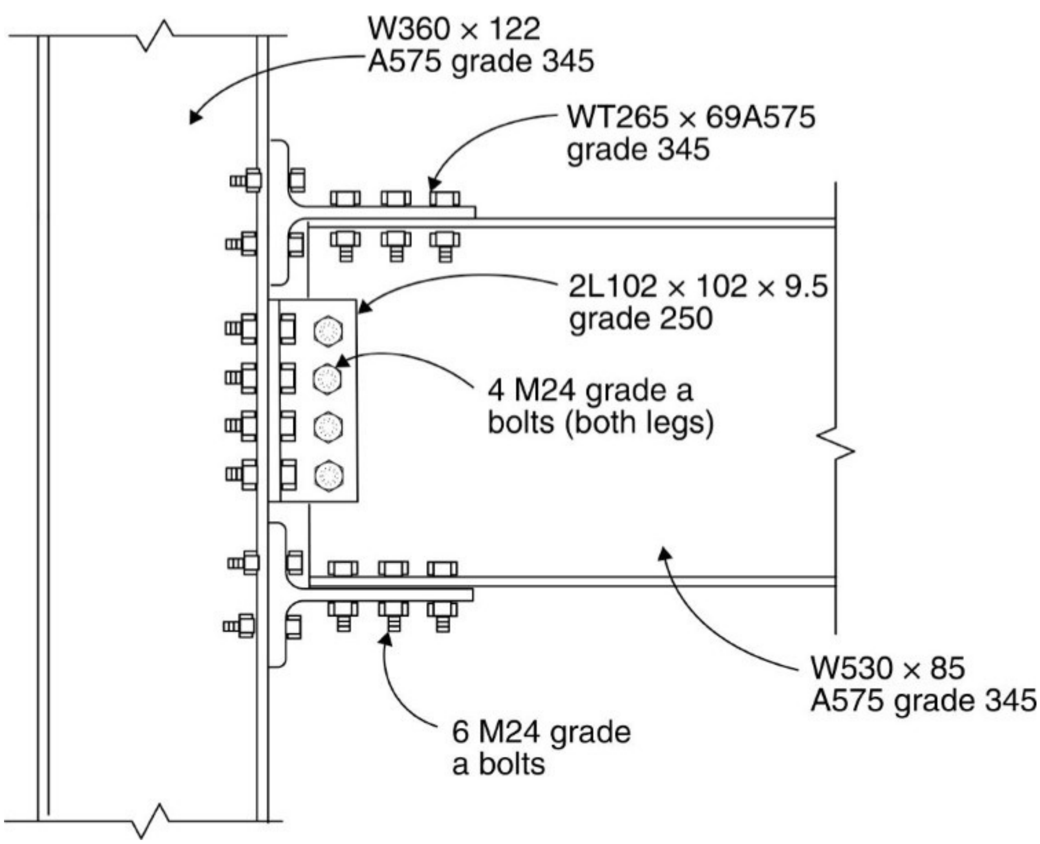

FIGURE 4.16 Final design for Example 4.3.

Design Example 4.4. Design a T-stub connection between a W530 × 85 beam and a W36 × 122 column for a special moment frame in a high seismic zone. All materials are 345-MPa steel. Use M24 Grade B bolts with threads excluded from the shear planes. Assume the dead and live load as 11 kN/m each and the clear span is 7 m. Note that the W530 × 85 section slightly exceeds the 82-kg/m limit in AISC 358. See Fig. 4.17 for definition of some dimensions.

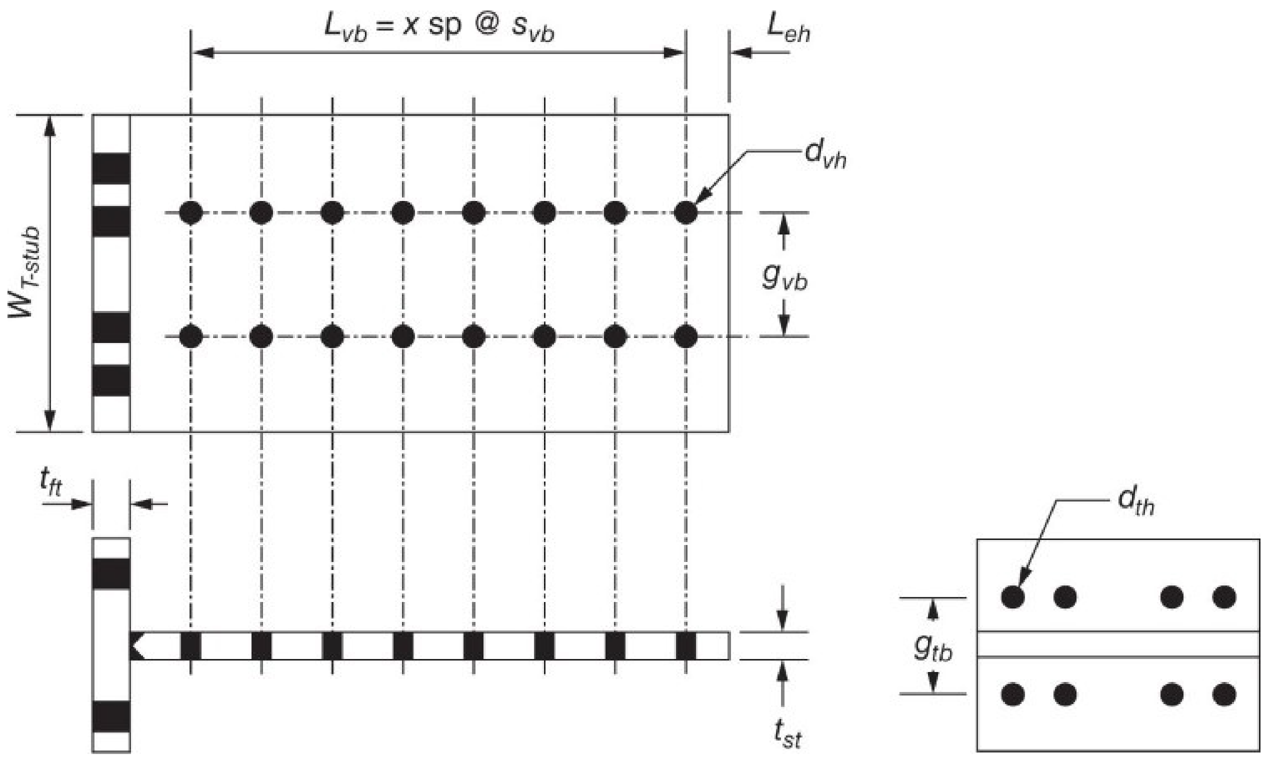

FIGURE 4.17 Nomenclature for T-stub design in Example 4.4.

1. Check local buckling, assuming highly ductile members (AISC 341, Table D1.1):

Flange: $$\lambda_{hd} = \left(\frac{b}{t}\right) = 5.03 \le 0.32\sqrt{\frac{E}{R_y F_y}} = 7.35, \text{ ok}$$

Web: $$\lambda_{hd} = \left(\frac{h}{t_w}\right) = 46.3 \le 2.57\sqrt{\frac{E}{R_y F_y}} = 59.0, \text{ ok}$$

2. Determine maximum moment required for design [AISC 358, Eqs. (2.4-1) and (2.4-2)]:

$$M_{pr} = C_{pr} Z_x R_y F_y = \left(\frac{(345+450)}{2\times 345}\right)(2100\times 10^3 \text{ mm}^3)(1.1\times 345 \text{ MPa}) = 918.2 \text{ kN-m}$$

If we assume that all bending forces are transmitted through the beam flange, the force in the beam flange consistent with this moment would be

$$F_{\text{flange}} = \frac{M_{pr}}{(d-t_f)} \frac{918.2 \text{ kN-m}}{(0.535-0.0165)\text{m}} = 1770 \text{ kN}$$

As a matter of interest, note that this flange force substantially exceeds the expected strength of the flange given by

$$R_t F_u A_{fn} = (1.1)(450)[166-2(27)](16.5) = 914.7 \text{ kN}$$

$$R_y F_y A_{fg} = (1.1)(345)(166)(16.5) = 1039 \text{ kN}$$

The difference comes from the fact that for the W530 × 85 the flanges only carry about 70% of the plastic moment and the effect of the additional C_{pr} factor. Thus the design will be expected to result in performance similar to that shown in Fig. 4.1, in which extensive plastic hinging in the beam is seen but little or no yielding occurs in the connection.

3. The maximum bolt diameter for shear (d_{bs}) will be taken as [AISC 358, Eq. (13.6-3M)]:

$$d_{bs} \le \left(\frac{Z_x}{2t_{fb}(d-t_{fb})}\right)\left(1-\frac{R_y F_{yb}}{R_t F_{ub}}\right)-3$$

$$d_{bs} = \left(\frac{2100}{2(16.5)(535-16.5)}\right)\left(1-\frac{(1.1)345}{(1.1)450}\right)-3 = 25.6 \text{ mm} \rightarrow \text{ Use M24 bolts}$$

4. Estimate number of A490 × M24 bolts required for shear:

Bolt shear:

$$\phi_n r_n = \phi_n F_{nv} A_{vb} = (0.9)(579)(452)/1000 = 235.5 \text{ kN}$$

Bearing on beam flange:

$$\phi_d r_n = \phi_d (2.4 d_b t_{fb} F_u) = (1.00)(2.4)(24)(16.5)(450)/1000 = 427.8 \text{ kN}$$

Bearing on T-stem:

$\phi_d r_n = \phi_d (2.4 d_b t_{st} F_u)$ → the thickness of the stem (t_{st}) is unknown; assume it will not control and recheck later

The number of bolts in shear (N_{sb}) required is

$$N_{sb} \ge \frac{1.25 M_{pr}}{\phi_n r_{n,\min}} = \frac{1.25(918.2 \text{ kN-m})}{235.5 \text{ kN}} = 4.9 \rightarrow N_{sb} = 6 \text{ bolts}$$

5. Determine the beam hinge location (S_h) and stem length (L_{stem}). The hinge will be located below the last row of bolts away from the column face. Assuming a bolt spacing of 80 mm, an end distance of 45 mm, and a distance between the first row of holes and the column (S_1) of 100 mm, the plate length will be 600 mm and the hinge will be located at S_h by AISC 358, Eq. (7.6.5):

$$S_h = S_1 + s\left(\frac{n}{2}-1\right) = 100 + 80\left(\frac{6}{2}-1\right) = 260 \text{ mm}$$

$$L_{\text{stem}} = K_{\text{des}} + 260 + 50 = K_{\text{des}} + 310 \text{ mm}$$

6. Compute the shear in the beam (V_h) at the location of the plastic hinges (Fig. 4.14). The actual distance between the hinges (L_v) is the total centerline distance minus the column depth minus two times the distance to the plastic hinge:

$$L_v = L - d_c - 2S_h = 7000 - 365 - 2(260) = 6115 \text{ mm}$$

The shear will be computed based on assuming a $w_u = 1.2D + 0.5L = 1.2(11) + 0.5(11) =$ 18.7 kN/m. Thus, from AISC 358, Eq. (7.6-13):

$$V_h = \frac{2M_{pr}}{L_v} + \frac{w_u L_v}{2} = \frac{(2)(918.2)}{6.115} + \frac{(18.7)(6.115)}{2} = 357.5 \text{ kN}$$

7. The actual moment at the face of the column (M_f) is

$$M_f = M_{pr} + V_h\, S_h = 918.2 + (357.4)(0.26) = 1011 \text{ kN-m}$$

8. The actual force on the plate is [AISC, Eq. (7.6-7)]:

$$F_{pr} = \frac{M_f}{1.05_d} = \frac{1011 \text{ kN-m}}{1.05(0.535)\text{m}} = 1800 \text{ kN}$$

9. Determine the width of the stem (W_T), assuming that the stem width will vary between something larger than the width of the beam flange at the beam end and somewhat less than the column flange width at the last row of bolts. Assume the gage between bolts (g_{vb}) is 100 mm and the length of the shear bolt pattern is 160 mm. The Whitmore width is

$$W_{\text{whit}} = 2L_{vb} \sin 30° + g_{vb} = 2(160)(0.5) + 100 = 260 \text{ mm}$$

The Whitmore width is between that of the beam flange (166 mm) and the column flange (363 mm). Use $W_t = 250$ mm

10. Determine thickness of stem (t_s) from gross and net area requirements:

$$t_s = \frac{F_{pr}}{\phi F_y W_T} = \frac{1800 \text{ kN}}{(0.9)(345)(250)} = 23.1 \text{ mm} \rightarrow \text{Use 24 mm}$$

$$t_s = \frac{F_{pr}}{\phi F_u W_T} = \frac{1800 \text{ kN}}{(0.75)(450)(250)} 21.3 \text{ mm}$$

11. Check that the plate will not buckle in compression between the last row of bolts and the face of the column ($S_j - t_{ft}$), previously assumed as 100 mm:

$$t_{st} \geq \frac{S_j - t_{ft}}{9.6} = \frac{100}{9.6} = 10.4 \text{ mm, ok}$$

12. Determine the number of tension bolts (N_{tb}) required. Assuming four bolts, the minimum bolt diameter in the absence of prying is given by

$$d_{tb} \geq \sqrt{\frac{F_{pr}}{N_{tb}\phi_n F_{nt}\left(\frac{\pi}{4}\right)}} = \sqrt{\frac{4(1800)}{(4)(0.9)(780)\pi}} = 28.5 \text{ mm} \rightarrow \text{Use M30 bolts}$$

$$\phi r_{nt} = \phi_n A_b F_{nt} = (0.9)(706)(780) = 495.6 \text{ N}$$

$$\phi R_{nt} = \mathrm{N}_{tb}\phi r_{nt} = 1985 \text{ kN} > F_{pr}, \text{ ok}$$

13. Determine preliminary size of the T stub:

The width can be estimated based on a gage of 100 mm and a minimum end distance of 1.5*d*:

$$a = 1.25 d_{tb} = 1.25(30) = 45 \text{ mm}$$

$$b_{ft} \geq g_{tb} + 2a = 150 + 2(45) = 240 \text{ mm}$$

$$b \approx 0.4\, g_{tb} = 60 \text{ mm}$$

$$T_{\text{req}} = \frac{F_{pr}}{4} = \frac{1800 \text{ kN}}{4} = 450 \text{ kN}$$

The force per bolt is

Following the nomenclature in AISC 358, Eq. (13.6-21):

$$p = \frac{2W_T}{n_{tb}} = \frac{2(250)}{4} = 125 \text{ mm/bolt}$$

$$a' = a + \frac{d}{2} = 45 + 15 = 60 \text{ mm}$$

$$b' = b - \frac{d}{2} = 60 - 15 = 45 \text{ mm}$$

$$t_{ft} \geq 2\sqrt{\frac{T_{\text{req}}(a' + b') - \phi r_{nt} a'}{\phi_d d F_{yt} p}} = 2\sqrt{\frac{450(105) - (495)(60)}{(1.00)(345)(125)}} = 40.3 \text{ mm}$$

The WT selected must have a $t_f > 40$ mm, $t_s = t_w > 24$ mm and $d > 260$ mm. Try a WT305.5 × 170.5 with $t_f = 43.9$, $t_w = 23.8$ mm, $b_f = 333$ mm and $d = 330$ mm.

14. Determine adequacy of the design, starting with the flange. Assuming the gage of the tension bolts (g_{tb}) as 230 mm in order to avoid clearance problems:

$$t_{st,\text{eff}} = t_{st} + 2k_1 = 23.6 + 2(56.6) = 136.8 \text{ mm}$$

$$b = \left(\frac{1}{2}\right)(g_{tb} - t_{s,\text{eff}}) = \left(\frac{1}{2}\right)(230 - 136.8) = 46.6 \text{ mm}$$

$$b' = b - \frac{1}{2}d_{bt} = 46.5 - 15 = 31.5 \text{ mm}$$

$$a = 1\tfrac{1}{2}d_{bt} = 45.0 \text{ mm} \le 1.25b = 1.25(46.6) = 58.3 \text{ mm, ok}$$

$$a' = a + \frac{1}{2}d_{bt} = 45 + 15 = 60 \text{ mm}$$

$$b_{ft} = g_{tb} + 2a = 230 + 2(45) = 320 \text{ mm} \approx b_f = 333 \text{ mm}$$

$$\text{Let } a = 51 \text{ mm}, a' = 66 \text{ mm}, b = 48 \text{ mm and } b = 33 \text{ mm, ok}$$

Three mechanisms can be postulated:

a. For a pure plastic mechanism in the tension flange, the required design resistance per tension bolt is

$$\phi T_1 = \frac{(1+\delta)}{4b'} p\phi_d F_y t_{ft}^2$$

$$\delta = \left(1 - \frac{d_{tht}}{p}\right) = \left(1 - \frac{33}{125}\right) = 0.736$$

$$\phi T_1 = \frac{(1+0.736)}{(4)(33)}(125)(1.00)(345)(43.9)^2 = 1093 \text{ kN/bolt}$$

b. For a mixed failure mode, with a plastic mechanism followed by fracture of the bolts is

$$\phi T_2 = \frac{\phi_d r_{nt} a'}{a' + b'} + \frac{\phi_d p F_y t_{ft}^2}{4(a' + b')} \ge T_{\text{reqd}}$$

$$\phi T_2 = \frac{(1.00)(495.6)(66)}{(66+33)} + \frac{(1.00)(125)(345)(43.9)^2}{(4)(66+33)} = 540.6 \text{ kN}$$

$$\phi T_2 = 68.8 + 39.5 = 108.3 \text{ kips/bolt}$$

c. For the limit state of bolt fracture without yielding of the tension flange, the design resistance per tension bolt is calculated as

$$\phi T_3 = \phi_d\, r_{nt} = (1.00)(495.6) = 495.6 \text{ kN/bolt}$$

The bolt fracture limit state governs:

$$\phi R_n = N_{tb}\phi T = n_{tb}\phi T_3 = (4\text{ bolts})(495.6) = 1982\text{ kN} > F_{pr} = 1800\text{ kN, ok}$$

In addition to the checks above for the connection itself, the column must be checked for following limit states: (1) panel zone shear, (2) need for continuity plates, (3) local web yielding, (4) web crippling, (5) compression buckling of the web, and (6) local flange bending.

The cyclic performance of a well-designed T-stub connection is shown in Fig. 4.18. The figure shows excellent energy dissipation and stiffness to a rotation of 0.04 rad, with a decline shortly afterwards due to local buckling of the beam (see Fig. 4.1).

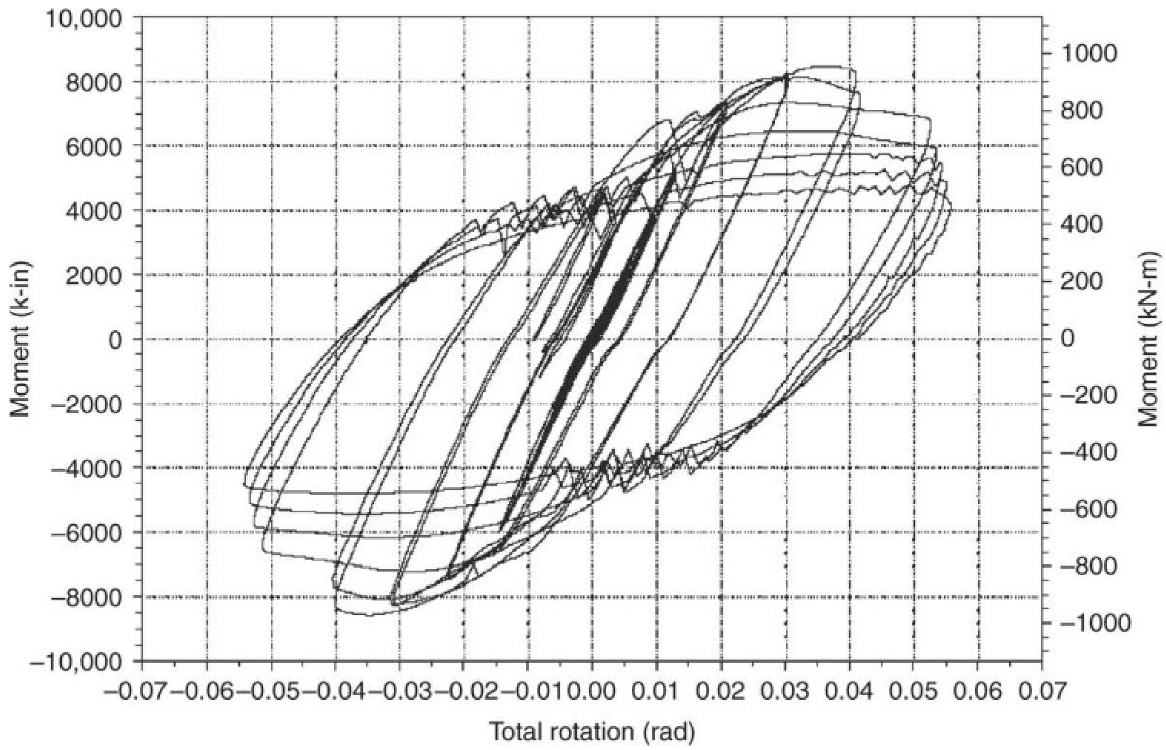

FIGURE 4.18 Cyclic behavior of connection in Fig. 4.1.

4.3.3 End-Plate Connections

End-plate connections are common in some areas of the country and very popular in prefabricated metal buildings. The mechanistic behavior of an end-plate connection is very similar to that of a T stub, with the difference being that the size of the plate is longer than that of the flange of a T stub. If the plate is thin or of moderate thickness compared to the column flange, yield lines will form between the holes in the plate resulting in a plastic mechanism. Because the pattern of yield lines can be complex, the computation of the strength of the plate is not as simple as for a T stub. In the latter case, only two yield lines occur on each half of the stub, one at the bolts and one at the intersection of the flange and web (Fig. 4.15). Two typical yield-line patterns for some common end-plate configurations are shown in Fig. 4.19 (Murray and Meng, 1996; Murray and Watson, 1996). The group patterns can be very complex and not easy to determine for cases with multiple bolts in one row. Yield lines around each individual bolt, in addition to the group patterns shown in Fig. 4.20, are also possible. If the end plate is thick, the behavior will shift to that of a thick T stub. In this case, the failure will be either by tension in the bolts to the column or bolt shear in the connection to the beam. In all cases, care should be exercised not to overstress the column flanges. The strength of the column flange can be checked by a yield-line approach (Nader and Astaneh, 1992), just as for the plate itself (Fig. 4.20). An excellent review of the development of end-plate connections is given by Griffiths (1984), and detailed design guidelines and design aids for their design under monotonic loading are available (Murray, 1990). Recently, Murray and Meng (1996) have suggested a direct formula for calculating the thickness, t_p, of an end plate for a four bolt unstiffened end plate:

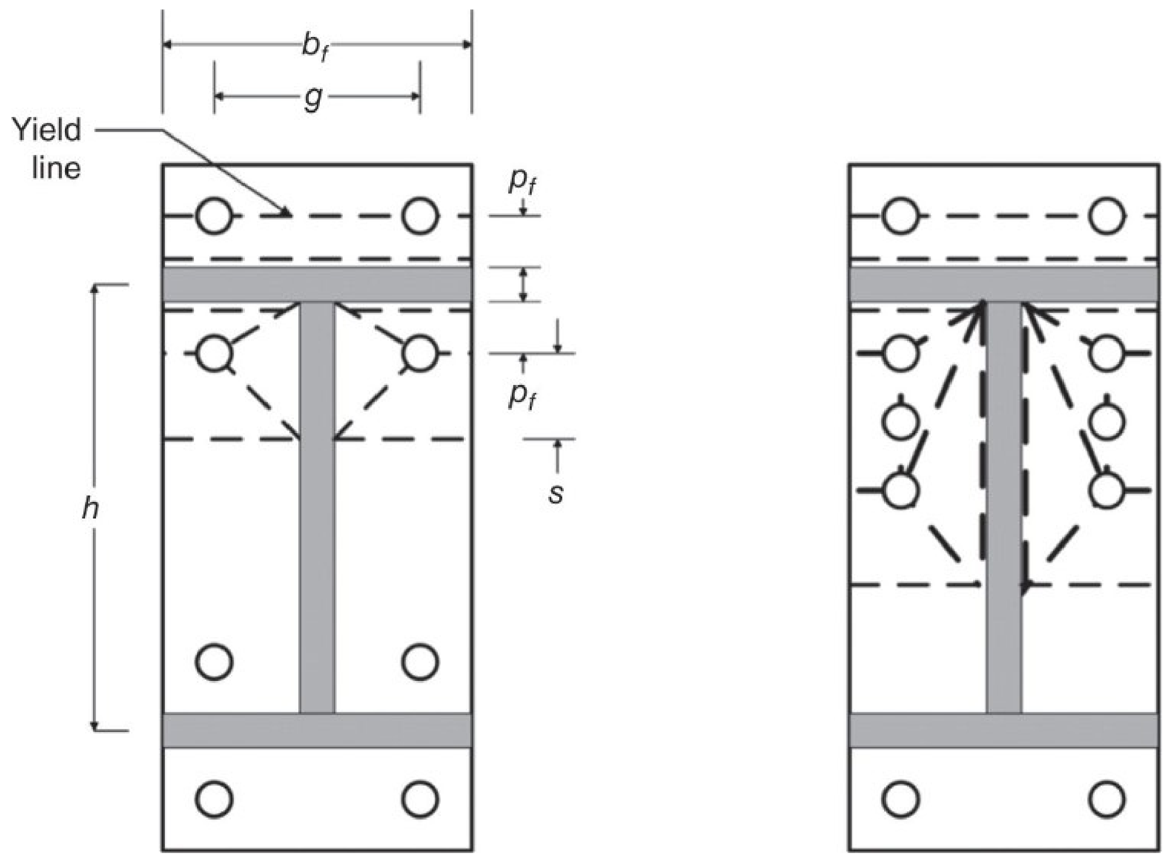

FIGURE 4.19 Typical yield line patterns for end plates.

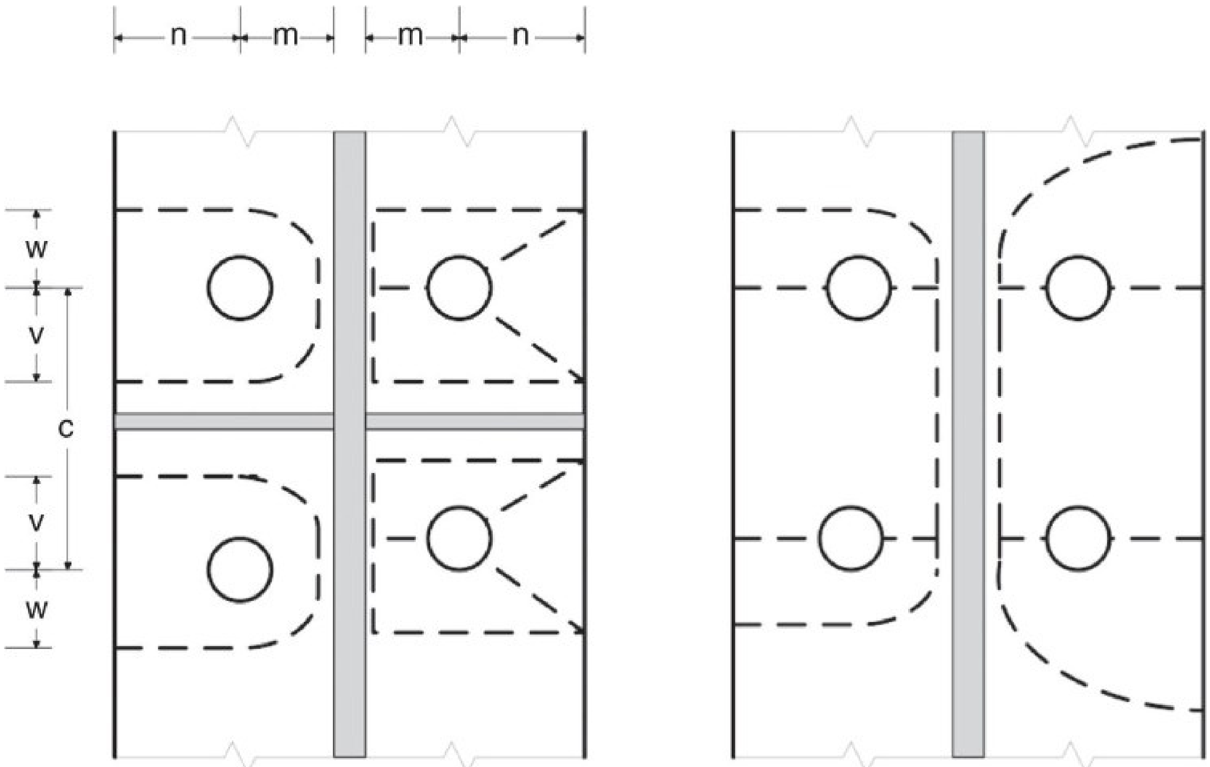

FIGURE 4.20 Typical yield line patterns for column flanges with and without stiffeners.

$$t_p = \sqrt{\frac{\left(M_u / F_{py}\right)}{\left[\left(\frac{b_f}{2}\left(\frac{1}{p_f}+\frac{1}{s}\right)+\left(p_f+S\right)\left(\frac{2}{g}\right)\right)\left(h-t_f-P_f\right)+\frac{b_f}{2}\left(\frac{1}{2}+\frac{h}{p_f}\right)\right]}}$$

$$s = \frac{1}{2}\sqrt{b_t g}$$

where F_{py} is the yield stress of the end-plate material and t_p is the thickness of the plate, b is the width, p_f is the distance from the beam flange to the bolt centerline, S is the distance to the last yield line, and the subscripts b and f refer to the plate and flange, respectively.

Once the plate thickness has been selected, the actual capacity of the connection can then be calculated as

$$M_u = \left[\left(\left(\frac{b_f}{2}\right)\left(\frac{1}{p_f}+\frac{1}{s}\right)+\left(p_f+s\right)\left(\frac{2}{g}\right)\right)\left(h-t_f-p_g\right)+\left(\frac{b_f}{2}\right)\left(\frac{1}{p_f}+\frac{1}{2}\right)\right]F_{yp}t_p^2$$

Once this computation is made, it must be checked against the maximum capacity of the bolts. The latter is governed by prying action and can be computed based on the techniques discussed in Examples 4.3 and 4.4, or by the flowcharts from Murray shown as Figs. 4.21 through 4.23.

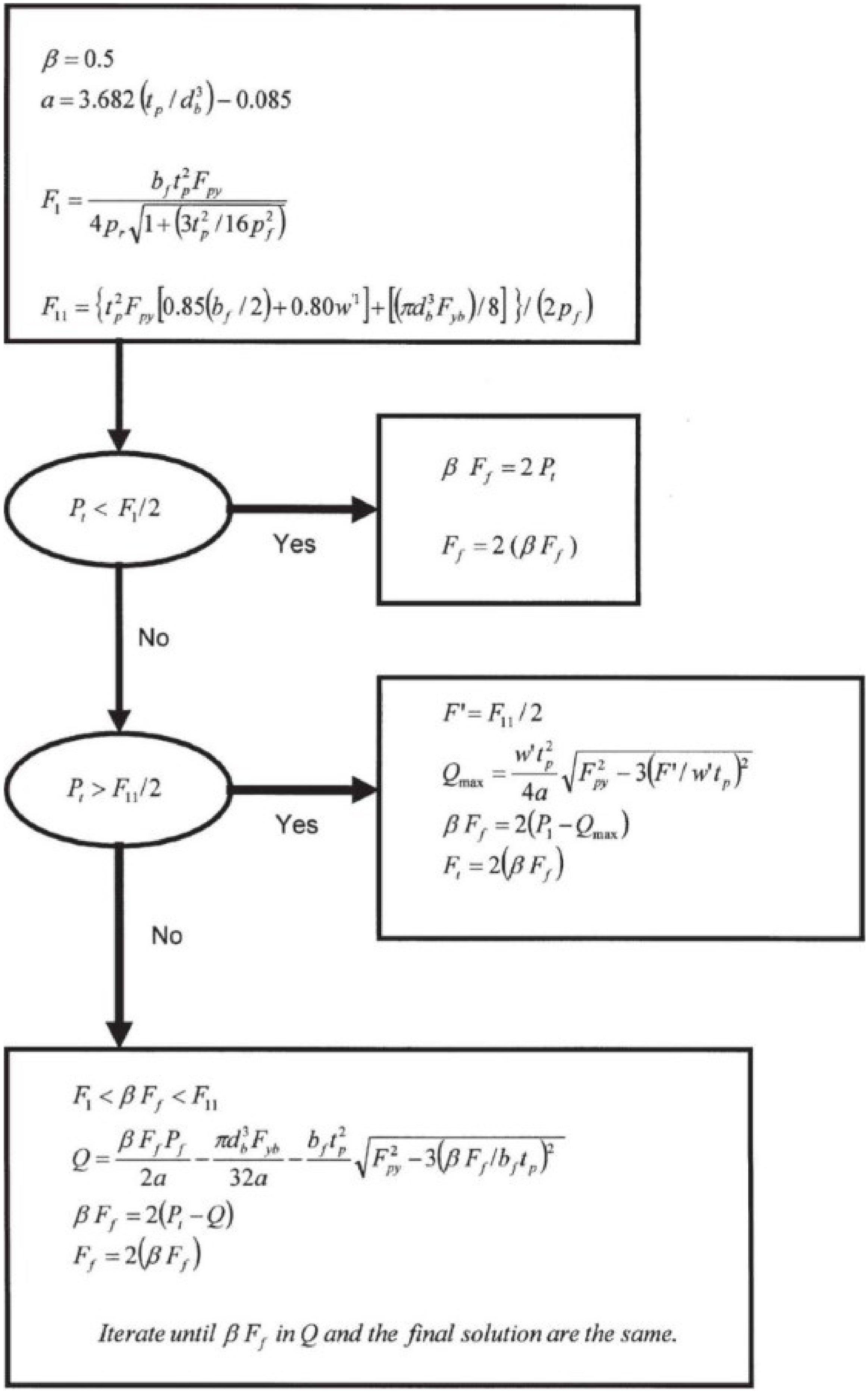

FIGURE 4.21 Flowchart for determining forces in interior bolts.

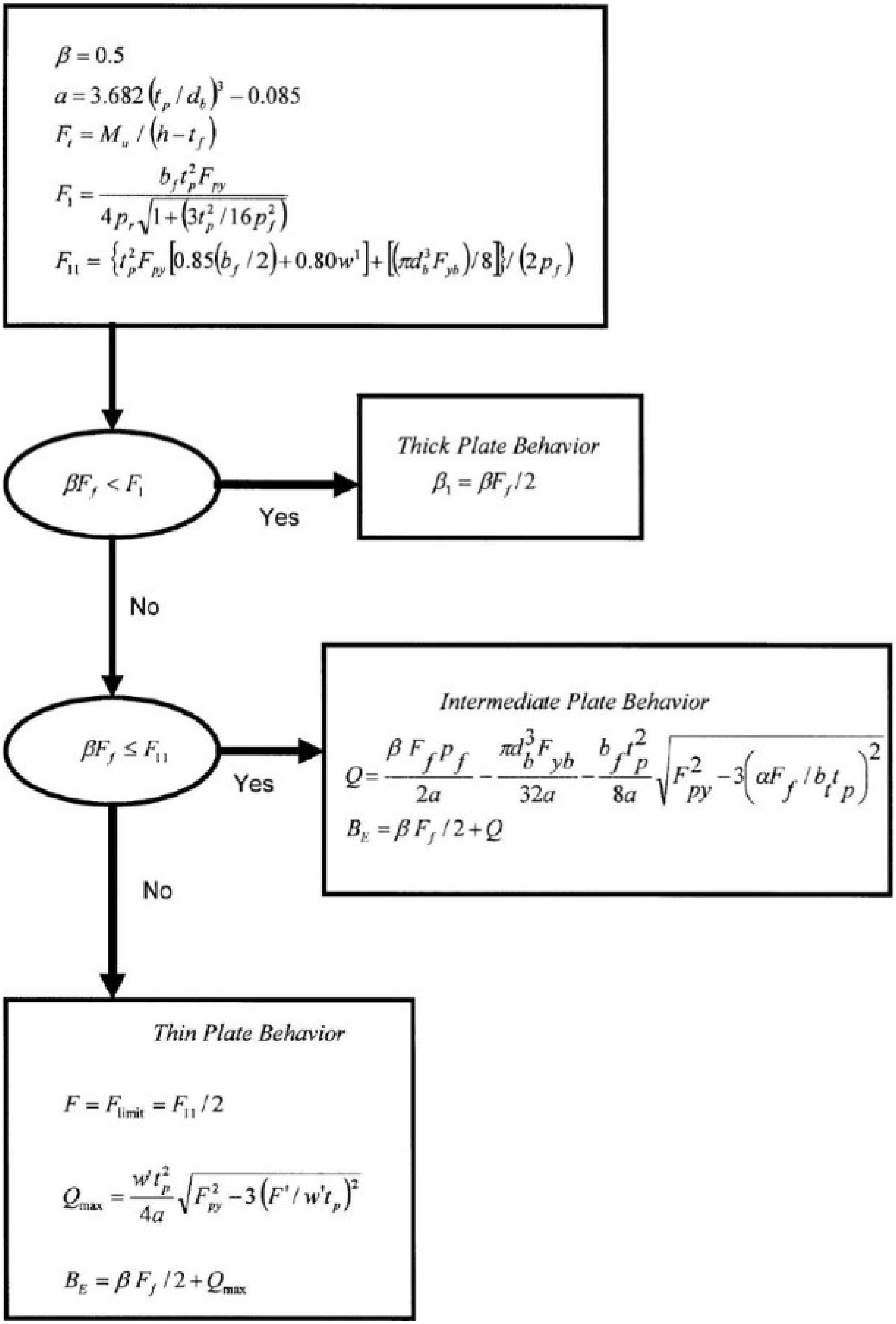

FIGURE 4.22 Flowchart for determining forces in interior bolts.

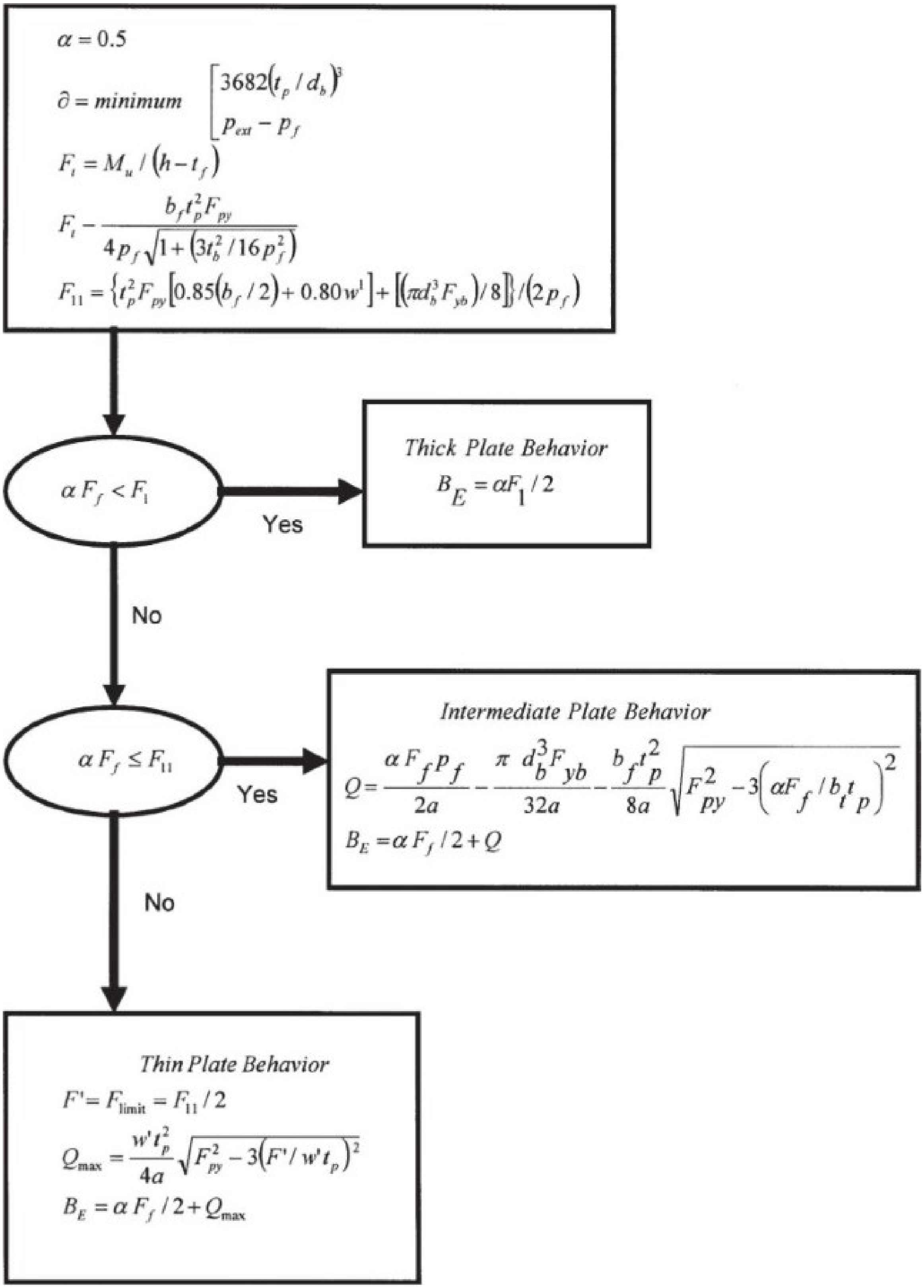

FIGURE 4.23 Flowchart for determining forces in exterior bolts.

Design Example 4.5. Determine the required end-plate thickness and bolt size for a four-bolt extended unstiffened moment end-plate connection. Use A572 Grade 50 for both the beam and end plate, and A325 for the bolts. The factored design moment is 225 kip-ft. See Fig. 4.24 for details of the beam and end-plate sizes.

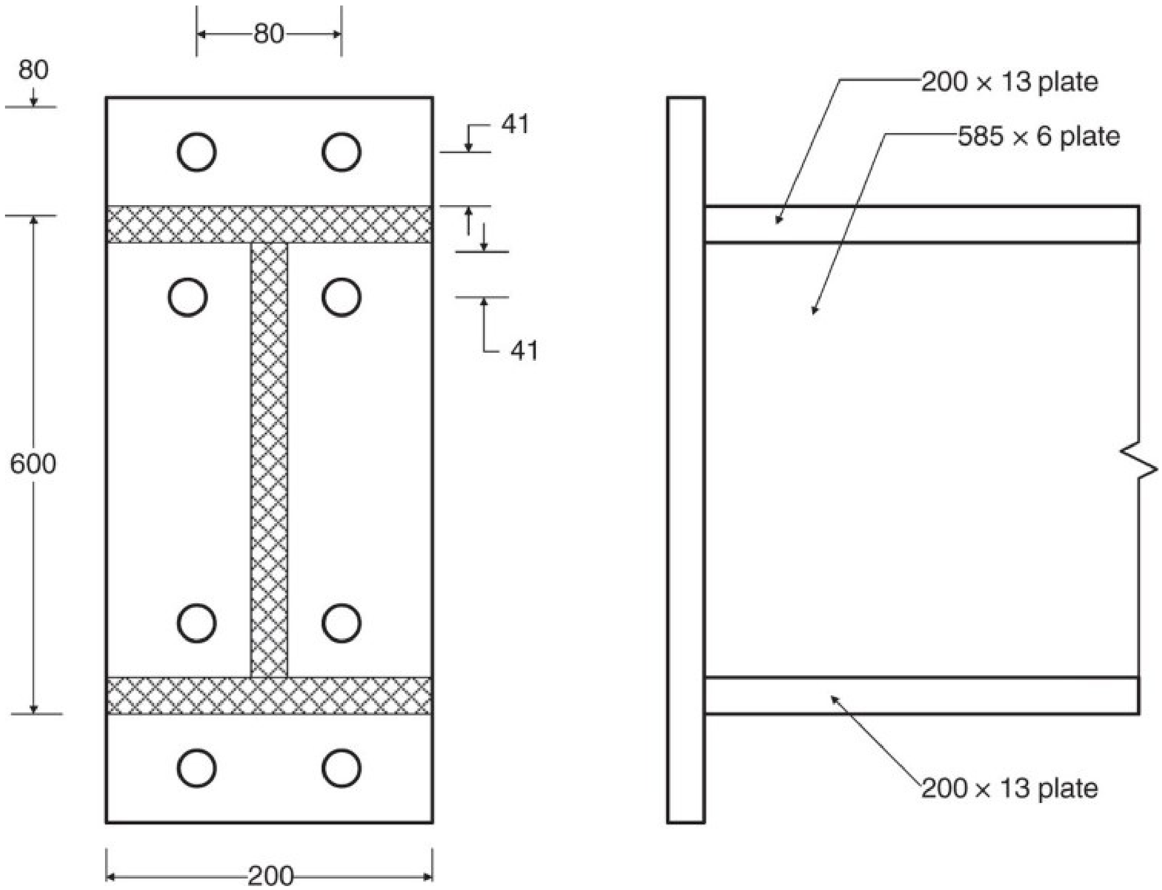

FIGURE 4.24 Details for Example 4.5 (all dimensions in mm).

1. Calculate s and the required end-plate thickness. Using the equations above and the dimensions in Fig. 4.19:

$$s = \frac{1}{2}\sqrt{b_t g} = \left(\tfrac{1}{2}\right)\sqrt{200(85)} = 65.2 \text{ mm}$$

$$t_p = \sqrt{\frac{\left(M_u / F_{py}\right)}{\left[\left(\frac{b_f}{2}\left(\frac{1}{p_f}+\frac{1}{s}\right)+\left(p_f+s\right)\left(\frac{2}{g}\right)\right)\left(h-t_f-p_f\right)+\frac{b_f}{2}\left(\frac{1}{2}+\frac{h}{p_f}\right)\right]}}$$

$$t_p = \sqrt{\frac{\left(300/345\right)(1{,}000{,}000)}{\left[\left(\frac{200}{2}\left(\frac{1}{41}+\frac{1}{65}\right)+(41+655)\left(\frac{2}{65}\right)\right)(600-13-41)+\frac{200}{2}\left(\frac{1}{2}+\frac{600}{41}\right)\right]}}$$

$$t_p = 12.6 \text{ mm} \rightarrow \text{Use } t_p = 14 \text{ mm}$$

2. Determine the critical moment (M_{crit}) as the smallest of the moment capacities of the end plate (M_{plate}) due to the formation of yield lines and failure of the bolts (M_{bolt}) due to prying action. The moment capacity governed by the end plate (M_{plate}) is calculated as

follows:

$$M_u = \left[\left(\left(\frac{b_f}{2}\right)\left(\frac{1}{p_f}+\frac{1}{s}\right)+(p_f+s)\left(\frac{2}{g}\right)\right)(h-t_f-p_g)+\left(\frac{b_f}{2}\right)\left(\frac{1}{p_f}+\frac{1}{2}\right)\right] f_{yp} t_p^2$$

$$M_u = \left[\left(\left(\frac{200}{2}\right)\left(\frac{1}{41}+\frac{1}{65}\right)+(41+65)\left(\frac{2}{85}\right)\right)(600-13-41)+\left(\frac{200}{2}\right)\left(\frac{1}{41}+\frac{1}{2}\right)\right] \frac{(345)(14)^2}{100{,}000}$$

$$M_u = 366.5 \text{ kN-m}$$

To compute the capacity of the connection based on the bolts, a bolt trial size must be chosen. Assume M20 Grade A bolts. The force yield capacity (P_t) of each pair of bolts, based on F_y = 300 MPa for the bolt material, is

$$P_t = 2 \times 620 \text{ MPa} \times 314 \text{ mm}^2 = 389.4 \text{ kN}$$

From the flowchart given in Fig. 4.16, determine the force in the inner bolts:

$$F_1 = \frac{b_f t_p^2 F_{py}}{4 p_f \sqrt{1+\left(3t_p^2/16p_f^2\right)}} =$$

$$F_1 = \frac{(200)(14^2)(345)}{4(41)\sqrt{1+\left(3(14^2)/(16\times 41^2)\right)}} = 81.6 \text{ kN}$$

$$w^1 = \frac{b_f}{2} - (d_b + 2 \text{ mm}) = \frac{200}{2} - (20 + 2 \text{ mm}) = 78 \text{ mm}$$

$$F_{11} = \frac{t_p^2 F_{yp}\left[0.85\left(\frac{b_f}{2}\right)+0.8w^1\right]+\left[\frac{\pi d_b^3 F_t}{8}\right]}{2p_f}$$

$$F_{11} = \frac{(14)^2(345)\left[0.85\left(\frac{200}{2}\right)+0.8(78)\right]+\left[\frac{\pi(20)^3(620)}{8}\right]}{2(41)} = 148.9 \text{ kN}$$

3. Determine the force in the flange (F_f):

Since $P_t > F_1/2$ and $P_t > F_{11}/2$, from the flowchart in Fig. 4.22, and noting that the units in the coefficient a for are in the English system:

$$a = \left[3.682\left(\frac{t_p}{d_b}\right)^3 - 0.085\right](25.4) = 29.9 \text{ mm} \rightarrow 25.4 \text{ mm/in}$$

$$F' = \frac{F_{11}}{2} = \frac{148.9}{2} = 74.5 \text{ kN}$$

$$Q_{\max} = \frac{w^1 t_p^2}{4a}\sqrt{F_{py}^2 - 3\left(\frac{F'}{w_1 t_p}\right)^2} = \frac{(78)(14)^2}{4(29.9)(1000)}\sqrt{(345)^2 - 3\left(\frac{74.5(1000)}{29.9(14)}\right)^2} = 37.8 \text{ kN}$$

$$F_t = \frac{M_u}{d - \frac{t_f}{2}} = \frac{300}{0.587} = 511 \text{ kN}$$

$\beta F_f = 2(P_t - Q_{\max}) \rightarrow 0.5\, F_f = 2(389.4 - 37.8) = 703.2$ kN or $F_f = 1406$ kN

$M_{\text{bolt}} = F_f\,(h - t_f) = 1406(0.587) = 825.5$ kN-M

$M_{\text{bolt}} = 825.5$ kN-m $> M_{\text{plate}} = 366.5$ kN-m, so $M_{\text{crit}} = M_{\text{plate}}$

$\phi M_{\text{crit}} = (0.9)(366.5$ kN-m$) = 329.8$ kN-m $> M_u = 300$ kN-m, ok

4. Determine the inner end-plate behavior. From Fig. 4.17 and the values from above for a, F_f, F_1, and F_{11}, $\beta F_t = 0.5(511) = 255.5$ kN $> F_{11} = 148.9$ kN, inner end-plate behavior is thin plate behavior. From the flowchart in Fig. 4.17, the inner bolt force is

$$B_E = (\beta\, F_t/2) + Q_{\max} = (0.5(511)/2) + 37.8 = 293.3 \text{ kN}$$

5. Check the outer bolt force. From Fig. 4.18, the outer bolt force also exhibits thin plate behavior and $B_E = 293.3$ kN.
6. Checking the bolt diameter:

$$d_b = \sqrt{\frac{2B_{\max}}{\pi F_1}} = \sqrt{\frac{2(293.3)}{\pi(620)}} = 0.72 \text{ in} \rightarrow \text{Use } 3/4 \text{ in diameter bolt as assumed}$$

While many models of end-plate behavior exist, there was little work on the design of end plates for cyclic loads until the mid 1990s. (Whittaker and Walpole, 1982; Tsai and Popov, 1988, 1990; Astaneh-Asl and Nader, 1992; Ghobarah et al., 1992). Astaneh and Nader (1992) reviewed the available data and proposed design provisions. They listed plastic yield line formation in the end plate and column flange bending as the most desirable failure modes: After the 1994 Northridge earthquake, there has been a substantial amount of work on behavior and design of end plates for seismic loads (FEMA, 2000).

For developing design provisions the end plate can be separated into two T stubs (Packer and Morris, 1977) and thus results in a very similar approach to design to that developed in the previous section. The design forces can be calculated from free-body diagrams such as those shown in Fig. 4.15. Replacing “a” with “n” and “b” with “v” in Fig. 4.15 to follow the nomenclature in (Astaneh-Asl, 1995) and using appropriate resistance (ϕ_b) and material overstrength factors (α) to satisfy capacity design criteria, equilibrium of forces between the force in the plate (F_{ep}) and the force in the beam flange (F_{fb}) gives

$$\phi_b F_{ep} = \alpha \phi_b F_{fb}$$

$$\phi_b \frac{(2)\left(M_v + M_{v'}\right)}{v} = \alpha \phi_b \frac{ML_{pb}}{\left(d_b - t_{fb}\right)}$$

$$\phi_b \frac{(2)\left(M_v + M_{v'}\right)}{v} = \frac{M_{vc}}{\left(d_b - t_{fb}\right)}$$

From Fig. 4.20:

$$F_{f1} = t_{fc}^2 f_y \left\{\frac{n}{v} + \frac{n}{m} + (n - 0.5D')\left(\frac{1}{v} + \frac{1}{m}\right) + \pi + \pi \sec^2\left(\tan^{-1}\left(\frac{2}{\pi}\ln\left(\frac{v}{m}\right)\right)\right)\right\}$$

$$F_{f2} = t_{fc}^2 f_y \left\{\left[\frac{1}{v} + \frac{1}{w}\right](2m + 2n - D') + \frac{(2v + 2w - D')}{m}\right\}$$

$$w = \sqrt{\left(m(m + n - 0.5D')\right)}$$

To ensure that no out-of-plane bending occurs (Astaneh-Asl and Nader, 1992):

$$\phi_b F_{fb} = \phi_b \frac{M_{pb}}{(d - t_{fb})} \le \begin{cases} \phi_b F_{f1} \\ \phi_b F_{f2} \end{cases}$$

$$\begin{cases} \text{if } \dfrac{b_p}{b_{fc}} = 1.0 \quad \text{then } t_{fc} \ge t_p \\[2ex] \text{if } \dfrac{b_p}{b_{fc}} = 0.5 \quad \text{then } t_{fc} \ge 0.65\, t_p \end{cases}$$

where M_{pb} is the plastic capacity of the beam, t_{fb} is the thickness of the beam flange, d is the distance between flange centerlines, and b_{fc} is the width of the column flange. Interpolation is permitted between b_p/b_{fc} values of 1.0 and 0.

4.4 FLEXIBLE PR CONNECTIONS

The connections described in the previous sections, all fall in the category of full-strength, partially restrained connections. With respect to stiffness, these connections have high initial stiffness and can probably be analyzed as rigid connections for service loads. There are a number of other common steel connections, primarily the top-and-seat angle with and without stiffeners, that offer partial-strength, partial restraint behavior. Design examples for this type of connection are available in the literature [see pp. 9-253 to 9-261 of the Manual of Steel Construction LRFD (AISC, 1993), for example] and will not be covered here. In most cases these connections cannot provide sufficient lateral stiffness to resist large wind or earthquake loads unless all the connections in the structure are of this type and the effect of the slab is taken into account (Fig. 4.25) or the angles are stiffened (Fig. 4.26). For the design of this type of PR composite connections, shown in Fig. 4.25, the reader is referred to Leon et al. (1996).

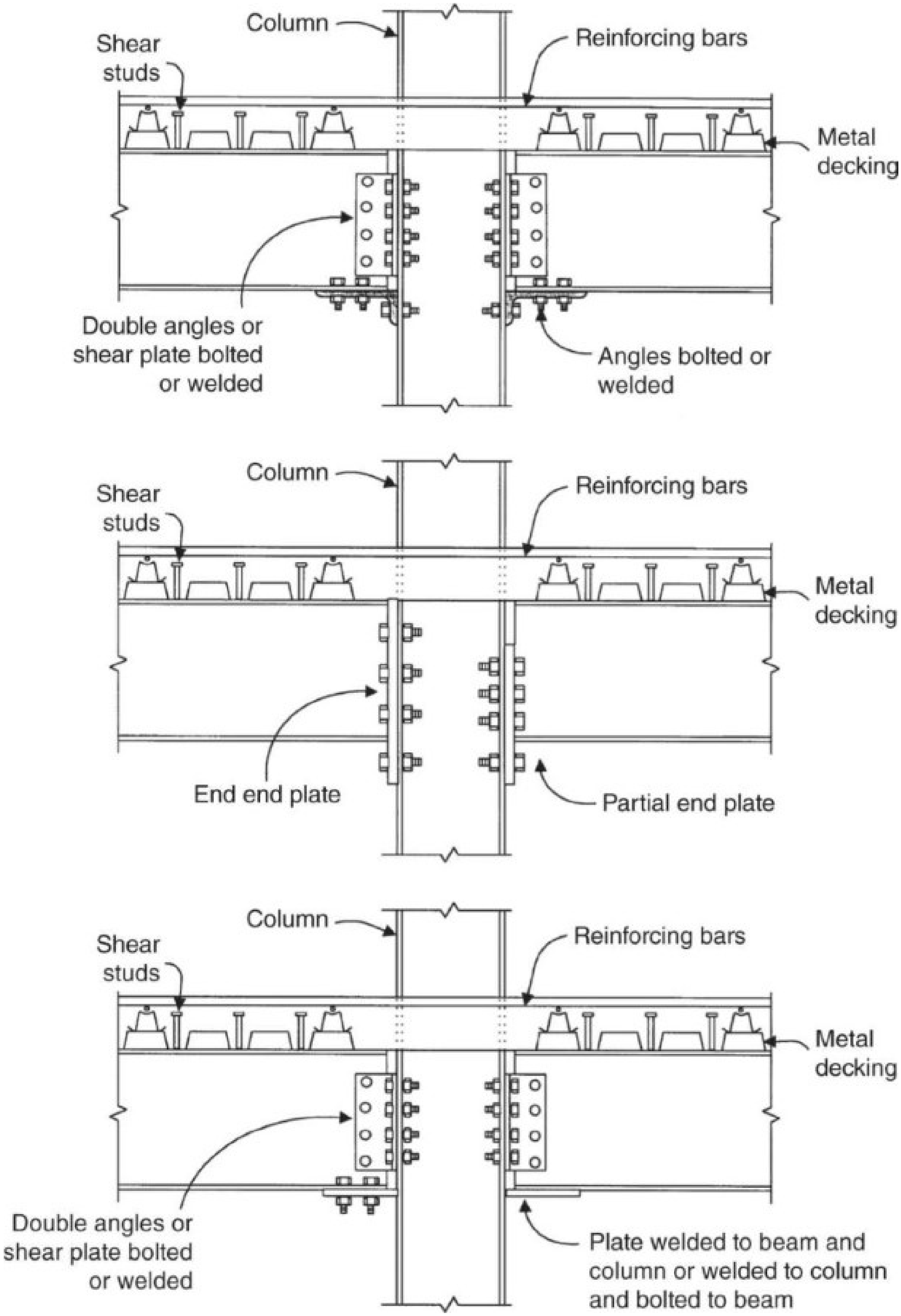

FIGURE 4.25 Flexible composite PR connections.

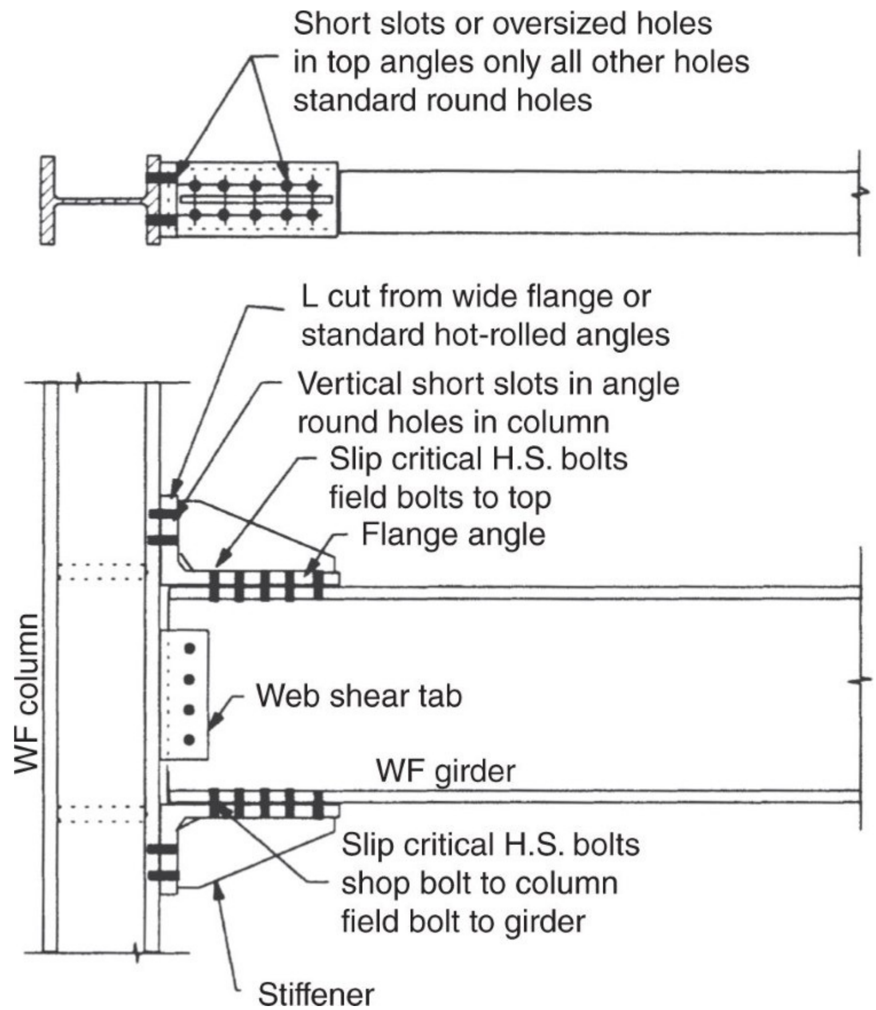

FIGURE 4.26 Stiffened seat connection (Astaneh-Asl, 1995).

4.5 CONSIDERATIONS FOR ANALYSIS OF PR FRAMES[1]

The common practice for analysis of multistory frames assumes that joints are rigid and beam and columns intersect at their centerline. Using this method there is no allowance for connection and panel zone flexibility, the spans of the beam and columns are overestimated, and the joints have no physical size and are reduced to a point. Since the PR behavior of most connections was recognized early, several modifications have been proposed to classical linear analysis techniques to account for connection flexibility. The first attempts involved modifying the slope-deflection method by adding the effect of linear rotational springs at beam ends (Batho and Rowan, 1934; Rathbun, 1936). Johnston and Mount (1941) gave a complete listing of coefficients to be used in the slope-deflection method including both the flexibility of the connections and the finite widths of the members. These methods were for hand calculations, and thus were limited to the analysis of relatively small structures. In an exception to this, Sourochnikoff (1950) used the beam line method along with experimental results obtained by Rathbun (1936) to compute the nonlinear cyclic response of a one-story one bay partially restrained frame.

Monforton and Wu (1963) incorporated linear connection flexibility into the computerized direct stiffness method. This development permitted the analysis of large structures, but was still limited to linear analysis where the connections have constant stiffness. Lionberger and

Weaver (1969) published the results from a program that performed fully dynamic lateral load analysis on plane frames. The connections in their program were modelled by a nondegrading bilinear model, which included the sizes of the rigid panel zones. Moncarz and Gerstle (1981) used a nondegrading trilinear model to analyze steel partially restrained frames subjected to lateral load reversals. When the first databases for connections were developed (Frye and Morris, 1975; Ang and Morris, 1984; Nethercot, 1985; Kishi and Chen, 1986), nonlinear expressions for moment-rotation curves became widely available. This led to the development of numerous computer programs that modelled the nonlinear behavior of PR connections. Shin (1992) and Shin and Leon (1997) devised hysteresis rules for nonsymmetrical composite connections with degradation of the unloading stiffness based on the maximum attained rotation, and implemented them in a dynamic nonlinear plane frame analysis program.

The dynamic performance of frames incorporating partially restrained composite connections has been studied by Nader and Astaneh-Asl (Astaneh-Asl et al., 1991; Nader and Asataneh, 1992) and Leon and Shin (1995). The numerical results obtained by Leon and Shin (1995) using a modified tri-linear degrading model showed excellent agreement with test results from a two-story, two-bay, half-scale frame. The analytical studies for PR frames showed good seismic performance for ground motions expected in zones of low to moderate seismicity. In particular they showed less problems with local buckling of members and equal or better energy dissipation capacity than rigid frames. In addition these studies showed that the lateral drifts of PR frames were within ±20% of those of companion rigid frames when four-, six-, and eight-story frames were subjected to the El Centro, Parkfiled, and Pacoima ground motions. The results of these studies confirmed those of Astaneh-Asl and Nader (1992), and verified their shake table results. Further verification of the good performance can be found in the work of Osman et al. (1993) who presented the analysis results for eight-story frames with end-plate connections and flexible panel zones of various thickness.

This review consciously limits itself to some of the more important original contributions to the analysis of PR frames. In the last two decades there has been a tremendous amount of work done on analytical details that are beyond the design scope for regular PR frames, which is the subject of this chapter. As noted in the introduction to this chapter, there are now robust analytical tools that designers can use to efficiently analyze PR frames including the effects of nonlinear connection performance.

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CHAPTER 5
SEISMIC DESIGN OF CONNECTIONS

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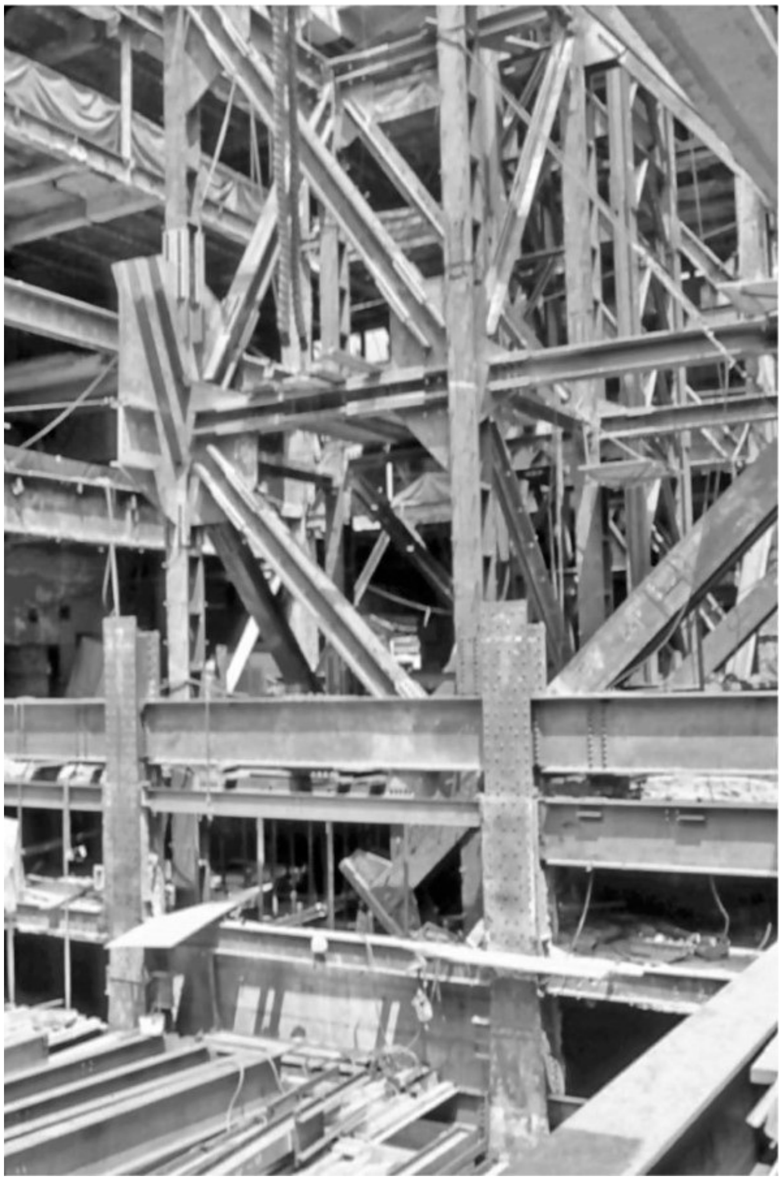

(*Courtesy of The Steel Institute of New York.*)

5.1 SPECIAL DESIGN ISSUES FOR SEISMIC DESIGN

The structural design philosophy for most loading conditions, such as gravity loads due to everyday dead and live loads or expected wind loadings, is that the structural system, including the connections, resist the loads essentially elastically, with a safety factor to account for unexpected overloading within a certain range. The parallel philosophy for resisting earthquake-induced ground motions is in striking contrast to that for gravity or wind loading. This philosophy has evolved over the years since the inception of earthquake-resistant structural design early in the twentieth century, and continues to develop as engineers learn more about the performance of structures subjected to strong earthquakes. The present general philosophy for seismic design has been most succinctly stated in the *Bluebook* of the Structural Engineers of California (SEAOC, 1999) for a number of years. The document states this approach as the following:

> Structures designed in conformance with these Requirements should, in general, be able to:
>
> Resist a minor level of earthquake ground motion without damage.
>
> Resist a moderate level of earthquake ground motion without structural damage, but possibly experience some nonstructural damage.
>
> Resist a major level of earthquake ground motion—of an intensity equal to the strongest earthquake, either experienced or forecast, for the building site—without collapse, but possibly with some structural as well as nonstructural damage.
>
> It is expected that structural damage, even in a major design level earthquake, will be limited to a repairable level for most structures that meet these Requirements. In some instances, damage may not be economically repairable. The level of damage depends upon a number of factors, including the intensity and duration of ground shaking, structure configuration type of lateral force resisting system, materials used in the construction and construction workmanship.

It is clear, then, that when subjected to a major earthquake, buildings designed to meet the design requirements of typical building codes, such as the International Building Code (IBC, 2012), are expected to damage both structural and nonstructural elements. The intent of the building code under this scenario is to avoid collapse and loss of life. Because of the economic impact, structural design to resist major earthquake ground motions with little or no damage has been limited to special buildings, such as postdisaster critical structures (e.g., hospitals, police, and fire stations) or structures that house potentially hazardous materials (e.g., nuclear power plants).

Structural design for large seismic events must therefore explicitly consider the effects of response beyond the elastic range. A mechanism must be supplied within some elements of the structural system to accommodate the large displacement demand imposed by the earthquake ground motions. In typical applications, structural elements, such as walls, beams, braces, and to a lesser extent columns and connections, are designed to undergo local deformations well beyond the elastic limit of the material without significant loss of capacity. Provision of such large deformation capacity, known as *ductility,* is a fundamental tenet of seismic design. Note that new technologies (e.g., base isolation and passive energy dissipation) have been developed to absorb the majority of the deformations and, therefore, protect the “main” structural elements from damage in a major earthquake. Such applications are gaining increasing application in areas of high seismicity. Addressing such systems is beyond the scope of this text, which will focus on the seismic design of steel connections in typical applications.

In most cases, good seismic design practice has incorporated an approach that would provide for the ductility to occur in the members rather than the connections. This is especially the case for steel frame structures, where the basic material has long been considered the most ductile of all materials used for building construction. The reasons for this approach include the following:

- The failure of a connection between two members could lead to separation of the two elements and precipitate a local collapse.
- The inelastic response of members is more easily defined and more reliably predicted.
- The inelastic action of steel members generally occurs at locations where the distribution of strain and stress does not induce constraint that could lead to a state of triaxial tension. Under certain circumstances, while connections can induce significant constraint that inhibits material yielding.
- Local distributions of strain and stress in connections can become quite complicated, and be very different from simple models typically used in design.
- Connection failures in frame structures could jeopardize the stability of the system by reducing the buckling restraint provided to the building columns.
- The repair of connection damage may be more difficult and costly than replacing a yielded or buckled member.

Building codes have incorporated this philosophy into their seismic design requirements for a number of years. The most common method employed to incorporate this approach has been to require that the connections be designed to resist the expected member strength of the connecting elements, or the maximum load that can be delivered to the connection by the system. This implies that a conscious effort has been made by the designer to preclude the connections from undergoing severe inelastic demands. As such, a strength-based design approach as employed by the latest codes [e.g., 2012 IBC and 2009 National Earthquake Hazards Reduction Program (NEHRP) provisions] is a much more direct and fundamental procedure than allowable stress methods that were previously followed. Seismic design of steel structures using LRFD is clearly a more rational, consistent, and transparent approach. As such, the 2010 AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2010) is based primarily on an LRFD approach. Connection design procedures in this document are based on Chapter J, “AISC Specification for Structural Steel Buildings” (AISC, 2010).

The AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2010) include a number of requirements that are intended to ensure that this philosophy can be realized in the actual seismic performance. For example, the provisions require that the expected (rather than the nominal) yield strength of the materials be considered in comparisons of relative strengths between various members and/or connections. This term, R_y, ranges from 1.1 to 1.6 depending on the material specification chosen.

Other design approaches intend for the connections themselves to absorb substantial energy and provide major contributions to the displacement ductility demand. Examples of such a system would include both fully restrained (FR) and partially restrained (PR) connections in moment-resisting frames. To properly incorporate these elements in seismic design requires a much greater level of attention than for standard connection design or for moment connections subjected only to typical static loads. In addition to typical strength design requirements, such connections should take factors such as the following into account:

- Toughness of joining elements in the connections, including any weldments.
- High level of understanding of the distribution of stress and strain throughout the connection.
- Elimination (or at least control) of stress concentrations.
- Detailed consideration of the flow of forces and the expected path of yielding in the connection.
- Good understanding of the properties of the materials being joined at the connection (e.g., through-thickness, yield-to-tensile ratio).
- The nature of the connection demands being high-strain, low-cycle fatigue versus low-strain high-cycle fatigue typical of other structural applications such as bridges.
- The dynamic nature of the response which induces strain rates well below impact levels.
- The need for heightened quality control in the fabrication, erection, and inspection of the connection.

While these types of considerations are particularly critical for connections where inelastic response is anticipated, it also behooves the designer to take factors such as these into account for all connections of the seismic force resisting system.

In the AISC *Seismic Provisions*, all connections in the lateral force-resisting system are required to meet a number of basic design requirements, which go beyond those required of joints in typical steel connections. For bolted connections, the design of bolted joints require the following:

- All joints must use fully tensioned, high-strength bolts.
- Bearing design values are allowed, within the limits of the lower nominal bearing strength, $2.4dtF_u$ unless required strength of the connection is based on the expected strength of the member.
- Bolted joints are not to be used in combination with welds in the same force component of the connection.

For welded joints, the requirements include:

- Provision of approved welding procedure specifications that meet American Welding Society (AWS) D1.1 and are within the parameters established by the filler-metal manufacturer.
- All welds must have a Charpy V-notch (CVN) toughness AWS classification or manufacturers certification of 20 ft-lb at 0°F (27J at –18 C). For welds noted to be demand critical, an additional toughness of 40 ft-lb at 70°F (54J at 21 C) must be demonstrated.
- In areas of large expected strain referred to as *protected zones*, discontinuities created by fabrication or other erecting operations are not permitted, in an effort to avoid premature

fracture.

5.2 CONNECTION DESIGN REQUIREMENTS FOR VARIOUS STRUCTURAL SYSTEMS

Proper system selection is a critical element in successful seismic design. Various systems, such as fully and partially restrained moment-resisting frames, concentrically braced frames, and eccentrically braced frames, are addressed in the AISC *Seismic Provisions*. These provisions have specific requirements for the different structural systems that address connection design.

For moment-frame systems, special moment frame (SMF) and intermediate moment frame (IMF) connections have specified values for both inelastic deformation and strength capacities, since it is expected that these connections will absorb substantial energy during the design earthquake. Deformation capacities are to be demonstrated by qualified cyclic testing of the selected connection type. At the minimum acceptable drift deformation angle (0.04 rad for SMF, 0.02 rad for IMF), the provisions require that the nominal beam plastic moment, M_p, be reached unless local buckling or a reduced beam approach is followed, in which case the value is reduced to 0.8 M_p. The minimum beam shear connection capacity is defined as resisting a combination of full-factored dead load, a portion of the live and snow load (if any), and the shear that would be generated by the expected moment capacity (including R_y) of the beam due to seismic actions. Finally, for SMF, the joint panel zone shear is required to have a capacity able to resist the actions generated by the hinging of the beams framing into the connection. For ordinary moment frames (OMF), the strength requirement is similar, but there is no required rotation deformation limit. No specific joint panel zone requirements are defined for OMF systems.

The design requirements for PR connections in SMF and IMF are similar to those required for FR connections as described previously. For OMF structures, a set of requirements are provided to ensure a minimum capacity level of 50% of that of the weaker connected member, and that connection flexibility is considered in the determination of the overall frame lateral drifts.

Another moment frame system in the AISC *Seismic Provisions* is the special truss moment frame (STMF). The system was developed by Professor Subhash Goel and his students at the University of Michigan (Itani and Goel, 1991; Goel and Itani, 1994; Basha and Goel, 1994). As with other steel systems, the concept of the STMF is to focus the inelastic behavior in specific elements of the truss, known as the *special segment*. The connections between the various elements of the truss and between the truss and the frame columns are designed to have a strength sufficient to develop the expected yield force and required deformation level of the special segments.

The connection design requirements of AISC *Seismic Provisions* are similar for both special concentrically braced frames (SCBF) and ordinary concentrically braced frames (OCBF). For OCBF, the connections that are part of the bracing system must meet the lesser of the following:

- The nominal axial tensile strength of the bracing member, including R_y.
- The maximum force that can be transferred to the brace by the remainder of the structural system. An example of how this provision could be invoked would be the uplift capacity of a system with spread footing foundations.
- The amplified force demands, as defined by the system overstrength factor, Ω_0, as defined in ASCE 7 (ASCE, 2010).

For SCBF, the connection strength must exceed the lesser of the first two elements in this list, fully ensuring that the connections are not the weak elements in the system.

For SCBF, both the tensile and flexural strength must be considered in the design of the connections. The flexural strength of the connections in the direction of brace buckling is required to be greater than the nominal moment capacity of the brace, unless they are specifically designed to provide the expected inelastic rotations that can be generated in the postbuckling state. This type of detail typically includes a single gusset plate where there is adequate separation between the end of the brace and the connecting element so that the gusset plate can bend unrestrained, as developed from research at the University of Michigan (Astaneh, 1989). In addition, the potential for buckling of gusset plates that may be used in bracing connections must be addressed. Beam-to-column connections in SCBF frames must also be able to demonstrate a required rotation capacity of 0.025 rad or be designed to have a moment capacity equal to develop the expected capacity of the beam element.

The eccentrically braced frame (EBF) was developed through years of research at the University of California by Egor Popov and his students, was the first to explicitly require that elements and connections within the system be designed to limit the inelastic response to special members known as “links.” For example, in the 2010 AISC *Seismic Provisions*, the design of connections between links and brace elements must consider both the expected overstrength of the material and the strain hardening that is expected to occur in properly detailed link elements. The design of such connections must also be detailed such that the expected response of the link elements is not altered.

In a number of EBF configurations, the link beams are located at the end of a bay, adjacent to a supporting column. Since severe inelastic rotation demands are expected in link beams during major seismic events, there was concern that without special precautions, link-to-column connections in these EBF configurations may be subject to the same type of connection fractures that numerous moment connections suffered in the Northridge earthquake. As a result, the provisions require that these connections be tested to demonstrate that they have adequate rotation capacity. Without testing to qualify the connection detail, the links are conditions that are required to be proportioned to yield in shear and the connections must be reinforced to preclude inelastic behavior at the face of the column.

Two new systems were introduced in the 2005 AISC *Seismic Provisions*. The first of these is the buckling restrained braced frame (BRBF). This special class of concentrically braced frame relies on brace elements that are specially designed to preclude compression buckling over the length of the member. As a result, the energy dissipation and ductility of these braces is significantly improved over that of conventional braced frames. In BRBFs, the tension and compression capacity of the braces are approximately equal, with the compression capacity being approximately 10% greater in most cases. As with the other systems, the connections between the braces to the other members of the frame are designed for the expected capacity of the braces, increased by 1.1 to account for potential strain hardening. Beam-to-column connections in BRBFs need to be designed for the same requirements as SCBFs as noted above.

The other system introduced in the 2005 AISC *Seismic Provisions* is the special plate shear wall (SPSW). In this system, thin steel plates are connected to horizontal and vertical boundary elements. (HBE and VBE). The plate elements are designed to yield and behave in a ductile manner. The connections between the plates and the boundary elements are designed to develop the expected tensile capacity of the plate. In addition, the connections between the HBE and VBE are required to be fully restrained moment resisting connections designed to meet the requirements for OMFs. In addition the shear capacity of this connection must be able to transfer the vertical shear induced by the yielded wall plates.

5.3 DESIGN OF SPECIAL MOMENT-FRAME CONNECTIONS

5.3.1 Introduction

This section provides an overview of the requirements and concepts for the design of special moment frame (SMF) connections. The design basis presented is established based on as well as requirements given in the AISC 341 *Seismic Provisions* (2010), and AISC 358 *Prequalified Connections for Special and Intermediate Steel Moment frames for Seismic Applications* (AISC, 2010). First, general concepts and objectives for design will be outlined, followed by specific connection types and design examples. Another excellent reference for Steel Moment Frame design is NIST Technical Brief Number 2, “Seismic Design of Steel Moment Frames: A Guide for Practicing Engineers (NIST, 2009).”

Figure 5.1 shows a typical unreinforced detail for a beam-to-column connection. The beam-to-column connection must be capable of transferring both the beam shear and moment to the column. Prior to the 1994 Northridge earthquake, the assumption for design was that the beam shear is transferred to the column by the beam web connection and the moment is transferred through the beam flanges. Numerous studies after this earthquake by FEMA and others demonstrated that the actual behavior is very different from this assumption. Common practice prior to the Northridge earthquake was to either bolt or weld the web to the column shear plate and to weld the beam flanges to the column flange using a full-penetration groove weld. The panel zone (the column web at the beam intersection) is subjected to a shear force due to these moments applied by the beam.

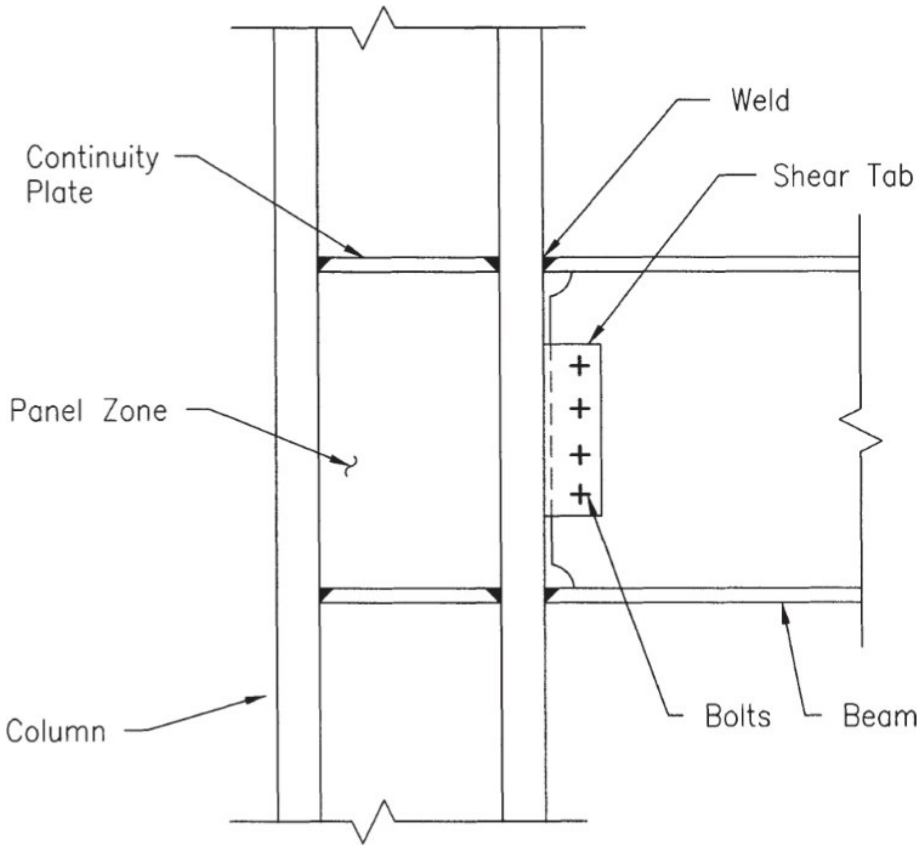

FIGURE 5.1 One-sided moment frame connection.

In the design of SMF connections the engineer must set objectives for both load and deformation capacities. Specifically, the load capacity requirement is based on the maximum attainable moment in the beam. The connection to the column must be sufficiently strong to develop the strength of the beam, thus reducing the risk of brittle failure in the connection. Inelastic deformation capacity is required to ensure ductility in predetermined locations under large deformation demands.

Load Capacities. A common philosophy adopted since the Northridge earthquake has been to design the connection at the column face to remain nominally elastic, and force the inelastic deformation to occur in the beam itself. The design strength of the connection between beam and column is determined by using a "capacity design" approach. The maximum probable moment and shear that the beam is capable of achieving are determined based on the probable strength of the beam. These maximums then become the design loads for the connection. The connection to the column is then designed based on nominal material properties.

The ability to estimate the maximum moment developed in the beam becomes quite important given the uncertainties regarding actual material behavior. The connection should be designed with the expectation of both beam overstrength and strain hardening in the plastic hinge region. A methodology for estimating the probable moment in the plastic hinge was presented in FEMA 350 (FEMA, 2000a). The approach taken here is based a similar approach presented in the AISC 341 and 358 (2010).

Beam overstrength should be accounted for by using the expected yield strength of the beam material. For example, the expected strength of A992, grade 50 steel is approximately

55 ksi (380 MPa), based on mill certificate test values. So for A992, grade 50 steel, the expected yield stress increase from the nominal is (55/50) = 1.1. (*Note:* For ASTM A36 steel this value is 1.5.) This factor is known as R_y in the AISC Seismic Provisions.

The strain-hardening effect in the beam can be quantified by applying a factor of 1.1 to the expected flange yield stress. Recent connection testing (Yu et al., 1997) has shown that an increase by a factor of 1.1 or higher is reasonable to account for strain hardening of the beam in the plastic hinge region. The resulting increase, is known as C_{pr} in AISC 358.

The location of the plastic hinge also must be accounted for. If the plastic hinge occurs at the face of the column ($x = 0$ in), the moment at the column face, M_f, will equal M_{pr}. However, it has been shown by numerous tests that the plastic hinge in a conventional SMF connection typically occurs away from the column face (or end of strengthened beam section), at a distance of approximately $x = d/3$ to $d/4$. Extrapolation over this distance to the column face results in an increased moment demand at the face of the column.

The moment demand at the column face is determined as follows (see Fig. 5.2):

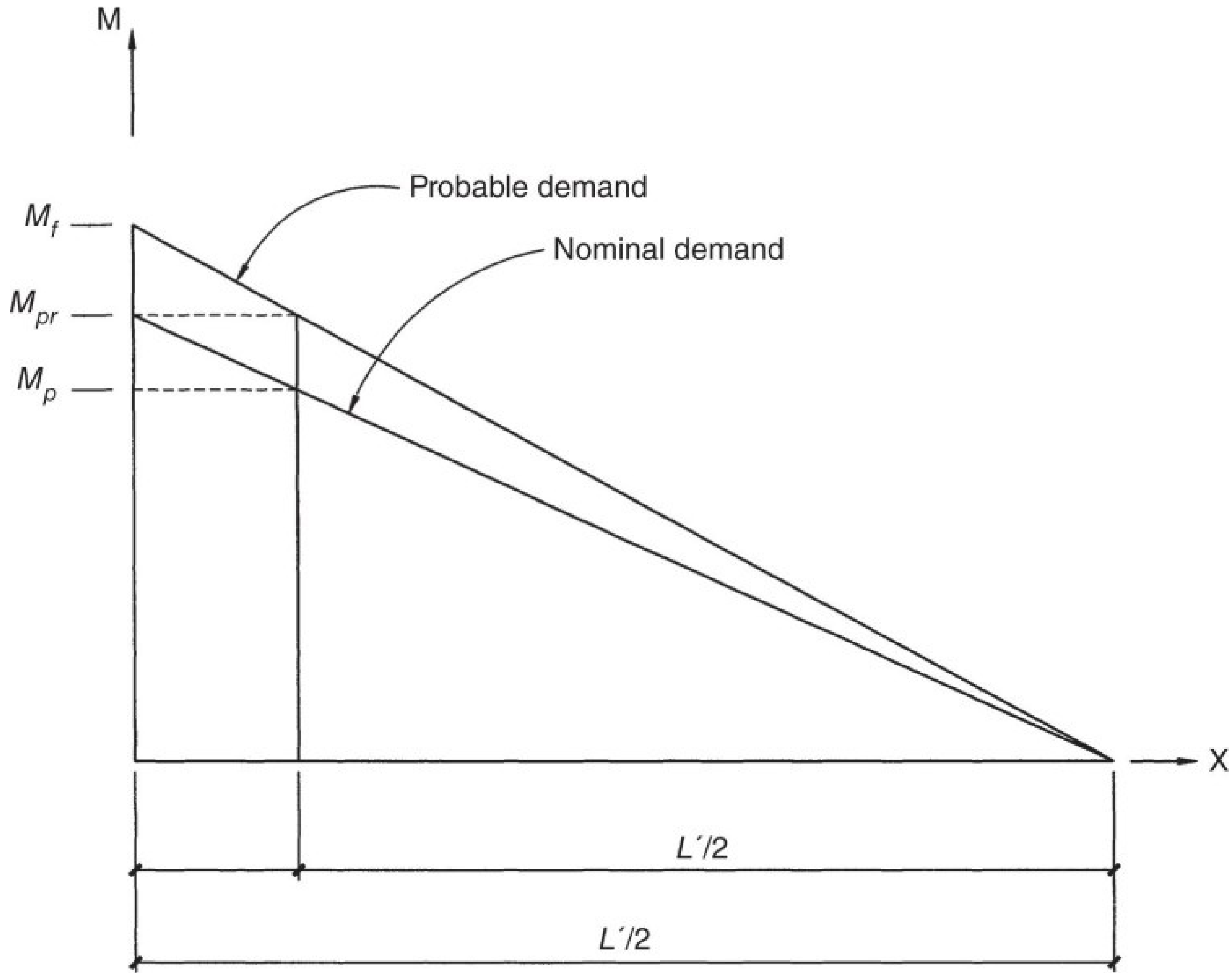

FIGURE 5.2 Beam seismic moment diagram.

1. Determine the maximum probable plastic moment of the beam, M_{pr}, including overstrength and strain hardening:

$$M_{pr} = C_{pr}R_yM_p = C_{pr}R_yZ_bF_y \tag{5.1}$$

where C_{pr} = 1.2 for A992 per AISC 358 (AISC, 2010).

2. Extrapolate the moment to the column face from the assumed seismic inflection point at beam midspan to find the maximum beam moment, M_f:

$$M_f = M_{pr} = V_px \tag{5.2}$$

where the shear

$$V_p = \frac{2M_{pr}}{L'} + \text{factored gravity loads at the hinge location}$$

$$M_f = M_{pr}\left(1 + \frac{2x}{L'} + \text{factored gravity moment at column face}\right) \tag{5.3}$$

3. The shear demand at the column face will be

$$V_f = \frac{2M}{L} + \text{ factored gravity loads at the column face} \tag{5.4}$$

Thus, the nominal capacity of the connection at the column face must be designed to resist the load demands M_f and V_f.

Deformation Capacities. Obtaining large story drift ratios in an SMF is dependent on the inelastic rotation capacity of the connections. This inelastic rotation may occur by hinging of the beam or column, or by shear yielding in the panel zone, or by a combination of these effects. As the strong column–weak beam (SCWB) is commonly preferred to weak column–strong beam (WCSB), the case of column hinging is not covered here. See Roeder et al. (1990) for further information on WCSB performance.

The story drift of a moment frame is closely related to the total joint rotation. This rotation is composed of both the elastic and inelastic deformations in the frame members (plastic hinges in the beam, shear yielding in the panel zone, etc.). Inelastic deformations in each component of the connection add cumulatively to the total plastic rotation of the connection. This parameter has become a valuable tool in determining the acceptability of connection designs. Connections that have exhibited adequate plastic rotation capacity in tests are generally expected to perform better in seismic events. Inelastic rotation demands may be estimated during the design using various nonlinear analysis techniques.

For special moment frame systems, AISC requires a minimum level of approximately 0.03 rad of plastic rotation (corresponding to the 0.04 rad drift angle) in a qualifying test that follows a specified loading protocol. This may be obtained by a combination of yielding in the beam, panel zone, or column. The ability of a connection to withstand such deformation without significant loss of strength is heavily dependent on ductile detailing of the entire connection region.

5.3.2 Post-Northridge Developments in Connection Design

Historically, moment connection design has relied on the previously described load-transfer assumptions and welded flange/bolted web connection details to allow the strength of the beam to fully develop prior to connection failure. Tests performed by Popov and Stephen (1970) and others indicated that this type of detail could be used for design, as the beam plastic moments were reached and, in some cases, significant amounts of ductility were observed. Indeed, it was believed that the typical steel SMF was well equipped to withstand large seismic force and deformation demands.

With the connection fractures caused by the Northridge earthquake came new questions related to this force transfer mechanism. Was the pre-Northridge connection detail fundamentally flawed? Can it be substantially improved by proper control over material and workmanship? Soon after the Northridge connection fractures were discovered, practitioners and researchers alike began to investigate these questions, and ultimately, to arrive at connection details that can be relied upon to deliver sufficient levels of force and deformation capacity.

Many successful testing programs were performed that now provide guidance and direction for SMF connection design to engineers. Full-scale testing has become an extremely useful tool in helping to understand SMF connection behavior.

For SMF and IMF AISC 341 requires the use of connection designs which have been proven to consistently perform well in tests. Due to the variation of member sizes, material strengths, and other variables between projects, project-specific testing programs may be needed. Alternatively, AISC provides specific acceptance criteria for using past test results of comparable connection designs (AISC, 2010), or AISC’s 358 Prequalification Standard (AISC, 2010).

One of three primary philosophies: (1) a toughening scheme, (2) a strengthening scheme, and (3) a weakening scheme have been used in the development of post-Northridge connection concepts. Often, some or all of these schemes are used in combination.

5.3.3 Toughened Connections

Design Philosophy. To toughen the connection, significant attention is paid to the complete-penetration weld details between the beam and the column. Notch-tough electrodes are now typically specified (a common requirement complete joint penetration beam flange to column flange welds is for Charpy V-notch values of 20 ft-lb at 0°F (27J @ –18C) and 40 ft-lb at 70°F (54J @ 21 C)). In addition, bottom flange backing bars are removed and replaced with reinforcing fillet welds in order to eliminate the notch effect at the root pass of the weld and to remove any weld flaws, which are more prevalent at the bottom flange where the beam web prevents continuous weld passes. At the top flange it is not common to remove the backing bar simply to add a reinforcing fillet to secure the bar to the column flange. Research performed by Xue et al. (1996) supports this approach.

This scheme may be used either as a stand-alone design method or as a supplement to either of the second two schemes. The use of a notch-tough electrode and corrective measures for the backing bar notch effect are critical components to any connection design. In short, taking such measures to toughen the groove weld is considered as a minimum amount of effort to ensure adequate ductile behavior of the connection, but likely do not fully meet the SMF rotation requirements. Other recommendations include improved control in welding and inspection practices.

Both the FEMA/SAC project and AISC studied the unreinforced connection in depth. Two key issues that were studied were the beam web connection and the configuration and preparation of the weld access holes adjacent to the beam flange welds. It was determined that in order to achieve SMF level inelastic rotation demands, the beam web to column connection should be a complete joint penetration weld. In addition the weld access hole preparation should take on a certain shape and size depending on the thicknesses of the beam flange and web. This configuration is depicted in the AISC *Seismic Provisions* (AWS, 2009). IMF performance can be achieved with a bolted web connection and the improved access hole configurations. These details were first provided in FEMA 350 (FEMA, 2000a), and were developed as prequalified connections by AISC in 2010.

Another important aspect of the connection is the addition of column continuity plates. Although the use of continuity plates has been based on member geometry for some time, it is now recommended that unless otherwise justified by testing, “continuity plates be provided and that the thickness be at least equal to the thickness of the beam flange for two-sided connections.” Welding of continuity plates to column flanges should be performed with full-penetration groove welds, while the plate-to-column web weld may be a double-sided fillet. Notch-tough electrodes should be used in all cases, and care should be taken to avoid welding in the *k* region of the column.

5.3.4 Strengthened Connections

Design Philosophy. Another method of ensuring sufficient connection capacity is by strengthening the portion of the beam directly adjacent to the column, where the maximum moment occurs during seismic loading. The increased capacity near the column flange, M_f, forces the plastic hinge to form in the unstrengthened section of the beam (see Fig. 5.3).

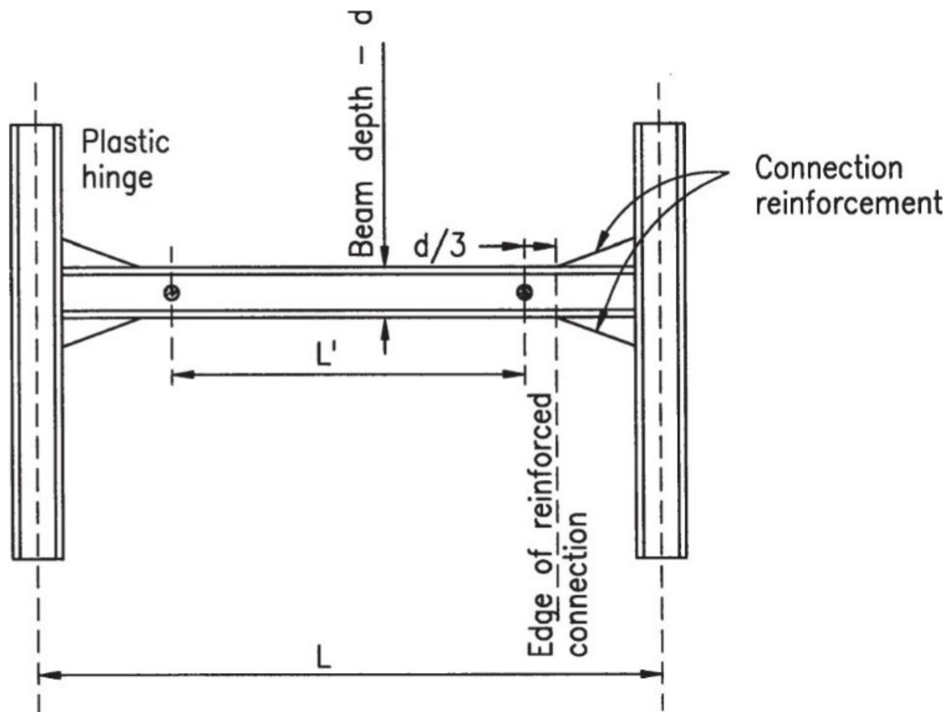

FIGURE 5.3 Location of plastic hinge in a one-bay frame.

The method used in this approach is to protect the previously vulnerable beam-flange complete-penetration welds with the addition of cover plates, rib plates, side plates, or haunches at the beam-to-column interface. The effective section modulus of the beam at the connection is increased, which decreases the bending stress at the extreme fiber of the section, as well as the total force resisted by the flange welds.

Strengthening these connections will invariably increase the stiffness of the frame. The effect this has on determining story drifts and building period must be considered in the design, but in most cases is relatively minor.

Another consideration is the satisfaction of the AISC requirements for panel zone strength and the strong column–weak beam condition. The extrapolated moment, M_f, can now be well above the beam plastic moment, M_p, and must be considered. The AISC *Seismic Provisions require* minimum level of panel zone strength so that the panel zone can share the inelastic response with the beam hinges.

Cover-Plated Connections. In the years immediately following the Northridge earthquake one popular method of strengthening the connection was to weld cover plates to the top and bottom beam flanges. Full-scale testing of cover-plated connections in was performed by Engelhardt and Sabol (1995), Noel and Uang (1996), and others. In general, these tests showed the ability of cover-plated connections to perform well in the inelastic range, and it was included in FEMA 350 (FEMA, 2000a).

Proper detailing is essential to obtain ductile behavior from a cover-plated connection. Typically, cover plates are fillet-welded to the beam flanges and groove-welded to the column flange. A common detail is shown in Fig. 5.4. For ease of field erection, the bottom cover plate is oversized and the top plate undersized, to allow for downhand welding at each location. A variation to this technique uses oversized top and bottom cover plates, with the top plate shop-welded to the beam and the bottom plate field-welded. This allows the use of wider plates, while allowing downhand welding at both locations.

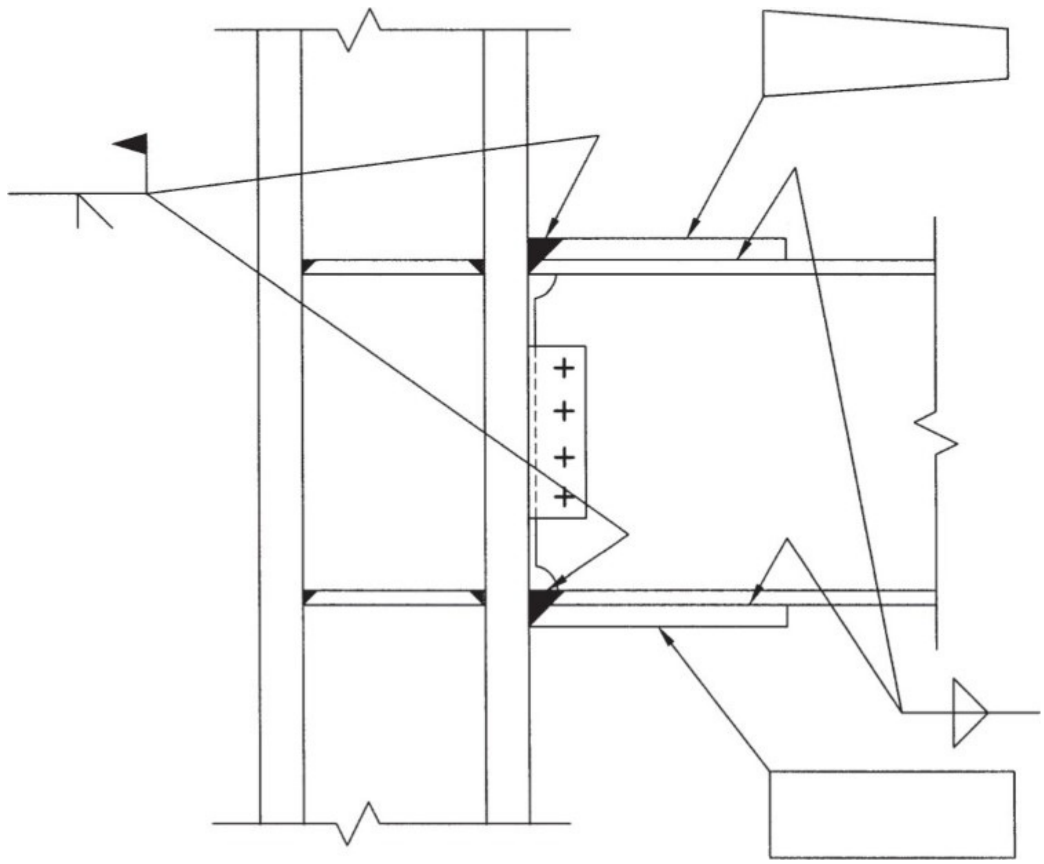

FIGURE 5.4 Cover-plate detail.

Note that only the long sides of the cover plates are welded to the beam flange. Welds loaded in the direction of their longitudinal axes perform significantly better in the inelastic region than those loaded in a perpendicular direction (AISC, 2010), hence cross-welds to the beam flanges at the end of the cover plates are not recommended.

Another detailing issue is the type of groove weld used at the cover-plate–to–column-flange connection. Two options are shown in Fig. 5.5 for this weld detail. Type I is the preferred detail. Although the type II detail uses less weld metal, the sharp angle of intersection between the cover-plate weld and the beam-flange weld creates a less desirable "notch" effect. From a fracture mechanics standpoint, the type II detail is more susceptible to horizontal crack propagation into the column flange. The designer must consider the amount of heat input and residual stresses in the joint region for either type detail. It is good practice to have a maximum total weld thickness of 2 times the beam flange thickness, or the thickness of the column flange, whichever is less. This is a means to conserve the amount of heat input to the welded joint region.

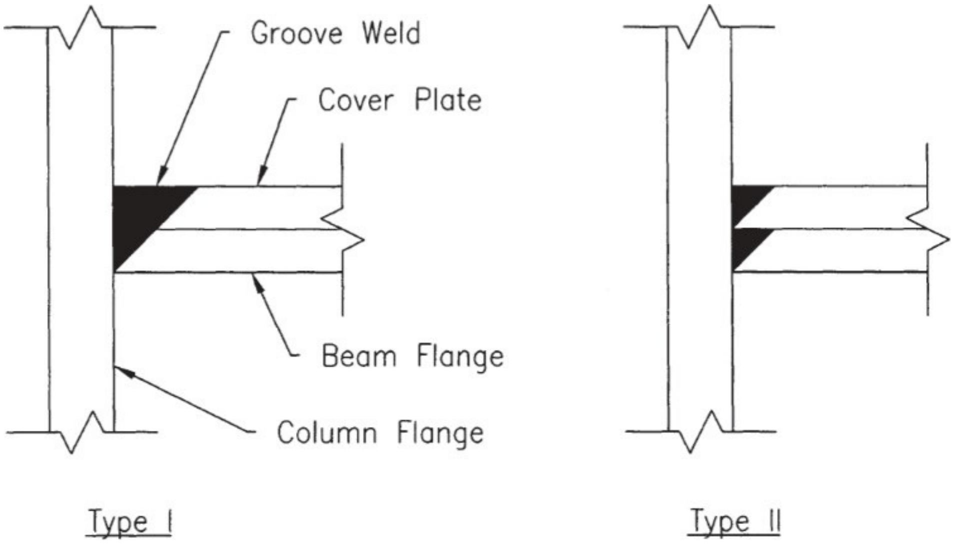

FIGURE 5.5 Cover-plate groove-weld types.

The thickness of the cover plate used is an essential variable to consider. The area of weld required between the strengthened beam section and the column face must be sufficient to resist the amplified beam moment, M_f. Once the required cross-sectional area of weld is obtained, it may be comprised of a combination of beam-flange weld and cover-plate weld or by plate weld alone if a thicker plate is used. The latter, known as a flange-plate connection, provides no direct connection of the beam flanges to the column flange, only to the cover plates. Full-scale tests of this type of connection were reported by Noel and Uang (1996) (see Fig. 5.6). If this option is chosen, care must be taken to avoid deformation incompatibility between the thin beam flange and relatively thick cover plate, resulting in premature fracture of the longitudinal cover-plate fillet welds.

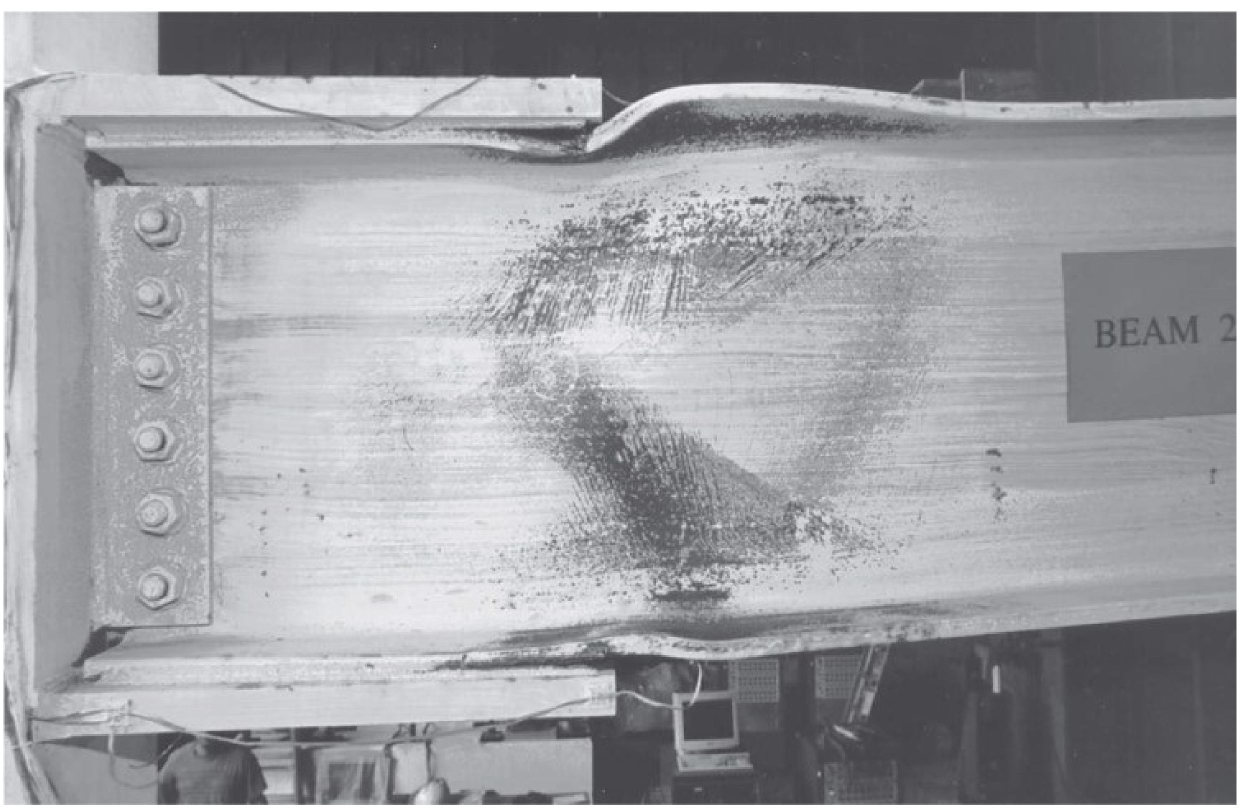

FIGURE 5.6 Cyclic performance of a flange-plated connection. (*Courtesy of Forell/Elsesser Engineers, Inc., San*

Francisco, CA.)

Haunched Connections. Another method of strengthening the connection is by the addition of a haunch at the beam-flange–to–column-flange connection. The haunch is typically located on the bottom flange only, due to the presence of the floor slab on the top flange. The addition of a haunch to both flanges is a more expensive option, but has been shown to perform extremely well in tests. Haunches are typically made from triangular portions of structural tee sections or built-up plate, and stiffeners are provided in the column and beam webs (see Fig. 5.7).

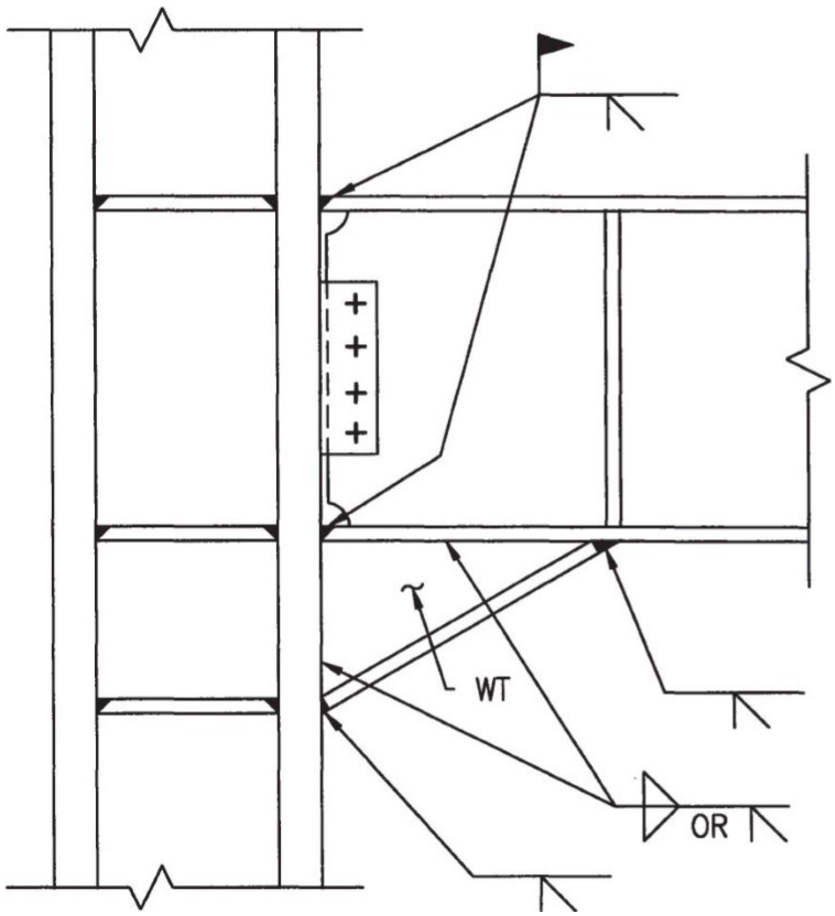

FIGURE 5.7 Bottom flange haunch connection.

Full-scale testing of bottom flange–welded haunch connections to date includes work done by Engelhardt et al. (1996), Popov and Stephen (1970), Uang and Bondad (1996), and Noel and Uang (1996). Whittaker et al. (1995) report good performance of connections made with top and bottom flange–welded haunches. The bolted haunch was studied by Ksai and Bleiman (1996). These details are also included in FEMA 350 (FEMA, 2000a).

Although work by Yu et al. (1997) questioned the validity of the classical beam theory bending stress ($f = Mc/I$) in haunch design, a number of test specimens designed using this theory have performed very well (see Fig. 5.8). The geometry of the haunch should be such that: (1) the moment, M_f, is resisted while satisfying the $0.9F_{yc}$ through thickness requirement and (2) the haunch aspect ratio is sufficient to develop adequate force transfer from the beam flange. A moderate balance is required here as the longer the haunch is, the higher the demand moment at the column face, M_f, becomes. The design methodology presented by Yu et al.

(1997) recognizes a more realistic force transfer mechanism in the haunch connection. In this approach, the haunch flange is modeled as a strut which attracts vertical beam shear, hence reducing the beam moment, M_f, and the tensile stress at the beam flange welds.

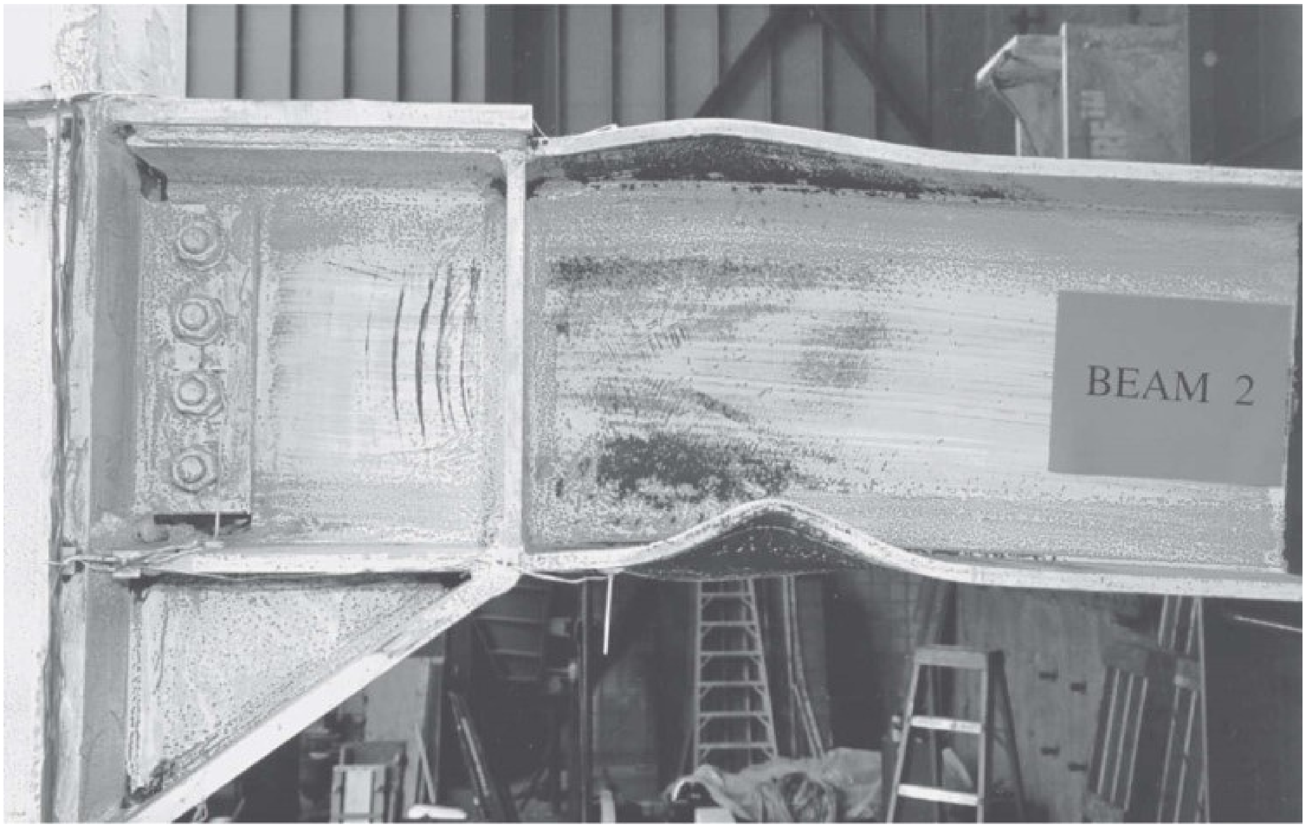

FIGURE 5.8 Yielding and buckling patterns of a beam subjected to cyclic loading. This connection incorporates both a bottom flange haunch and a top flange cover plate. *(Courtesy of Forell/Elsesser Engineers, Inc., San Francisco, CA.)*

Vertical Rib-Plate Connections. A vertical rib plate serves a similar purpose as the cover plate and the haunch; strengthening the section by increasing the section modulus while distributing the beam-flange force over a larger area of the column flange (see Fig. 5.11). The engineer may place a single rib plate at the center of the beam flange, but a more common practice is to position multiple ribs on each flange to direct the beam-flange force away from the center of the beam flange. By doing so, the stress concentration at the center of the beam-flange groove weld is somewhat alleviated.

Testing of rib-reinforced connections was been limited, but a few examples have shown that this method of strengthening can lead to ductile connection behavior (Engelhardt and Sabol, 1995; Anderson, 1995; Tsai and Popov, 1988).

It should be noted that while meeting the intent of providing substantial inelastic rotation performance can be met by the various strengthened connection approaches, they have not been widely used because of the extra fabrication and erection expense when compared to other details.

5.3.5 Weakened Connections

Design Philosophy. Weakening the connection is achieved by removing a portion of the beam flange to create a reduced beam section, or RBS (see Fig. 5.9). The concept allows the designer to “force” a plastic hinge to occur in a specified location by creating a weak link, or fuse, in the moment capacity of the beam. Figure 5.10 shows the moment diagram of a beam under seismic loading. The geometry of the RBS must be such that the factored nominal moment capacity is not exceeded, at the critical beam section adjacent to the column.

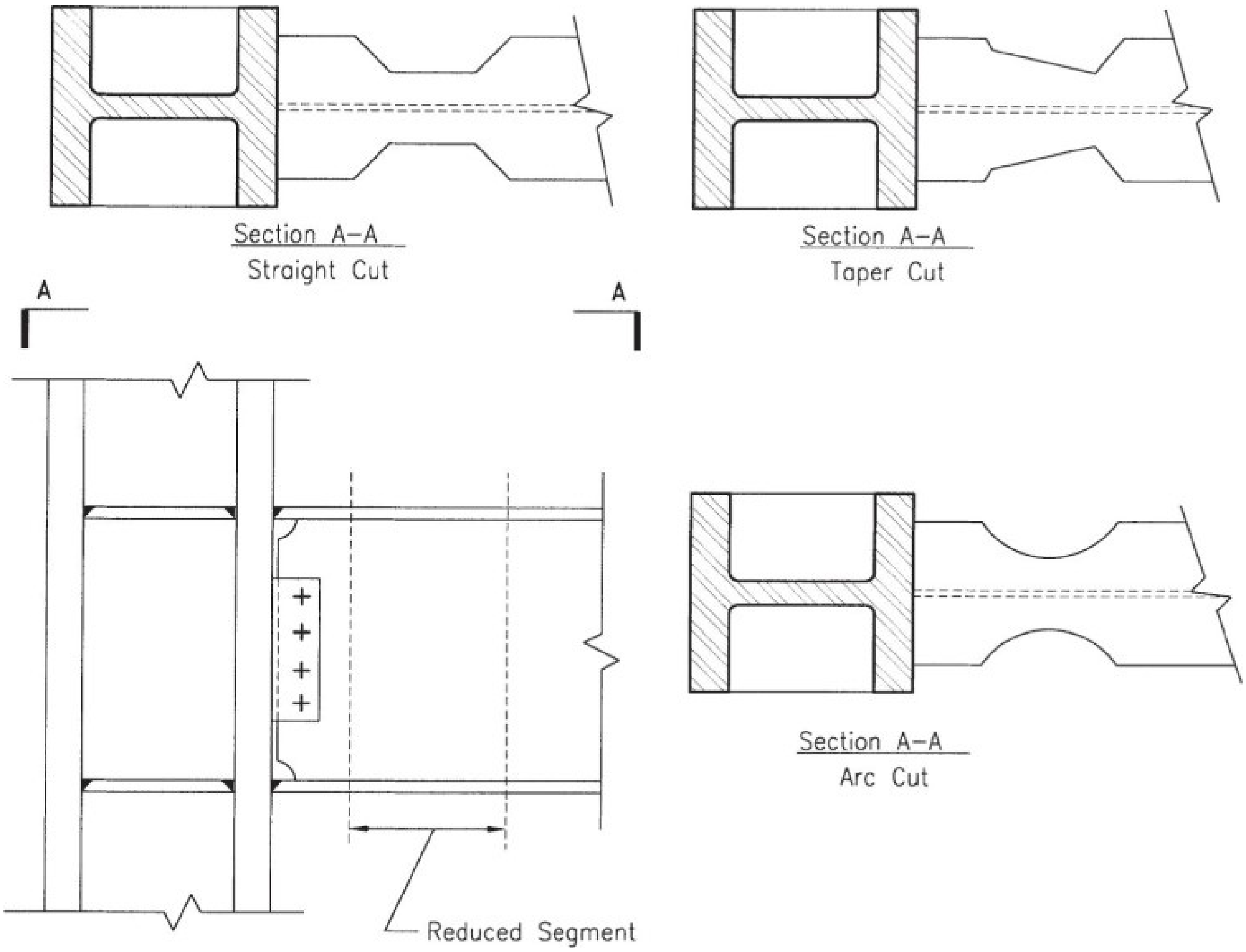

FIGURE 5.9 Reduced beam section connection.

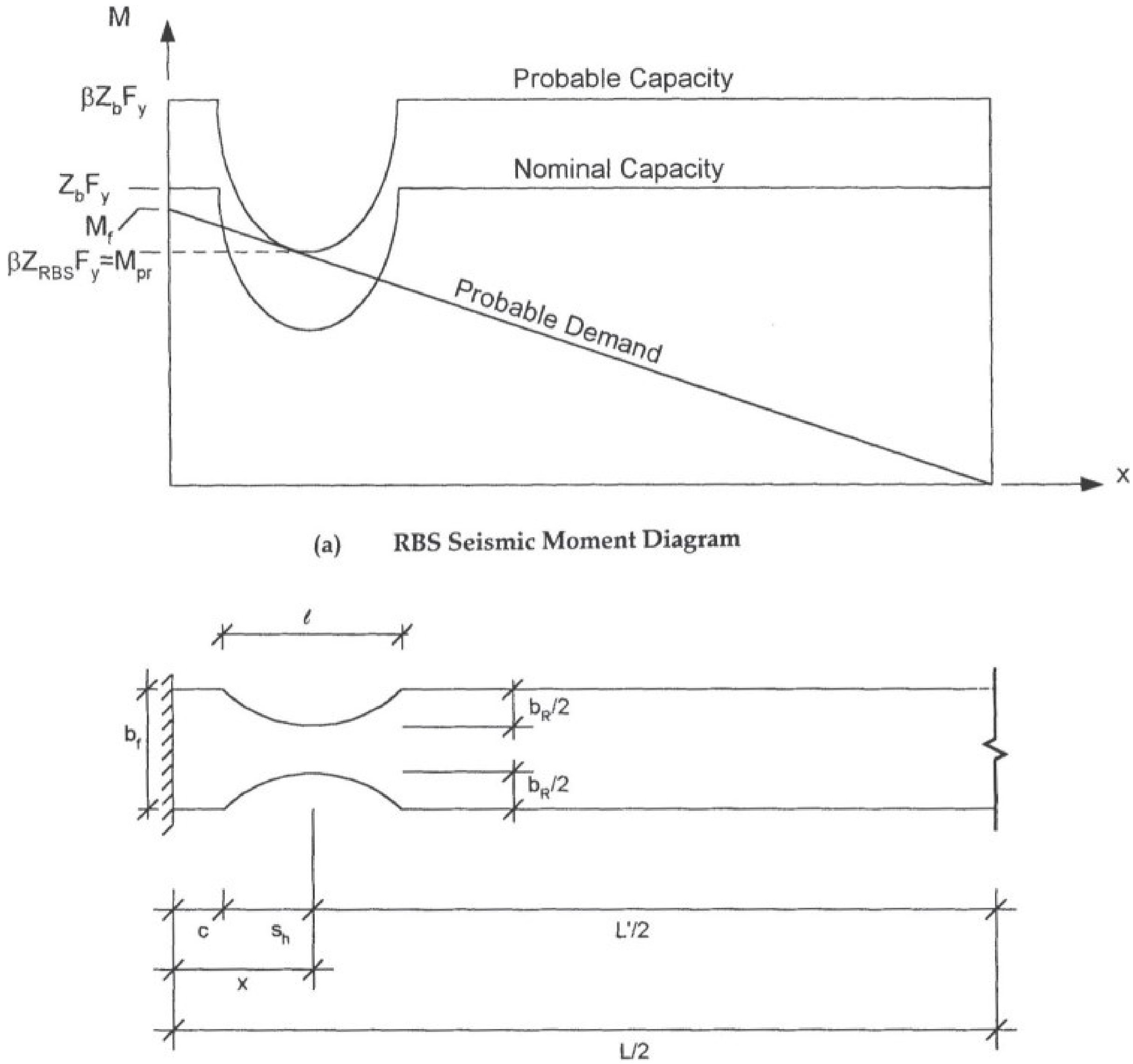

FIGURE 5.10 Reduced beam section moment diagram and flange geometry: (*a*) RBS seismic moment diagram and (*b*) RBS beam flange geometry (arc-cut section).

This method has potential benefits where the strengthening scheme had drawbacks. With a reduced M_p, the overall demand at the column face, M_f, must, by design, be less than the nominal plastic moment of the beam. Therefore, SCWB and panel zone strength requirements are easier to achieve.

The drawbacks for RBS come in the form of reduced stiffness. The reduction in overall lateral frame stiffness is typically quite small (typically on the order of a 5% to 7% reduction). On the other hand, the reduction in the flange area can significantly reduce the stiffness (and stability) of the beam flange, creating a greater propensity for lateral torsional buckling of the beam in the reduced section. The addition of lateral bracing is recommended for lateral bracing near the RBS may be required if a structural slab is not present or if above minimum acceptable performance is desired.

Choice of RBS Shape. The shape, size, and location of the RBS all can significantly affect the connection performance. In the early studies various shapes were tested, as noted schematically in Fig. 5.9. Test programs were performed to investigate straight cut (Engelhardt et al., 1996), taper cut (Ivankiw and Carter, 1996; Uang and Noel, 1996), arc cut

(Engelhardt et al., 1996) and drilled flanges.

Each RBS shape has benefits and shortcomings relative to each other. For instance, tapered cuts allow the section modulus of the beam to match the seismic moment gradient in the reduced region. This creates a reliable, uniform hinging location. However, stress concentrations at the reentrant corners of the flange cut may lead to undesired fracture at these locations as reported by Uang and Noel (1996) (see Fig. 5.11). Curved flange cuts avoid this problem, but do not give the benefit of uniform flange yielding, although test results indicate that plastification does distribute over the length of the reduced section.

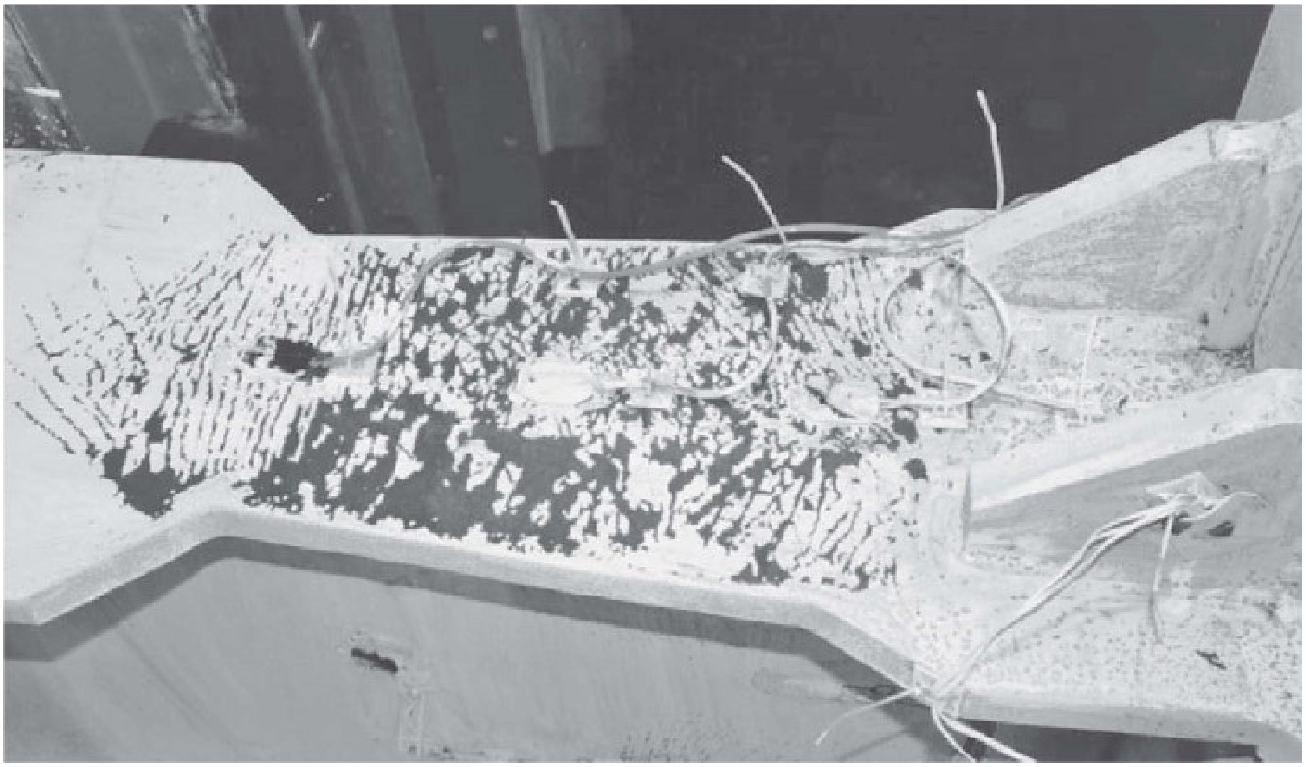

FIGURE 5.11 Yielding in the reduced section of a “taper-cut” beam flange subjected to cyclic loading. (*Courtesy of Ove Arup and Partners, Los Angeles, CA*.)

The lack of sharp reentrant corners and the ease of cutting made the circular arc-cut reduction a preferred option. In general, tests performed on arc-cut RBS connections have provided favorable results (Engelhardt et al., 1996). The design methodology presented by AISC 358 is applicable to curved arc reduction cuts and is now the preferred method for designing RBS connections.

Geometry Determination. Once a suitable bean size for frame drift is obtained, sizing the cut becomes the next obstacle. Keeping in mind the requirements for connection capacity at the face of the column (see Sec. 5.3.1), as well as gravity loading demands at the location of the RBS, the size of reduction must be chosen appropriately.

Since member sizes in SMFs are typically governed by drift requirements, it is initially assumed that the reduced section will still work for strength under gravity loading. This load case must be checked after the geometry is chosen based on seismic loading.

Given a beam span L, depth d, and hinge location s_h, the reduction variables l, c, and b_R (see Fig. 5.10) define the seismic moment gradient and may be tailored to satisfy the requirements described previously. The majority of RBS connection tests have used relatively similar values for these essential variables. For instance, the length of reduction, l, has typically ranged between $0.75d$ and d. The distance of the RBS away from the column face, c, has typically been chosen as approximately $0.25d$, however, work by Engelhardt et al. (1996) justifies using a value of 0.75 b_f. These values were shown to be effective in a number of testing programs AISC 358 provides complete guidance on the design of RBS connections.

The width of flange which is removed, b_R, determines the plastic modulus at the reduced section, $Z_{\text{RBS}} = Z_x - b_R t_f(d - t_f)$. This reduced modulus is then used to calculate the moment at the column face, M_f, using the method shown in Sec. 5.3.1. A practical upper bound on the value, b_R, has generally been 50% of the flange width, b_f. This limit is based on both stability and strength considerations. Excessive reduction can lead to premature lateral torsional buckling of the beam, which should be avoided. In the event that even a 50% flange reduction does not sufficiently reduce M_f. Supplemental strengthening may be considered in the area between the RBS and the column face. Reinforcing ribs at the column face have been shown to enhance the performance of RBS connections in tests (Uang and Noel, 1996).

Example 5.1: RBS Connection Design. Design an RBS connection between a W36 × 150 beam and a W14 × 426 column. The beam span in 30 ft. The flange reduction will be an arc-cut shape. We will use the guidelines of AISC 358 and gravity loads will be neglected (see Fig. 5.12).

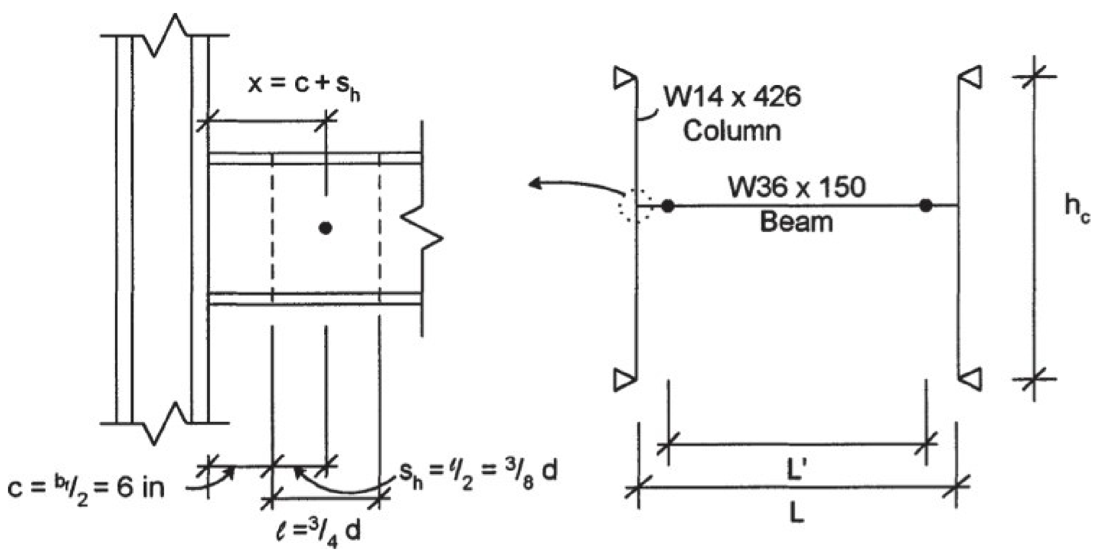

FIGURE 5.12 Frame and connection used in Example 5.1.

$$L' = L - d_c - 2x = 302 \text{ in}$$

$$M_{pr} = C_{pr} R_y Z_{RBS} F_y = 1.2(Z_{RBS})(50 \text{ ksi})$$

$$= 60 Z_{RBS}$$

$$V_p = \frac{2M_{pr}}{L'} = \frac{2(60 \text{ ksi}) Z_{RBS}}{302 \text{ in}}$$

$$= 0.39 Z_{RBS}$$

$$M_f = M_{pr} + V_p x = Z_{RBS}[60 + 0.39(19.5 \text{ in})]$$

$$= 67.6\ Z_{RBS}$$

- Find the required flange reduction:

$$Z_{req} = \frac{M_f}{50 \text{ ksi}} = 1.35\ Z_{RBS} \qquad Z_{req} = Z_b$$

$$Z_{RBS} \leq 0.74\ Z_b = 430 \text{ in}^3$$

$$Z_{RBS} = Z_x - b_R t_f (d - t_f)$$

$$430 \text{ in}^3 = 581 \text{ in}^3 - b_R(1 \text{ in})(35.85 \text{ in} - 1 \text{ in})$$

$$b_R \geq 4.3 \text{ in}$$

$$\frac{b_R}{b_f} = \frac{6 \text{ in}}{12 \text{ in}} = 50\% \text{ reduction, ok}$$

$\therefore$ Try a 6-in flange reduction

$$Z_{RBS} = 372 \text{ in}^3 (6100 \text{ cm})^3$$

$$M_f = 67.6(372) = 25{,}147 \text{ kip-in } (2.8 \times 10 \text{ to the 6th Nm})$$

- Check the through-thickness stress:

$$f_{tt} = \frac{M_f}{S_b} = \frac{25{,}147}{504} = 49 \text{ ksi (340 MPa)} = \phi F_{ye} = 0.9(54 \text{ ksi}), \text{ ok}$$

$\therefore$ Use a 6-in beam-flange reduction (50%) (see Fig. 5.13).

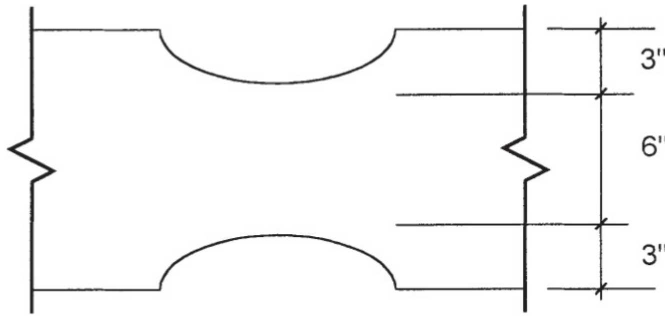

FIGURE 5.13 RBS flange reduction.

- Check that panel zone strength and SCWB requirements will meet the requirements of the AISC *Seismic Provisions*.
- Continuity plates:
- Add 1-in-thick continuity plates at the top and bottom flange level, to match the beam flange assuming a two-sided frame configuration.

It should be noted that the preceding discussion presents some, but not all, of the connection design approaches that have been developed since the Northridge earthquake. In fact, a few approaches were patented and have been widely used; these patented connections have not been addressed here. AISC 358 now presents numerous options for moment connections including some of the proprietary connections. It is recommended that designers begin by reviewing this document in depth in order to select a connection approach that is most appropriate for the project.

5.4 CONCENTRICALLY BRACED FRAMES

5.4.1 Introduction

Concentric braced frames have found wide application in lateral force-resisting systems, typically having been chosen for their high elastic stiffness. This system is characterized by horizontal and vertical framing elements interconnected by diagonal brace members with axes that intersect. The primary lateral resistance is developed by internal axial forces in the framing members. The AISC *Seismic Provisions* make a distinction between ordinary concentrically braced frames (OCBF) and special concentrically braced frames (SCBF). SCBF frames are specifically detailed and typically sized to withstand the full inelastic behavior of the lateral system. This section will describe the connection design for both types of concentric braced frames. Another excellent reference for SCBF design is NIST Technical Brief Number 8, "Seismic Design of Special Concentrically Braced Frames: A Guide for Practicing Engineers" (NIST, 2013).

Figure 5.14 shows several types of braced frames. The V-braced systems shown require the intersected beams to be specially designed when used in SCBF structures in order to ensure their stability once the bracing system begins to exhibit inelastic behavior. During a large earthquake, it is expected that the compression brace will buckle before the tension brace begins to yield. At the connection to the beam, there is an imbalance of forces from the braces that needs to be resolved by the beam member. As the lateral loading continues and both braces yield, the maximum force imparted to the beam will be the difference in the strengths of the buckling brace and the tension yielding brace. The direction in which this force acts depends on the bracing configuration. The brace connections and the beam need to be able to transfer these loads.

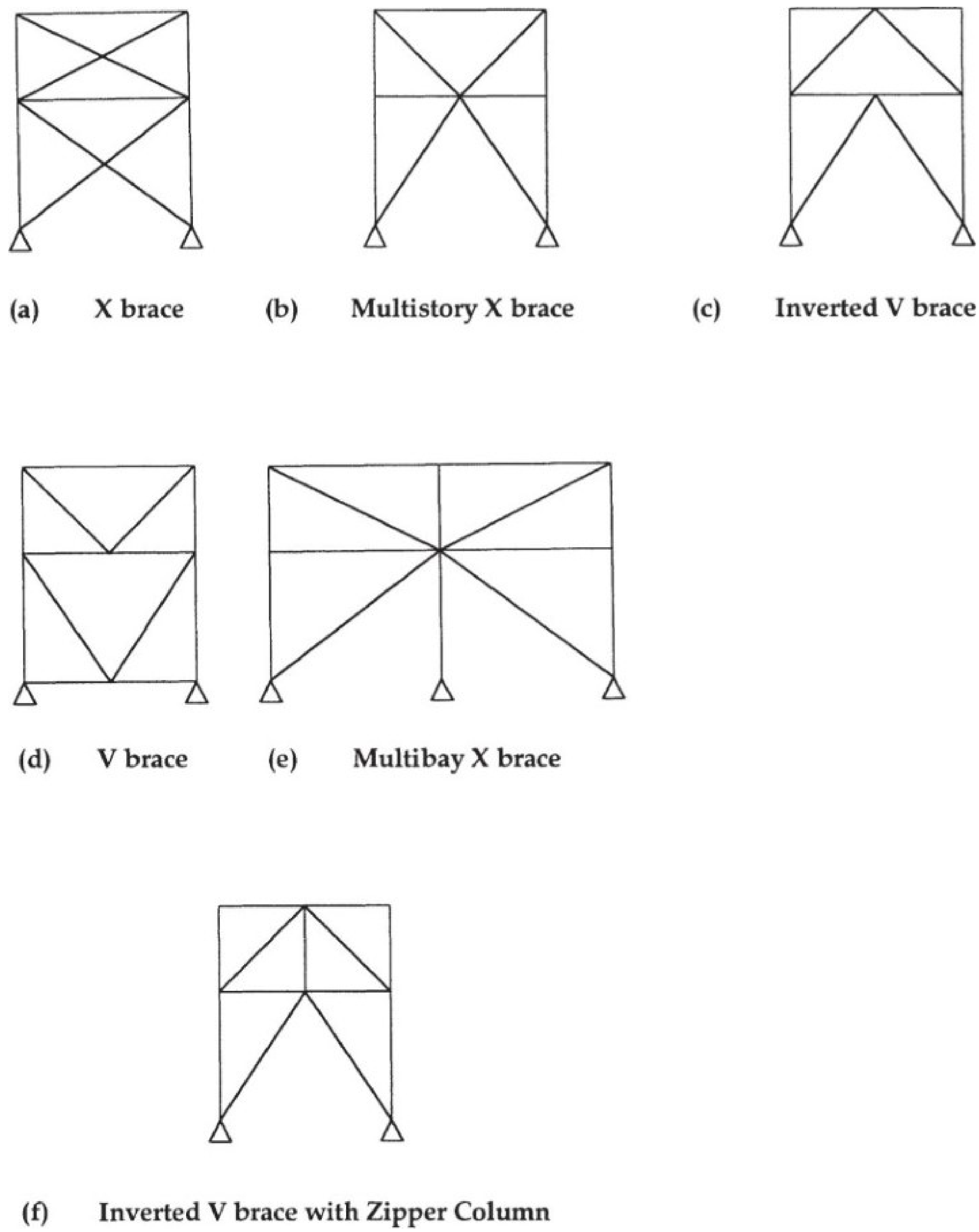

FIGURE 5.14 Concentric braced frame types: (*a*) X brace; (*b*) multistory X brace; (*c*) inverted V brace; (*d*) V brace; (*e*) multibay X brace; and (*f*) inverted V brace with zipper column.

SCBF systems have different requirements than OCBF systems. The width-thickness ratio of the rectangular hollow structural shape (HSS) sections in SCBFs is limited to $\sqrt{0.64}\,E/F_y(F_y)$. This is intended to minimize local buckling of the brace elements and results in larger wall or flange thickness. Since the connections are typically designed for the brace capacity in SCBF systems, the force level for the design of the connection will increase.

Due to the better behavior of the system, AISC allows more slender elements in SCBFs than in OCBFs. The slenderness of OCBFs are limited to $KL/r \le 4\sqrt{E/F_y}$, whereas SCBF braces are only limited to $KL/r \le 4\sqrt{E/F_y}$. This seems to contradict testing which has shown that the hysterectic response during inelastic cyclic reversals improves as the slenderness of the compression member decreases. Locally, brace behavior is improved with stocky members, however, inelastic analyses which analyze the entire system indicate that large reductions in the slenderness can cause the compression capacity to approach the tension capacity, which results in a soft story effect. This will occur if, once the compression braces of a story buckle, the tension members on the same story yield before compression members on other floors buckle. Since the buckling strength is close to the tension capacity, the postbuckling reduction in strength is often enough to yield the adjacent tension members. The addition of a “zipper column” as shown in Fig. 5.14, avoids this condition by better distributing the forces throughout the height of the system as the members exceed their elastic limits.

5.4.2 Connection Design and Example

This section will present an example design of a connection in a special concentrically braced frame. Throughout the example, it will be noted how the criteria would differ for an ordinary concentrically braced frame. Figure 5.15 shows the brace configuration and Example 5.2 presents a spreadsheet outlining the entire connection design. The connection presented addresses the intersection of a HSS steel brace with a beam-column connection. Similar approaches may be followed for other brace configurations and section types.

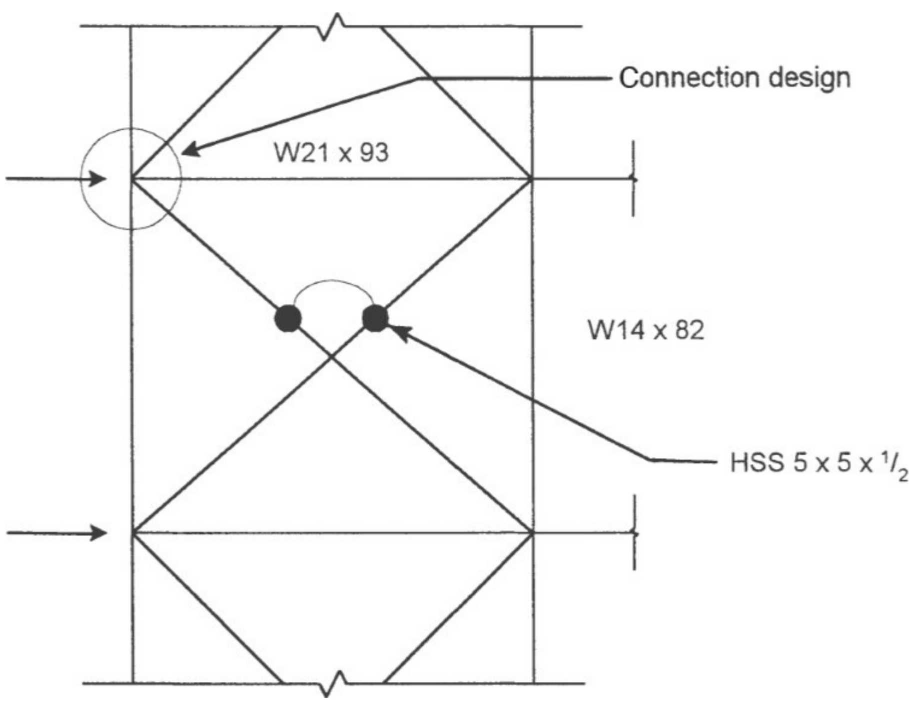

FIGURE 5.15 Design example.

Force Level. The design of the connection is dependent on the design forces during compression and tension. Designing the connection for the capacity of the member ensures the connection is not the yielding element in the system. The maximum force the connection will be subject to is the yielding of the brace member in tension defined as $R_y F_y A_g$.

The R_y factor accounts for the expected material overstrength and strain hardening of the member. Had this connection been designed for use in an OCBF, the design force level could have been reduced to the maximum expected force as defined by the maximum force that can be delivered by the system, or a load based on the Amplified Seismic Load in ASCE 7 (ASCE, 2010).

The spreadsheet analysis begins by determining the section sizes, material, and geometry which will determine the brace's force magnitude and direction at the connection.

Force Distribution. The force distribution from the brace to the beam and column can be calculated using the uniform force method described in AISC publications. This method provides a rational procedure for determining the interface forces between the gusset plate and the horizontal and vertical elements at the connection. The axial and shear forces are distributed in the connection based on stiffness, while the required moment for equilibrium is assigned to the beam or column or to the beam and column equally.

An alternate method to determine the forces in the connection may use the fin truss approach originally proposed by Whitmore and modified by Astaneh (1989). This approach discretizes the gusset plate into radial elements and distributes the force based on axial stiffness and the angle of incidence. The procedure has been applied successfully on single-member connections, but appears overly conservative for multimember gusset-plate configurations when the forces are not independent of one another.

Example 5.2 defines the geometry of a rectangular plate, where the 2*t* offset required to allow an unrestrained bending zone of the brace is provided between the points of support of the plate to the beam and column and the end of the brace. This configuration not only provides a simple plate geometry, but also eliminates the need for stiffeners on the plate. Had the plate utilized smaller leg dimensions, a tapered plate would be required, but the buckling line perpendicular to the brace would start from the bottom edge of the plate upward to a free edge of the plate. This free edge should be supported by stiffener plates to ensure that during buckling the plate remains stable and bends perpendicular to the brace. Should the buckling line migrate to the stiff supported points not perpendicular to the brace, such as the ends of the tapered plate without stiffeners, it is feared that the brace may effectively buckle about a different axis at each end imparting torsional forces into the brace. Figure 5.16 shows this condition.

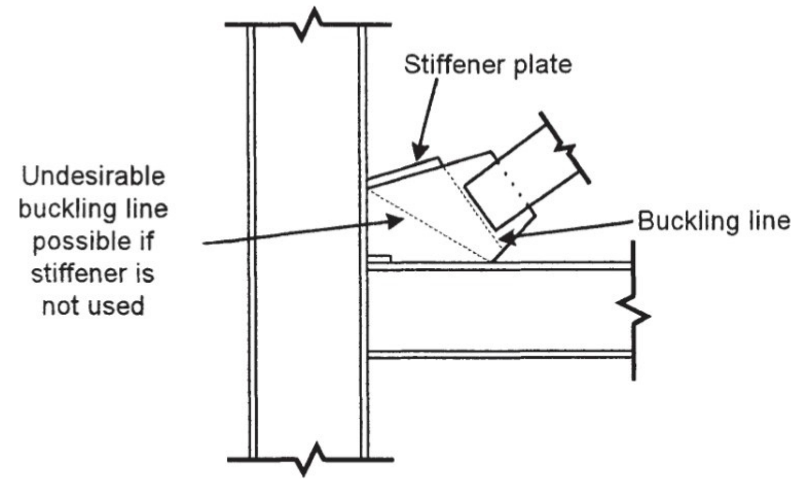

FIGURE 5.16 Tapered gusset plate with stiffeners.

Example 5.2 shows the dimensions calculated based on the geometry specified and the resulting load distribution using the uniform force method. Figures 5.17 and 5.18 show the geometric variables. The axial force on the beam and the shear force on the column can be significant from the gusset plate. In frames where the brace intersects the column from each side or the beam from top and bottom, large demands may overstress the section requiring either the size be increased or the webs be strengthened with doubler plates.

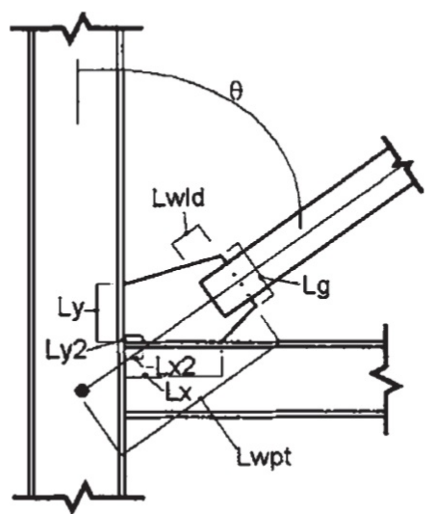

FIGURE 5.17 Gusset-plate connection geometry.

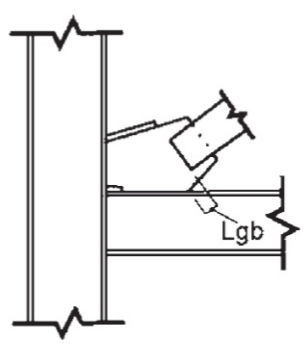

FIGURE 5.18 Gusset-plate distance requirements.

Brace–to–Gusset-Plate Connection. The connection of the HSS brace to the gusset plate uses four fillet welds along a slot to fit the gusset plate. Half of the force is transferred to the plate by each half of the HSS section. The centroid of the half section is no longer at the centroid of the plate, but rather is offset from the face of the plate toward the remaining wall of the tube section. The eccentricity between this centroid and the weld to the plate creates bending along the length of the welds. Of course, equal and opposite bending exists on the other side of the plate resulting in no net bending on the plate provided the HSS is slotted along its centerline. The welds must be sufficiently strong to resist this bending stress. Damage to similar connections was found after the Northridge earthquake where the lack of sufficient weld to resist this bending resulted in the sides of the tube peeling away from the gusset plate. Welds may be strengthened by either increasing their thickness or length. Although increasing the length is the most efficient locally, it may increase the gusset-plate size and connections to the beam and column depending on the configuration.

The connection of the brace to the gusset plate is also subject to block shear. For the HSS steel brace, the plate may yield around the perimeter of the full HSS section (two lines of shear and one of tension) or along each weld line in shear (four lines of shear). The HSS section may also yield along the HSS walls (four lines of shear). Other section types would have similar mechanisms.

Another consideration is the reduced net section of the brace resulting from the slot that is provided for the weld between the gusset plate. This net section has caused failure of tested braces, and should therefore be reinforced. Common means of such reinforcement are added plates that are shop welded to the vertical faces of the HSS members.

Gusset-Plate Design. The gusset plate may either allow for out-of-plane rotation of buckling braces or may restrain the brace elastically. The design philosophy chosen will affect the slenderness ratio used for the brace. If the connection is not capable of restraining the rotation of the buckling brace, an effective length factor, K, of 1.0 is used. If, however, the connection can restrain the bending demands of the buckling brace a smaller value of K may be used. The connection must then be strong enough to restrain the bending capacity of the brace taken as $1.1R_yM_p$ about each axis. Although more efficient brace members may be utilized, more robust connections will be required which will at least partially offset the material savings.

Example 5.2: Design of Special Concentric Braced Frame Connection

Calculation of special concentric braced frame connection to accomodate brace buckling behavior and conform to the *AISC Seismic Provisions for Structural Steel Buildings* *Uniform Force Method*

(INPUT IS INDICATED BY BOLD ITALICS)

Global Input

General

Title: ***TS5x5x1/2 Brace***

Global Data

Location:	(U)pper / (L)ower =	***L***		

Brace Data

Shape:	Shape =	***TS5x5x.5***		
Orientation:	(S)trong / (W)eak	***W***		relative to the out of plane direction
K_ip:	X-X eff. len. fact. =	***1.50***		brace effective length factor in the plane of the frame
K_op:	Y-Y eff. len. fact. =	***1.00***		brace effective length factor out of the plane of the fram
Fybr:	Yield =	***46***	**ksi**	
Ry:	Overstrength =	***1.1***	**kips**	1.5 for A36, 1.3 for A992, 1.1 for other grades
Put:	Ry*Fy*Ag =	**423**	**kips**	
λ:	KL/(r*pi)*sqrt(Fy/E) =	**1.159**		(Out-of-Plane Buckling Governs)
Pcr:	0.658^(LAMBDA^2)*A*FY	219.18		

Upper beam

Shape:	Shape =	***W21X93***	
TOS:	Top of steel =	***10.00***	**ft**
Fyub:	Yield =	***50.00***	**ksi**

Lower beam

Shape:	Shape =	***W21X93***		
TOS:	Top of steel =	***0.00***	**ft**	
Fylb:	Yield =	***50.00***	**ksi**	
X:	Bay width =	***10***	**ft**	Horizontal length between work points

Column

Shape:	Shape =	***W14X82***	
Fycol:	Yield =	***50.00***	**ksi**
Orientation:	(S)trong/(W)eak =	***S***	

Member Properties Summary

		Brace:	
	section =	TS5x5x.5	
Abr:	area =	8.36	in^2
dbr:	depth =	5.0	in
bfbr:	flange width =	5.0	in
twbr:	wall thick. =	0.5	in
tfbr:	wall thick. =	0.5	in
rxx:	X-X rad. of gyr. =	1.8	in (strong axis)
ryy:	Y-Y rad. of gyr. =	1.8	in (weak axis)

		Upper beam:	
	section =	W21X93	
Abm:	area =	27.3	in^2
dbm:	depth =	21.62	in
bfbm:	flange width =	8.42	in
kbm:	k =	1.6875	in
twbm:	web thick. =	0.58	in
tfbm:	flange thick. =	0.93	in
k1bm:	k1 =	0	in

		Lower beam:	
	section =	W21X93	
Abm:	area =	27.3	in^2
dbm:	depth =	21.62	in
bfbm:	flange width =	8.42	in
kbm:	k =	1.6875	in
twbm:	web thick. =	0.58	in
tfbm:	flange thick. =	0.93	in
k1bm:	k1 =	0	in

		Column:	
	section =	W14X82	
Acol:	area =	24.1	in^2
dcol:	depth =	14.31	in
bfcol:	flange width =	10.13	in
kcol:	k =	1.625	in
twcol:	web thick. =	0.51	in
tfcol:	flange thick. =	0.855	in
k1col:	k1 =	1	in

Connection Geometry

Lwpt:	Length from W.P. to end of gusset along brace =	***42***	**in.**
Ly:	Gusset dim. along col. from face-of-beam =	***22***	**in.**
Ly2:	Face of beam to start of column-gusset weld =	***5***	**in.**
Lx:	Gusset dim. along beam from face-of-column =	***21.07***	**in.**
Lx2:	Face of Column to start of beam-gusset weld =	***5***	**in.**
Lwld:	Weld length for brace flanges-to-gusset pl. =	***12***	**in.**
Lg:	Gusset dimension perpendicular to brace =	***9***	**in.**
t:	Gusset thickness =	***0.75***	**in.**
Fy_gus:	Gusset yield stress =	***36***	**ksi**
Fu_gus:	Gusset Ultimate stress =	***58***	**ksi**
θ:	Theta =	**0.785**	**rad.**
α:	Length to mid-gusset along bm. = (Lx-Lx2)/2+Lx2 =	**13.035**	**in.**
β:	Length to mid-gusset along col. = (Ly-Ly2)/2+Ly2 =	**13.5**	**in.**
eb:	dlb / 2 =	**10.81**	**in.**
ec:	dcol / 2 =	**7.155**	**in.**
L_unb:	Approximate unbraced length of brace =	**9.14214**	**ft.**

To specify a rectangular plate

Lx req'd =	25.73	**in.**
Ly req'd =	22.07	**in.**

Note: The brace length is calculated assuming that the same columns are used on each side of the bay and that the basic gusset geometry is the same at both ends of the brace. This length is used to calculate the brace compression capacity.

Bending Zone

t:	Gusset thickness =	0.75	**in.**
2t:	Current value for twice the gusset thickness =	1.5	**in.**
Lgb:	Bending at the beam-to-gusset connection =	2.40	**in. >= 2t OK**
Lgc:	Bending at the column-to-gusset connection =	1.74	**in. >= 2t OK**

Compactness, Local Buckling

KL/r: (Out-of-Plane Buckling Governs)	Maximum slenderness ratio =	91.4214	**< 1000/sqrt(Fy) OK**
b/t:	Compactness =	10	**< 110/sqrt(Fy) OK**

Moment Reduction I

	Current	Req'd for Zero Mom.	
Lx =	21.07	29.31	**in.**
Ly =	22.00	13.76	**in.**
α =	13.04	17.16	**in.**
β =	13.50	9.38	**in.**

Connection Stiffness

Any moment required for equilibrium will be carried by the stiffer of the beam-to-gusset and the column-to-gusset connection. The designer can choose roughly how to distribute the forces to the framing members.

Distribute forces:

(E)qually to Beam and Column, Primarily to (B)eam, or Primarily to (C)olumn = ***E*** (Enter E, B, or C only)

Moment Reduction II

	Current	Req'd for Zero Mom.		
Lx =	21.07	25.05	**in.**	
Ly =	22.00	17.74	**in.**	
α =	13.04	15.02	**in.**	**Moment on interface**
β =	13.50	11.37	**in.**	**Moment on interface**

Load Distribution Manipulation

The beam and column shear capacities are checked for the additional shear being introduced into them. Beams may also need to be checked for additional bending stresses.

Load Adjustments — **CONCLUSIONS**

Vbm_r:	Reduce shear in beam by :	***0***	**kips**
Hcol_2:	Other horizontal shear into column :	***0***	**kips**
Vbm_2:	Gravity + other seismic shear into beam :	***0***	**kips**

Final Interface Forces

Gusset-to-Column

Fvcol:	Phi*Aweb*Fv = 0.9x0.6 x fycol x dcol x twcol =	197.049	**kips**	
Hcol:	Horizontal force on column = Ec*Pu/r	96.50	**kips**	
Hcol_Total	Total column shear = Hcol + Hcol_2 =	96.50	**kips**	**OK < Fvcol**
Vcol:	Vertical force on column = Beta*Pu/r	153.32	**kips**	
Mc:	Moment from gusset on column = Huc*(Beta-Beta_bar)	-205.76	**in.-kips**	

Gusset-to-Beam

Fvbm:	Phi*Aweb*Fv = 0.9x0.6 x fybm x dbm x twbm =	338.569	**kips**	
Hbm:	Horizontal force on beam = Alpha*Pu/r	202.62	**kips**	
Vbm:	Vertical force on beam (incl. Vbm_r red.) = Eb*Pu/r - Vbm_r	145.80	**kips**	
Vbm_Total	Total beam shear = Vbm + Vbm_2 =	145.80	**kips**	**OK < Fvbm**
Mbm:	Moment from gusset on beam =	289.82	**in.-kips**	
fbm:	Bending stress on beam = Mb/S	1.51	**ksi**	**OK < Fybm**

Brace-to-Gusset Connection
This section designs the welded connection between the tube walls and the gusset plate. This section applies for TS sections only. Additional and/or different checks will need to be performed for other shapes.

Brace Flange-to-Gusset Connection

	Electrode =	***E70***		
	Electrode correction =	1	**[LRFD 2nd ed. p8-158]**	
FLFORCE:	Design force in one side of tube = (Pu / 2)=	211.5	**kips**	
efl:	One side eccentricity =	1.88	**in.**	
a:	efl / Lwld =	0.156	**in.**	
c:	Interpolate from weld table values =	2.736	**[LRFD 2nd ed. p8-163]**	
GBlockTear:	Min of 0.75*(0.6*Fy*Agv+Fu*Ant) and 0.75*(0.6*Fu*Anv+Fy*Agt)	454.725	**kips**	**OK > Pu**
Gtear:	Gusset tearout cap. = 0.75x0.6 x Fu x t x Lwld x 2 =	470	kips	**OK > FLFORCE**
TStear:	Tube wall tearout = 0.9x0.6 x Fybr x tfbr x Lwld x 2 =	298	kips	**OK > FLFORCE**
	Required fillet weld size =	6.44	**16ths in**	

Eccentric weld table

a	*c*
0.150	2.750
0.200	2.640

Design of Gusset Plate
Check the tension and buckling capacity of the gusset plate.

CONCLUSIONS

Design Data

	Slope (m)	Y intercept y = mx + b
Line A	-1.00	42.43
Line B	3.37	-84.41
Line C	3.73	-89.50
Line D	0.00	32.78
Line E	0.27	23.98
Line F	1.00	3.66

Line Intersect.	Xint	Yint
Lines A & B	29.00	13.43
Lines A & C	27.88	14.55
Lines A & D	9.61	32.82
Lines A & E	14.55	27.88
Lines A & F	19.39	23.04

Tension

Weff:	Effective width of gusset for tension =	18.86	**in.**	
Tcap:	Capacity of gusset in tension = (0.9*Fy x t x Weff) =	458.21	**kips**	**OK > Pu**

Compression

K:	Gusset plate effective length factor =	***0.80***		
	Effective width of gusset for compression =	18.86	**in.**	(per Whitmore's area)
L:	Average Length for Compression =	17.30	**in.**	(used to calculate gusset
ry:	Radius of gyration (weak way) =	0.22	**in.**	comp. capacity)
λ:	Lambda (gusset) = KL/r*pi*sqrt(FY/E)	0.72		
Fcr:	Fcr for gusset =	29.03	**ksi**	
Pcap:	Compression capacity of gusset = 0.85*Ag*Fcr =	349.02	**kips**	**OK > Pbuckle**

Design of Gusset-to-Beam Connection
Design of the gusset-to-beam coonection as a welded connection.

Gusset-to-beam interface forces

Hbm:	Horiz. force at the gusset-beam interface = Hb	202.62	**kips**
Vbm:	Vertical force at the gusset-beam interface = Vb	145.80	**kips**
Mbm:	Moment at the gusset-beam interface = Mb	289.82	**in.-kips**

Gusset Stresses

Fvcap:	Gusset shear capacity =0.9x0.6 x Fy =	19.44	**ksi**	
fvult:	Shear stress on gusset = Hbm / (t x (Lx-Lx2)) =	16.81	**ksi**	**OK < Fvcap**
fault:	Axial stress = Vbm / (t x (Lx-Lx2)) =	12.10	**ksi**	
fbult:	Bending Stress = Mbm x 6 / (t x (Lx-Lx2)^2) =	8.98	**ksi**	
	Combined normal stress = fault + fbult =	21.07	**ksi**	**OK <Phi* Fy**

Weld Stresses

Average ultimate stress =	8.86	**ksi/in.**	
Peak ultimate Stress =	10.11	**ksi/in.**	
Peak stress / Average stress =	1.14	**< 1.4**	If this ratio is less than 1.4, size the weld for 1.4 times the average ultimate stress.
Implied orientation of stresses =	0.89747	rad	
Capacity increase for orientation =	1.35		

Beam Web Stresses

Beam web stress =	19.67	**ksi**	**OK < Fybm**

Weld Design Gusset-to-Beam

Electrode =	***E70***	
Electrode correction (LRFD 2nd ed. p8-158) =	1	
Required fillet weld size =	6.62	**16ths in.**

Design of Gusset-to-Column Connection — **CONCLUSIONS**

Design a fillet weld on each side of the gusset plate to the column.

Interface Forces

	Column Orientation =	Strong	
Hcol:	Horiz. force at the gusset-column interface = Hc	96.50	**kips**
Vcol:	Vertical force at the gusset-column interface = Vc	153.32	**kips**
Mcol:	Moment at the gusset-column interface = Mc	-205.76	**in.-kips**

Gusset Stresses

Fvcap:	Gusset shear capacity = 0.9x0.6 x Fy =	19.44	**ksi**	
fvult:	Shear stress on gusset = Vcol / (t x (Ly-Ly2)) =	12.03	**ksi**	**OK < Fvcap**
fault:	Axial stress = Hcol / (t x (Ly-Ly2)) =	7.57	**ksi**	
fbult:	Bending Stress = Mcol x 6 / (t x (Ly-Ly2)^2) =	5.70	**ksi**	
	Combined normal stress = fault + fbult =	13.26	**ksi**	**OK <Phi* Fy**
	Implied orientation of stresses=	0.83436	**rad**	
	Capacity increase for orientation=	1.32		

COLUMN STRONG WAY

Design welds between the gusset and the column flange.

Strong-Way Weld Stresses

Average ultimate stress =	5.97	**ksi/in.**	
Peak ultimate Stress =	6.71	**ksi/in.**	
peak stress/average stress =	1.13	**< 1.4**	If this ratio is less than 1.4, size the weld for 1.4 times the average ultimate stress.

Column Web Stresses

Column web yielding stress =	11.37	**ksi**	**OK < Fy**

Weld Design

Electrode =	***E70***	
Electrode correction (LRFD 2nd ed. p8-158) =	1	
Required fillet weld size =	5.00	**16ths in.**

COLUMN WEAK WAY **Not Applicable**
Design welds between the gusset and the column web, and plates between column flanges.
Weak-Way Weld Stresses

fvweb:	Unit shear weld to web = Vcol/(2 x (Ly-Ly2)) =	N.A.	**kips/in.**	
Fvww:	Vert. shear cap. of col. web = 0.55 x Fy =	27.50	**ksi**	
	Column web stress =	0.00	**ksi**	**OK < Fvww**

Weld Design Gusset-to-Column

Electrode =	*E70*	
Electrode correction (LRFD 2nd ed. p8-158) =	1	
Required fillet weld size =	N.A.	**16ths in.**

Plate Design
Design plate to be welded to the gusset plate and the column flanges

PLfy:	Yield strength for the plate =	***36.00***	**ksi**	
PLt:	Plate thickness =	***0.50***	**in.**	
PLL:	Length of plate =	N.A.	**in.**	
PLf:	Force in the plate =	N.A.	**kips**	
Fvpl:	Ult. shear cap. of pl. = 0.55 x PLfy =	19.80	**ksi**	
PLfv:	Shear stress in plate = (PLf / (2 x PLL x PLt)) =	N.A.	**ksi**	**N.A.**
PLfb:	Bending stress in plate =	N.A.	**ksi**	**N.A.**

Weld Design Plate-to-Column/Gusset

Electrode =	*E70*	
Electrode correction (LRFD 2nd ed. p8-158) =	1	
Req'd fillet weld size of plate-to-gusset =	N.A.	**16ths in.**
Req'd fillet weld of plate-to-column flange =	N.A.	**16ths in.**

Summary

USSET PL 'A'						WELDS (16th's)		
Thk (in)	Lcol (in)	Lbm (in)	Lbr (in)	Ld (in)	Lw (in)	A	B	C
0.75	22	21.07	12	42	9	5.00	6.62	6.44

PLATES Ref.	Thk (in)	Length (in)	idth (in)	Weld (16th)	Gap (in)
F	0.5	N.A.	12.6	N.A.	

An accepted design methodology for the gusset plate which allows member end rotation was researched by Goel and has been adopted by the AISC *Seismic Provisions*. The provisions require that the brace maintain a minimum distance of 2 times the thickness of the plate from the anticipated line about which the plate will yield flexurally as the brace buckles. This line is assumed to occur between points of restraint such as the end of the gusset-to-beam connection and gusset-to-column connection. Stiffener plates may also be used to support the plate. The design should also maintain this line perpendicular to the axis of the brace to ensure the brace will buckle perpendicular to the plane of the frame.

This example allows the buckling to occur in the out-of-plane direction while it is assumed that in-plane buckling is restrained and will not control the design. If rectangular sections with largely differing properties were chosen, the capacity in each direction would need to be investigated to determine which controls the design. An effective width of plate can be calculated using Whitmore's method presented in AISC and is checked for tension and compression. The tension capacity of the gusset is conservatively estimated at A_sF_y while the brace ultimate capacity is utilized. The gusset plate is checked for compression strength in an area where it is restrained by the beam, column, and/or stiffener plates on all sides but one: along the buckling line. The true effective buckling length is complicated at best, but conservatively may be estimated at 0.8. Alternately, 1.0 between hinge locations may also be used.

Gusset-Plate–to–Beam-and-Column Connection. The forces imparted from the gusset plate to the face of the beam and column are obtained from the analysis using the uniform force method. Unless specifically optimized otherwise, each connection will see axial, shear, and bending forces. The plate, as well as the welds, are designed to remain elastic under these forces. The capacity of the weld may be checked in a number of ways. It is conservative to calculate an effective eccentricity of the shear force to the weld and add it vectorially to the axial force resulting in an effective force with an eccentricity and angle to the weld. The AISC charts for eccentrically loaded weld groups may then be used to determine the weld capacity.

Beam-to-Column Connection. Connection of the beam to the column is designed to transfer the resulting axial, shear, and bending demands on the beam. Due to the connections' highly restrained configuration from the gusset plate(s), this connection must be very stiff to adequately resist the forces. Moment-frame type connections consisting of groove-welded flanges and either welded or bolted webs using slip-critical bolts are typically used. The web and flange connections are sized to develop their share of the forces at the joint. It is typically sufficient to use full-penetration-welded flanges and webs.

The last page of Example 5.2 summarizes the design and Fig. 5.19 shows the final detail of the connection.

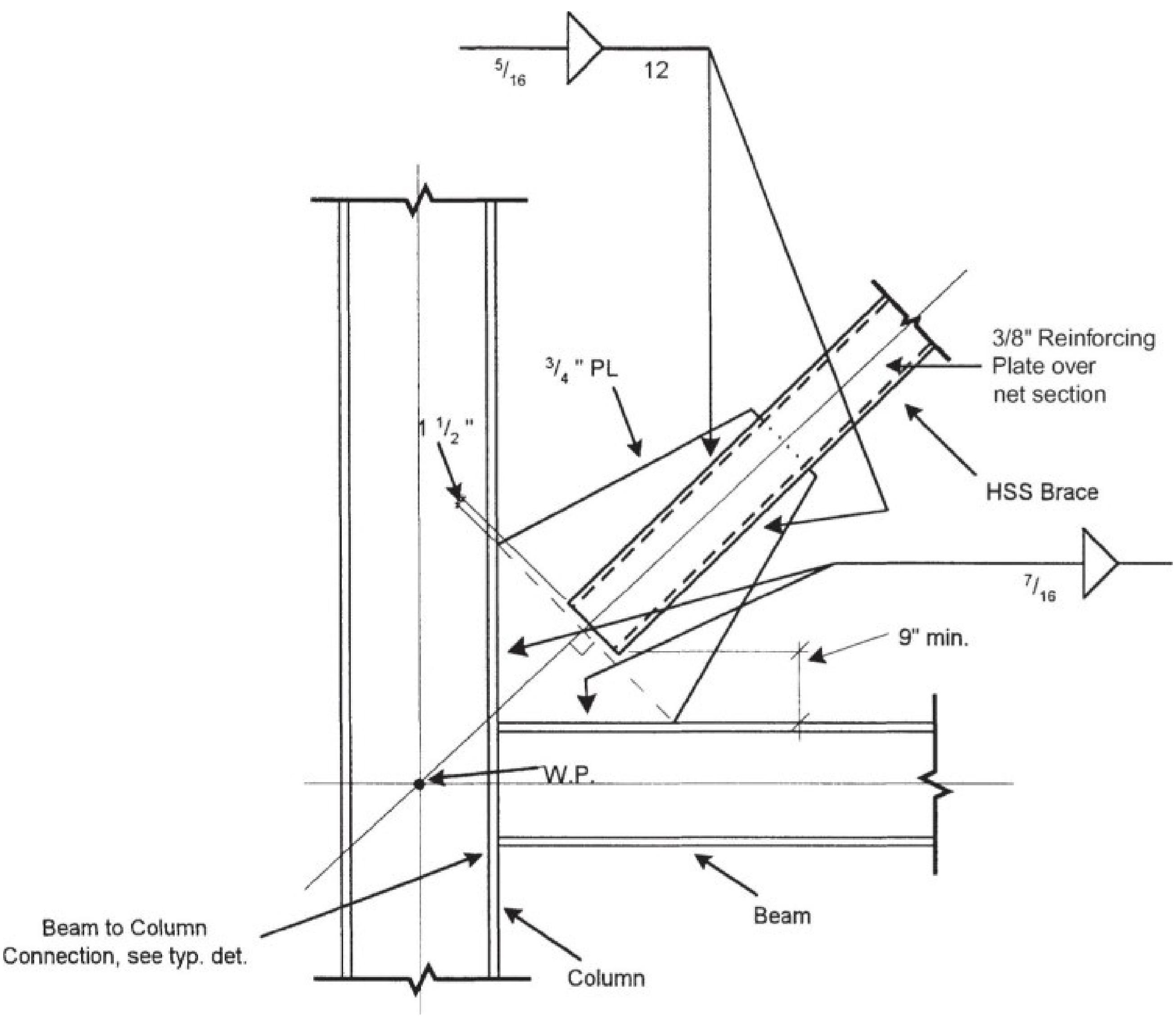

FIGURE 5.19 Brace connection.

5.5 *ECCENTRICALLY BRACED FRAMES*

Eccentrically braced frames (EBFs) are braced frame systems which utilize a link beam created by the eccentric connection of the diagonal brace or braces. The system provides energy dissipation through inelastic deformation of the link. The link either yields in shear (short links) or in bending (long links), while the beams, columns, and braces in the system remain elastic.

The design of the connections in an EBF is very similar to that of the SCBF. The methodology used in Example 5.2 could be used to design a brace-to-beam or brace-to-column connection in an EBF with the following exceptions. First, where the SCBF was designed based on the capacity of the brace, in an EBF the expected capacity of the link is used to size the brace and beam connections. Second, since the brace is not intended to yield, providing the 2*t* buckling line is not necessary. Finally, the eccentricity of the brace to the beam creates large bending demands in the link which are resisted by the beam outside of the link and by the brace member. Although braces have traditionally been considered pinned, in an EBF a brace can attract significant bending due to their fixed connections which must be accounted for in the design of the brace and its connection to the beam and/or column. The additional bending on the gusset plate may be superimposed with the force distribution obtained from the uniform force method.

5.6 BUCKLING RESTRAINED BRACED FRAMES

Like SCBF, connections between members of BRBFs are intended to be able to force inelastic behavior to occur in the braces. AISC 341 requires that the connections have a required strength of 1.1 times the expected strength of the brace. For BRBFs the expected strength of the brace is likely to be controlled by the compression yielding, which is generally on the order of 10% higher than the tension capacity. Because BRBFs are not subject to brace buckling, gusset plate designs that rely on folding on the yield line (see Example 5.2) are not required. Force distribution calculations using the uniform force method would still apply, however. NIST Technical Brief No. 11, “Seismic Design of Steel Buckling Restrained Braced Frames: A Guide for Practicing Engineers” (NIST, 2015), presents an excellent summary of the concepts to overall design of this system.

5.7 SPECIAL PLATE SHEAR WALLS

For SPSW systems, the concept for connection design is identical to the other ductile steel systems. The connections between the plates and the boundary elements are designed to develop the expected capacity of the plate, recognizing the characteristic angle of plate yielding. In addition, the connections between the HBE and VBE are required to be fully restrained moment resisting connections designed to meet the requirements for OMFs at the expected yield capacity of the HBE members. Further, the shear capacity of this connection must be able to transfer the vertical shear induced by the yielded wall plates in addition to the shear that is generated by fully yielding of the HBE as a moment frame member. The shear induced by the yielded wall plates can become especially significant at the top and bottom stories of the walls, and where there is a transition in thickness of the wall plate. At other levels, this shear is basically cancelled out by the wall plates above and below the HBE.

5.8 OTHER CONNECTIONS IN SEISMIC FRAMES

In addition to the connections between primary members of the seismic load-resisting system that have been discussed in this chapter, there are a number of other connections that are critical to the seismic performance of steel systems. The first is the splice of seismic frame columns. The AISC *Seismic Provisions* have a series of requirements to help ensure that these splices are able to resist all the forces necessary to develop the intended inelastic performance of the system without fracture. The first of these requirements is that the splices be designed to resist the amplified seismic loads for any tensile stresses. In addition, since partial joint penetration welds are more susceptible to fracture, they are required to be designed for twice the amplified seismic load. And, in all cases, the connection must be able to develop at least 50% of the expected tensile capacity of the smaller column. In addition to these requirements that apply to all steel systems, there are additional requirements for the highly ductile systems. For example, the requirements for SMF systems effectively require column splices that develop the expected tensile capacity of the smaller column, with a shear capacity sufficient to form a plastic hinge in the column. For the braced frame systems (other than OCBF), the column splices are generally required to resist a flexural strength of at least 50% of the smaller column, with a shear capacity to form a plastic hinge in the column.

The connection of the column to the base plate is a special case of the typical column splice. It is critical that the design engineer provide a base connection that is not subject to fracture, since it is well understood that these connections are often subject to inelastic behavior in order a full plastic mechanism to be developed in a well-proportioned steel frame structure. However, the design requirements in previous editions of building codes have not adequately addressed these connections, and the transition between steel and the concrete foundation elements, and the transition between concrete foundation and the supporting soils. AISC 341 now has a section that defines the requirements for the design of the column base and the anchorage of the base into the concrete foundation. Essentially, the requirements cause the base connection to be designed for the same force that the column has been designed for, in flexure, shear, and axial force.

In addition to the requirements for splices for columns that are part of the lateral resisting system, the AISC *Seismic Provisions* require a check of the columns that are not deemed to be part of the SLRS. This check verifies a minimum shear capacity needed to generate a plastic hinge in the column over a single story height. This requirement is the result of evaluating the beneficial effect that these nonseismic frame columns can have in the overall inelastic performance of steel framing systems. The additional capacity of these columns can help to avoid the formation of story mechanisms that can be very detrimental to seismic performance.

The other connections of note for good seismic performance are those provided for out-ofplane stability. Providing out-of-plane stability is critical to ensuring the expected performance of seismic systems that are anticipated to undergo substantial story drifts and large inelastic demands in the event of a design level earthquake. The AISC *Seismic*

Provisions include a series of requirements for the various systems. These requirements include both strength and stiffness checks for the bracing elements and connections, based on the provisions of the main AISC Specification (AISC 360). The design force for these connections varies depending on whether or not the stability bracing is located adjacent to a plastic hinge. Significantly higher bracing forces develop at these hinge locations, on the order of 6% of the beam flange capacity. Proper seismic performance of the entire frame necessitates that this stability bracing be provided.

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(*Courtesy of The Steel Institute of New York.*)

CHAPTER 6
STRUCTURAL STEEL DETAILS

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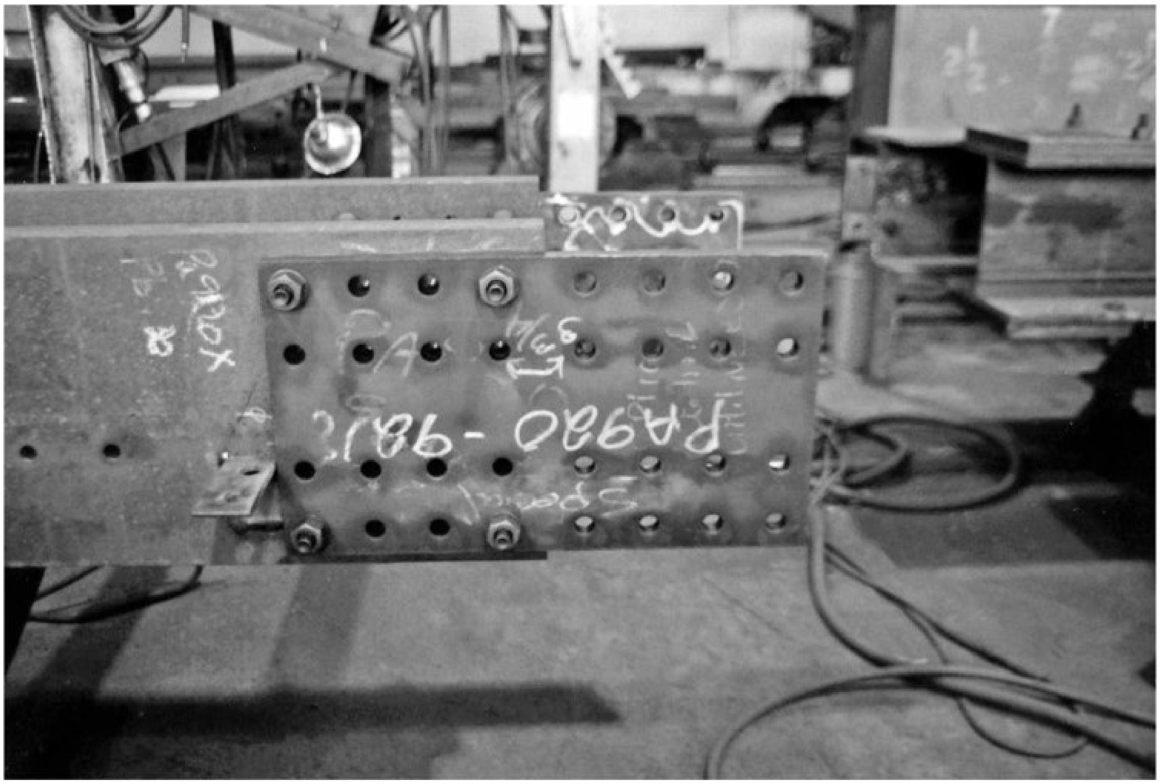
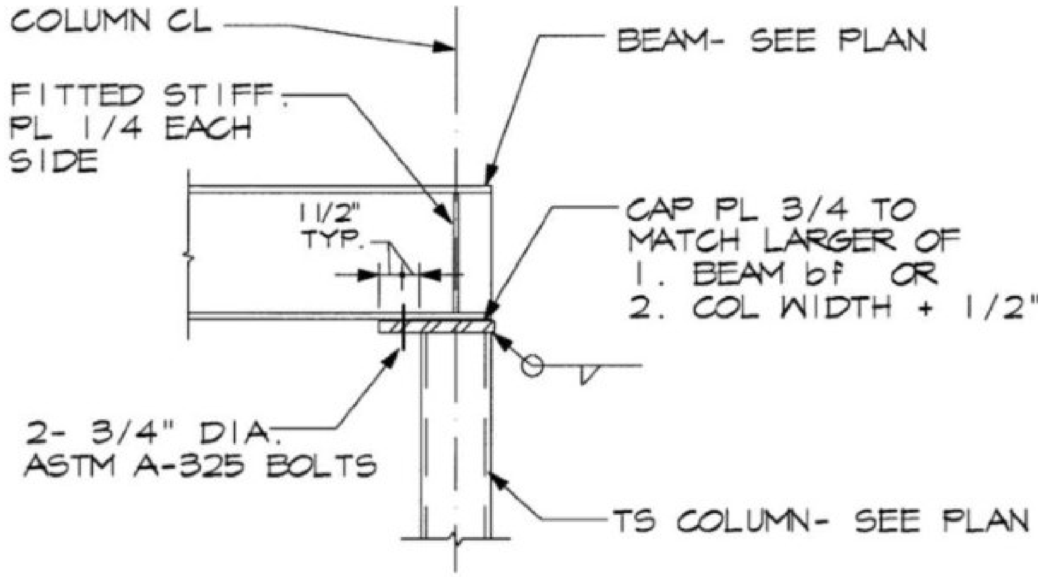
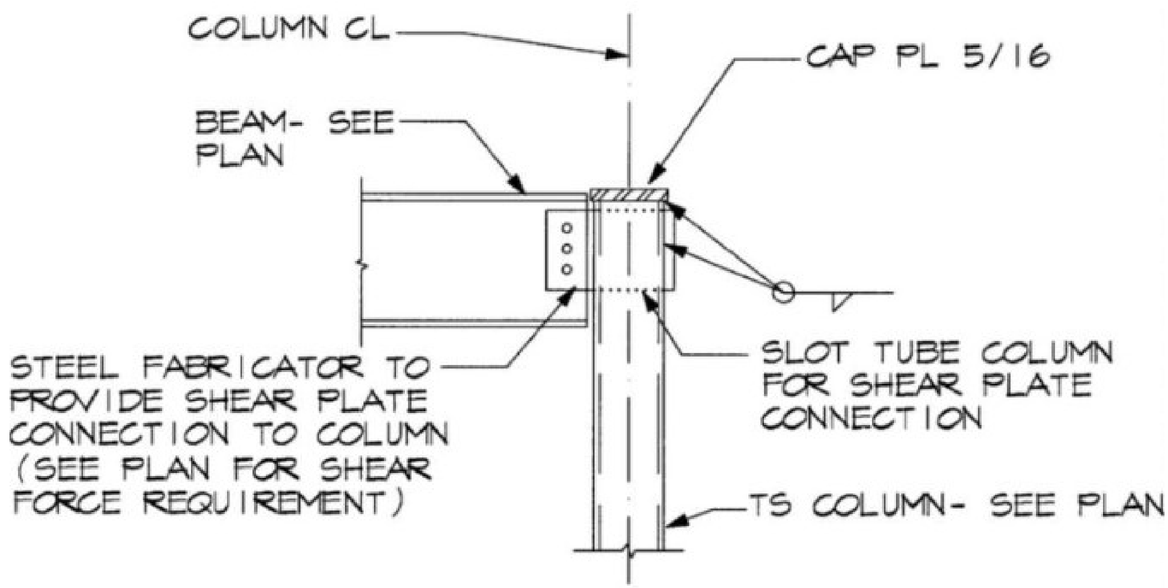


(Courtesy of The Steel Institute of New York.)



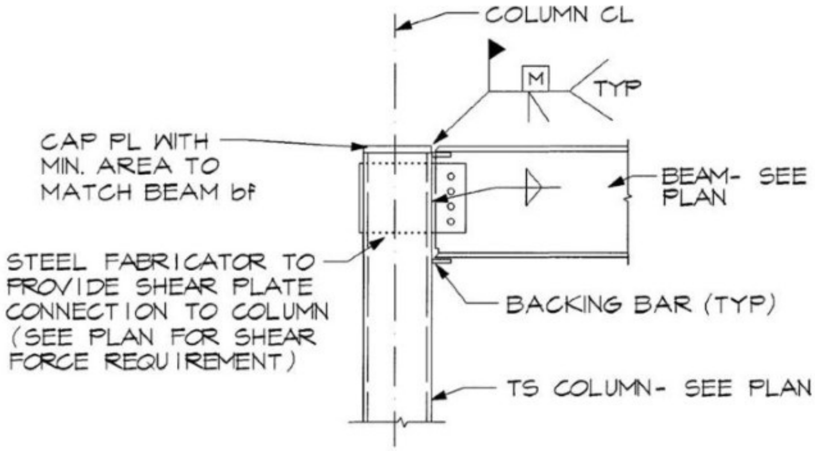
BEAM FRAMES OVER TS COLUMN



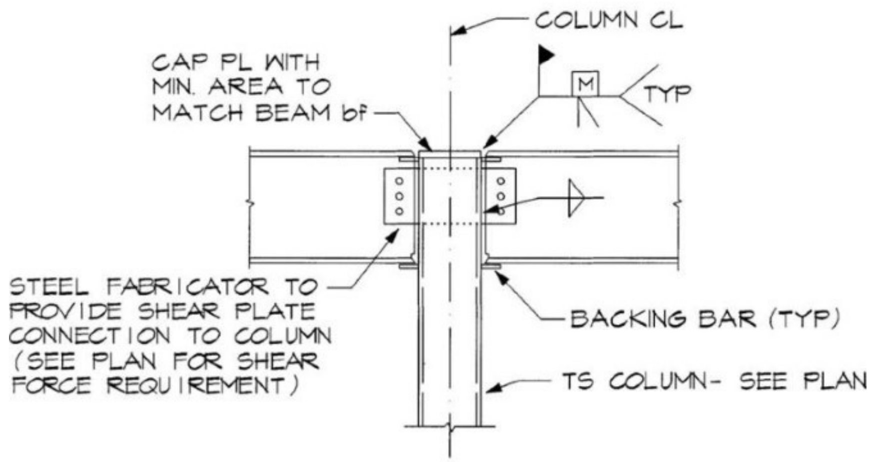
BEAM FRAMES INTO TS COLUMN

BEAM SHEAR CONNECTION DETAILS

NOT TO SCALE (DETAIL T5–CSH1)



BEAM FRAMES ONE SIDE OF TS COLUMN



BEAM FRAMES BOTH SIDES OF TS COLUMN

BEAM MOMENT CONNECTION DETAILS

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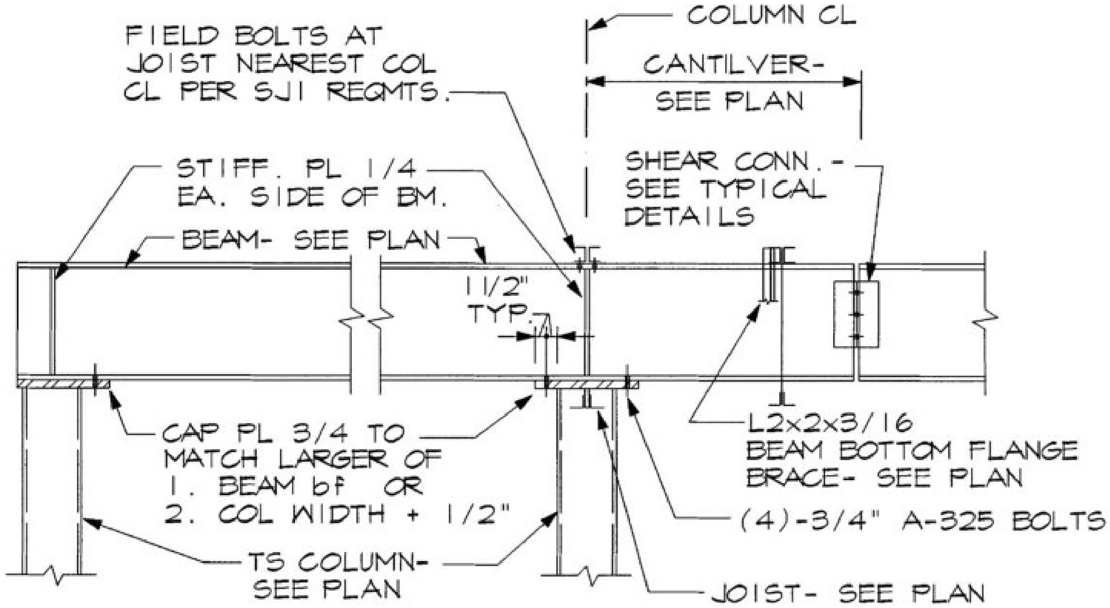


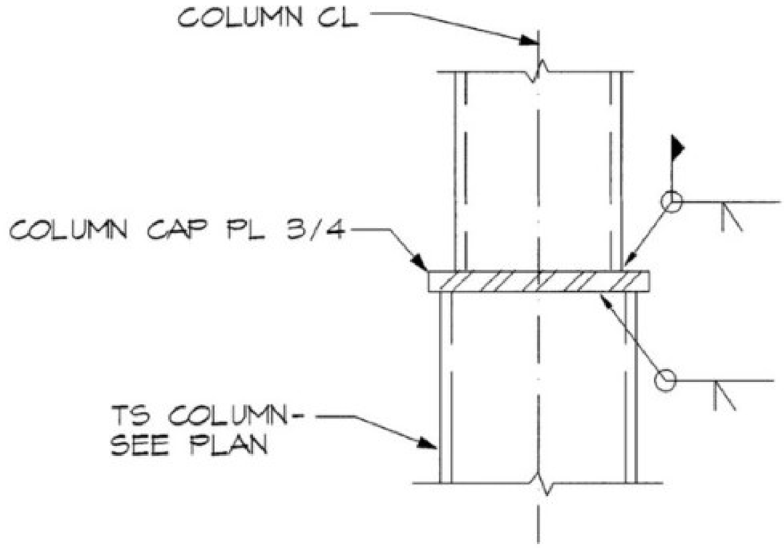
END TS COLUMN

INTERIOR TS COLUMN

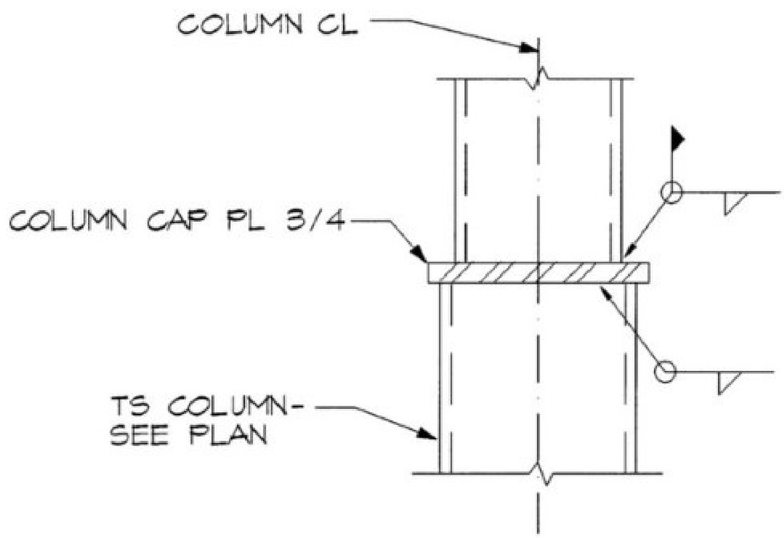
TYPICAL CANTILEVER ROOF BEAM DETAILS

NOT TO SCALE

(DETAIL T5-CBM1)



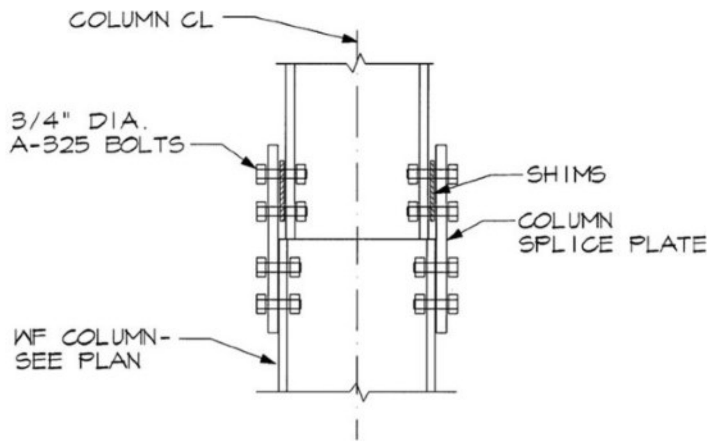
TS COLUMN SPLICE, FILLET WELDED



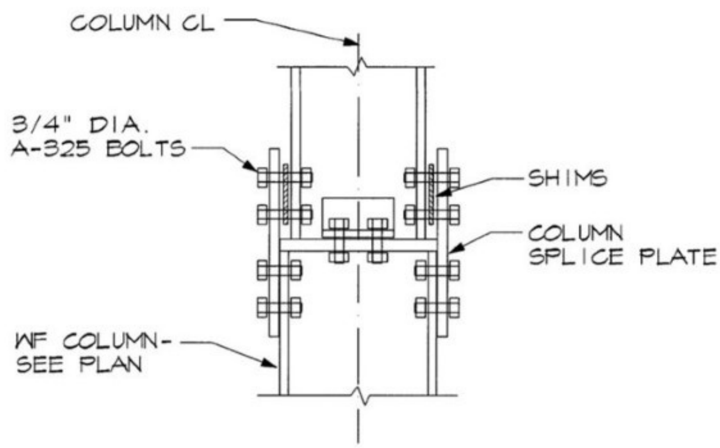
TS COLUMN SPLICE, PENETRATION WELDED

TYP. TS COLUMN SPLICE DETAILS

NOT TO SCALE (DETAIL T5–COL1)



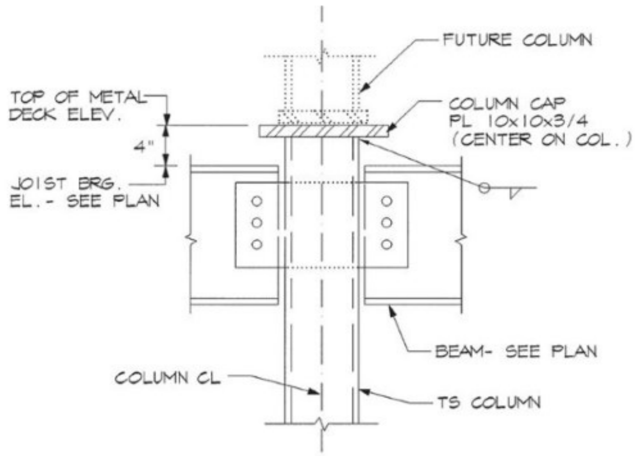
WF COLUMN SPLICE, BOLTED



WF COLUMN SPLICE, BUTT PLATE

TYP. WF COLUMN SPLICE DETAILS

NOT TO SCALE (DETAIL T5–COL2)

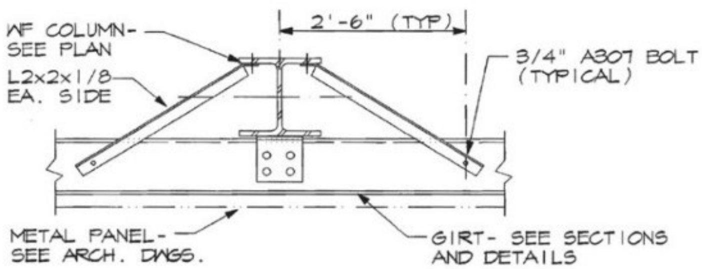


TYP. FUTURE COLUMN CONNECTION

NOT TO SCALE (DETAIL T5-COL3)

NOTES:

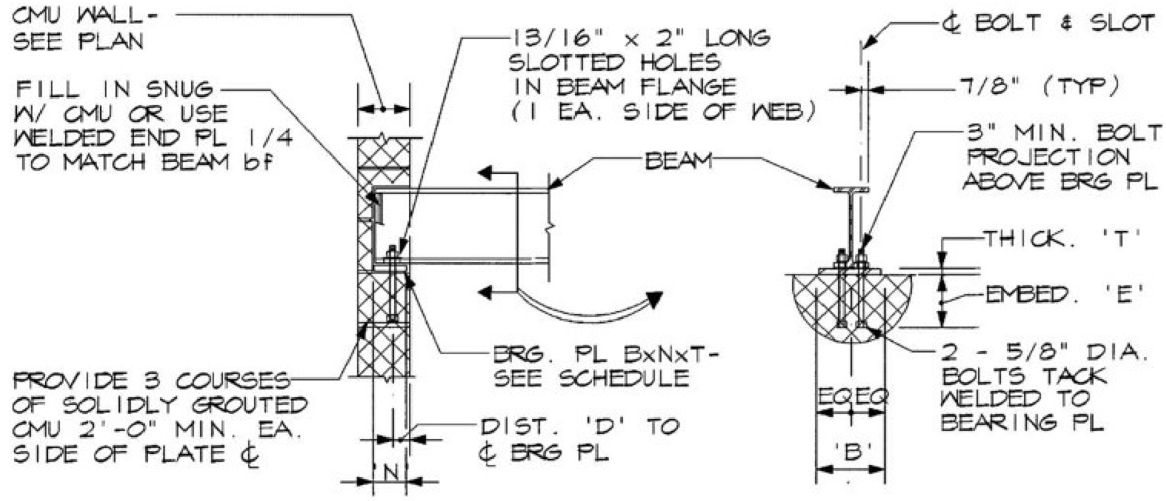
1. SEE WALL ELEVATIONS DETAILS FOR LOCATIONS.
2. DETAIL AT PERIMETER COLUMNS IS SHOWN. DETAIL AT CORNER COLUMNS IS SIMILAR.



TYPICAL COLUMN BRACE DETAIL

NOT TO SCALE (DETAIL T5-BRCE1)

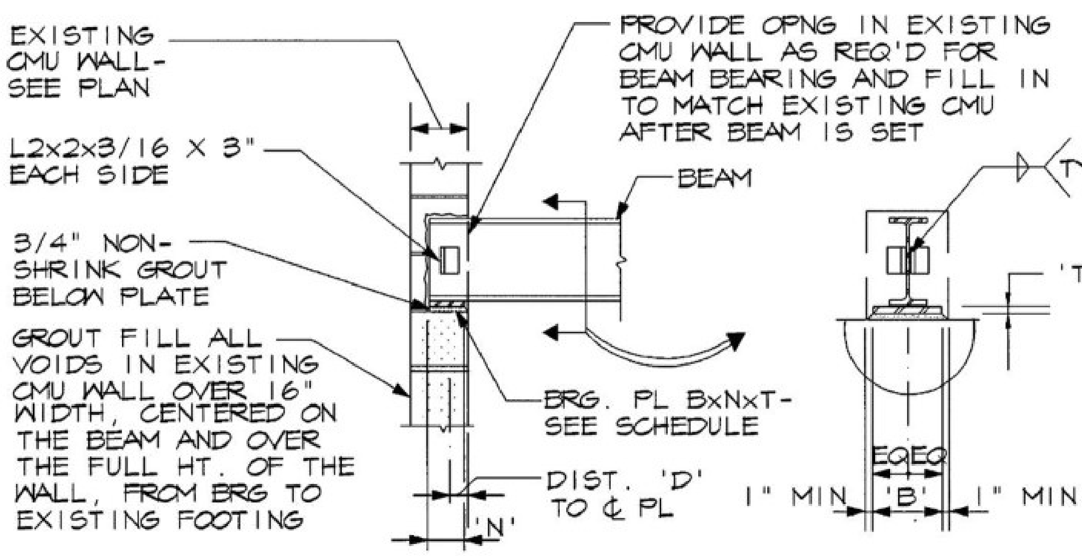
NOTE: DETAIL IS SHOWN FOR BEAMS PERPENDICULAR TO WALL, PROVIDE SIMILAR DETAIL FOR BEAMS PARALLEL TO WALL.



"FIXED" CONNECTION

TYPICAL BEAM BEARING ON MASONRY DETAILS

NOT TO SCALE (DETAIL T5-CMU1)

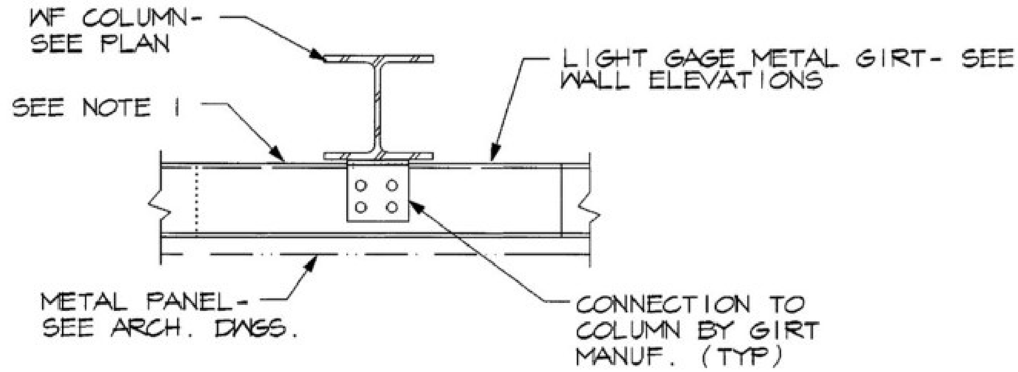


TYPICAL BEAM BEARING ON EXISTING MASONRY DETAILS

NOT TO SCALE (DETAIL T5-CMU3)

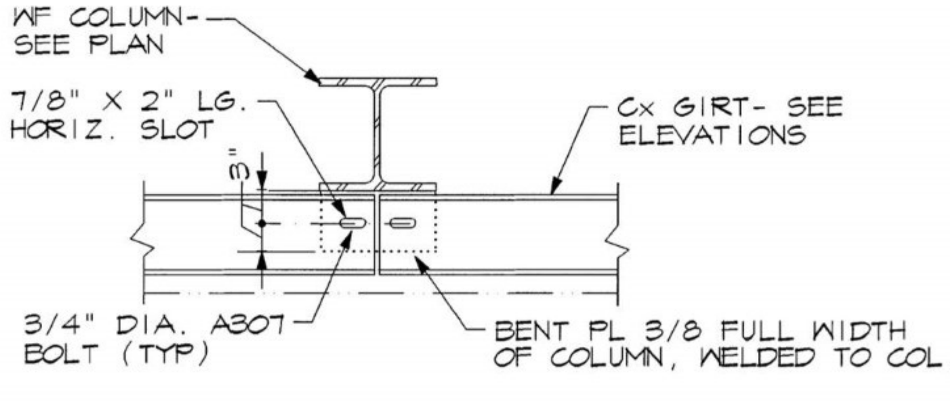
NOTES:

1. LAP GIRTS AT COLUMN AS REQUIRED BY THE MANUFACTURER.
2. PROVIDE GIRT WEB STIFFENERS AS REQUIRED BY THE MANUF.

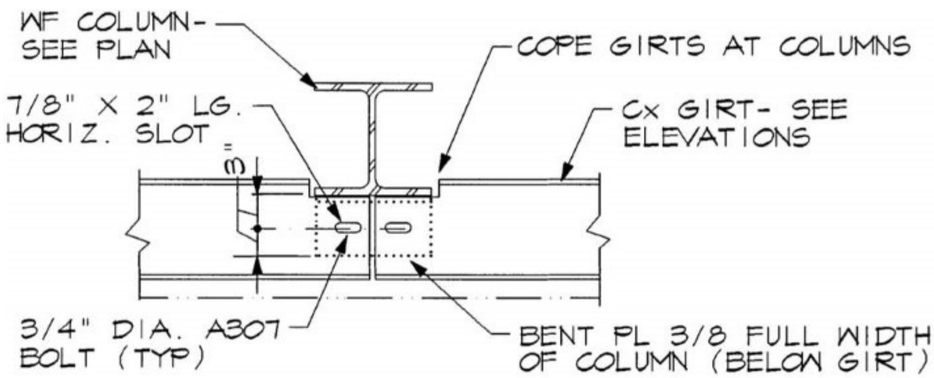


TYPICAL LIGHT–GAGE METAL GIRT CONNECTION DETAILS

NOT TO SCALE (DETAIL T5–GIRT1)



AT TYP. GIRTS

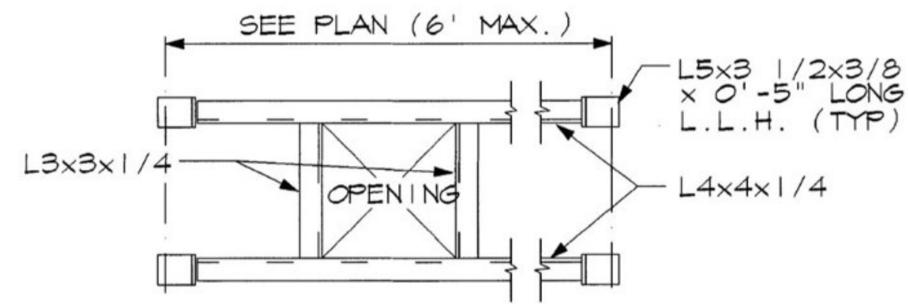


AT COPED GIRTS

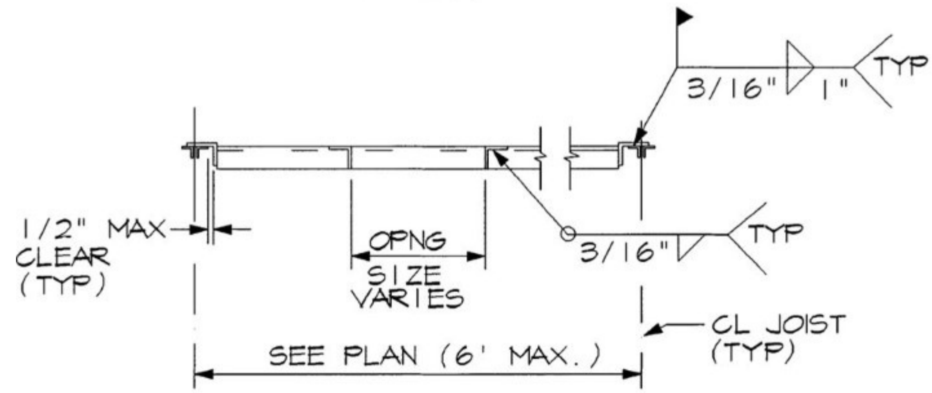
TYP. GIRT CONNECTION DETAILS

NOT TO SCALE (DETAIL T5–GIRT4)

NOTE: COORD. SIZE AND LOCATION OF ROOF OPENINGS WITH ACTUAL EQUIPMENT SELECTED.



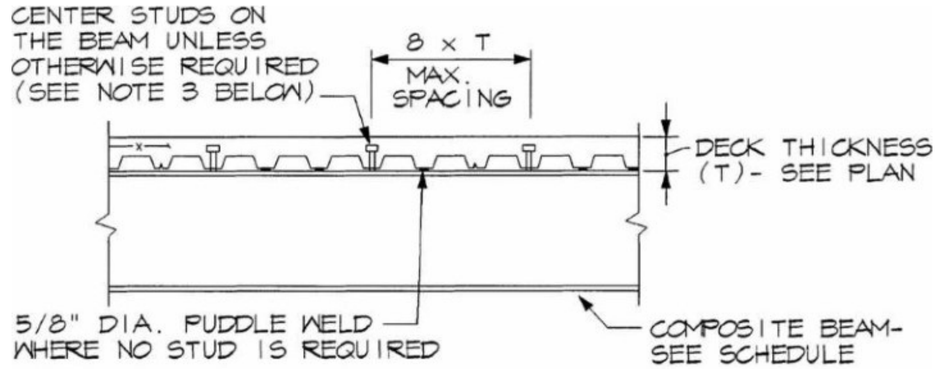
PLAN



SECTION

TYP. ROOF OPENING DETAIL

NOT TO SCALE (DETAIL T5-R01)



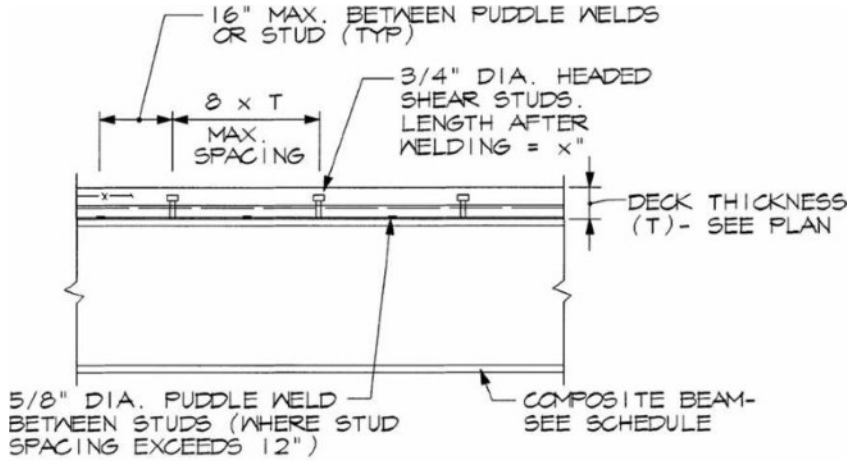
DECK IS PERPENDICULAR OR SKEWED TO BEAM

NOTES:

1. THE MIN. NUMBER OF STUDS FOR EACH BEAM IS SHOWN IN THE COMPOSITE BEAM SCHEDULE.
2. SPACE STUDS AS EVENLY AS POSSIBLE IN AVAILABLE DECK FLUTES. WHERE STUD SPACING EXCEEDS THE MAX. SPACING ALLOWED, PROVIDE ADDITIONAL STUDS TO SATSIFY THE SPACING REQUIREMENTS.
3. WHERE THE NUMBER OF STUDS EXCEEDS THE NUMBER OF FLUTES, PROVIDE TWO STUDS IN EVERY OTHER FLUTE, STARTING AT EACH END OF THE BEAM. THE TRANSVERSE SPACING BETWEEN TWO STUDS IN A SINGLE FLUTE SHALL BE 4 × STUD DIAMETERS (MIN.).
4. SEE THE COMPOSITE BEAM SCHEDULE FOR ADDITIONAL REQUIREMENTS. TURN THE NATURAL BEAM CAMBER UP.

TYP. COMPOSITE BEAM ELEVATION

NOT TO SCALE (DETAIL T5–COMP1)



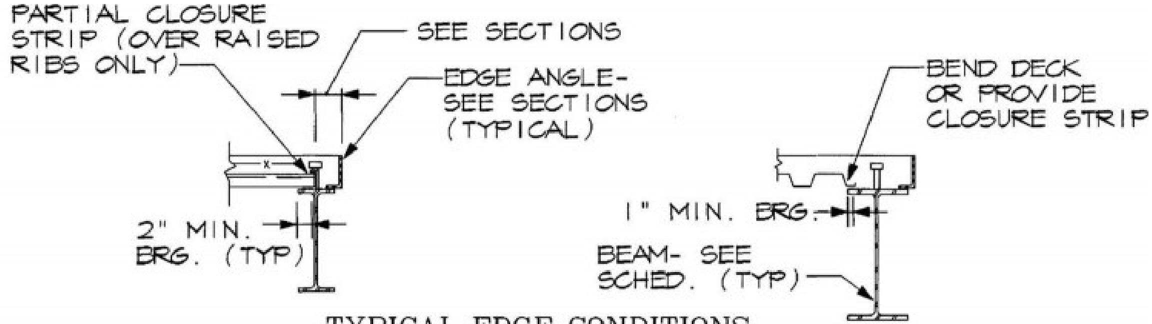
DECK IS PARALLEL TO BEAM

COMPOSITE BEAM SCHEDULE					
MARK	BEAM	MAX. END REACTION	MIDSPAN CAMBER	NUMBER OF STUDS	REMARKS
CB-1	W10x15	11 KIPS	—	12	INTERIOR
CB-2	W10x15	8 KIPS	—	8	INTERIOR
CB-3	W10x26	13 KIPS	—	12	INTERIOR
CB-4	W10x15	8 KIPS	—	3	CANTILEVER
CB-5	W10x26	13 KIPS	—	5	CANTILEVER
CB-6	W10x12	7 KIPS	—	3	CANTILEVER
CB-7	W18x76	86 k(@RT)	0.6"	22	GIRDER
CB-8	W16x40	36 k(@RT)	—	16	GIRDER

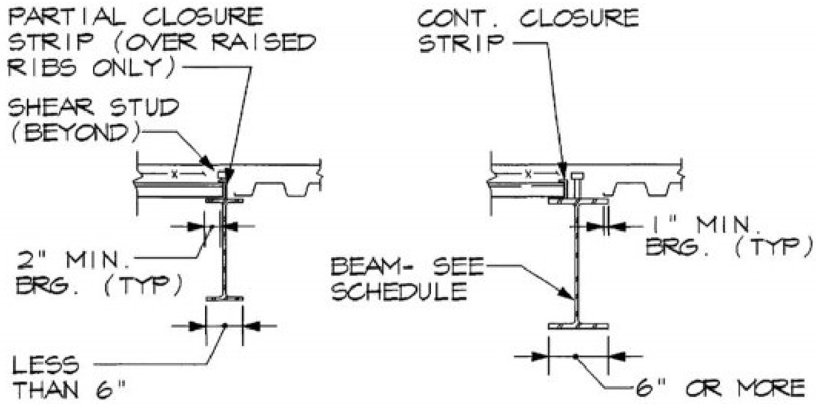
NOTE: NATURAL CAMBER OF BEAM SHALL BE TURNED UP.

TYP. COMPOSITE BEAM ELEVATION

NOT TO SCALE (DETAIL T5-COMP2)



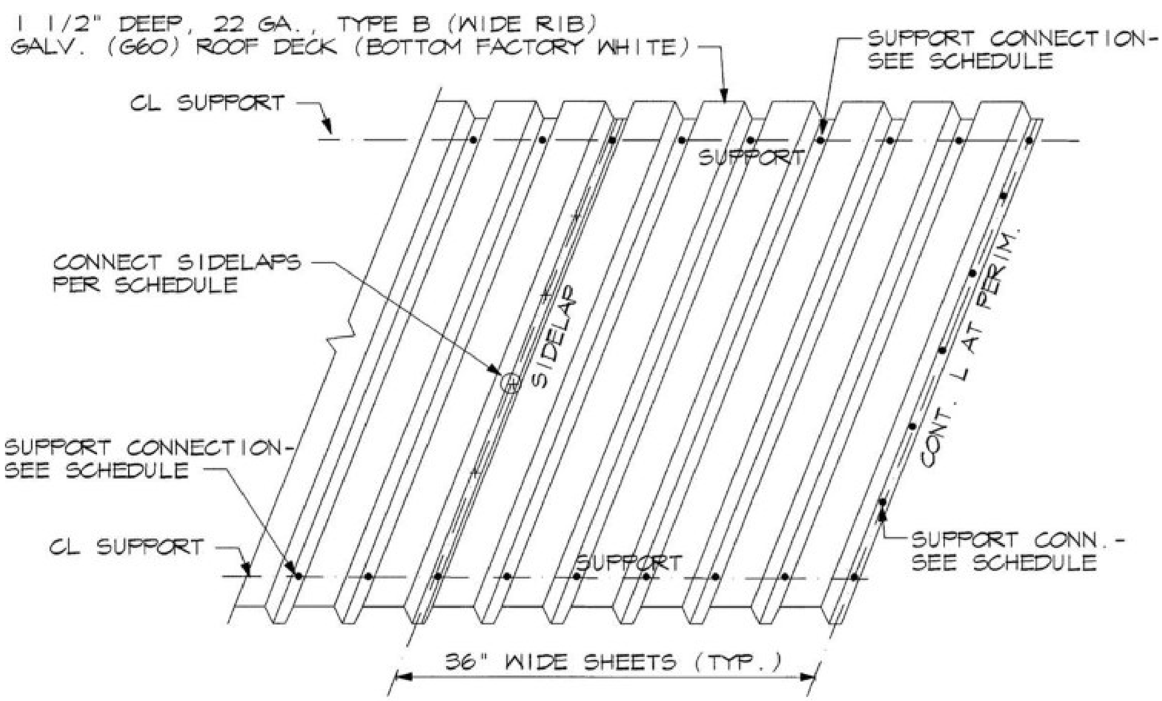
TYPICAL EDGE CONDITIONS



TYPICAL INTERIOR CONDITIONS

TYPICAL COMPOSITE DECK CLOSURE DETAILS

NOT TO SCALE (DETAIL T5-COMP3)

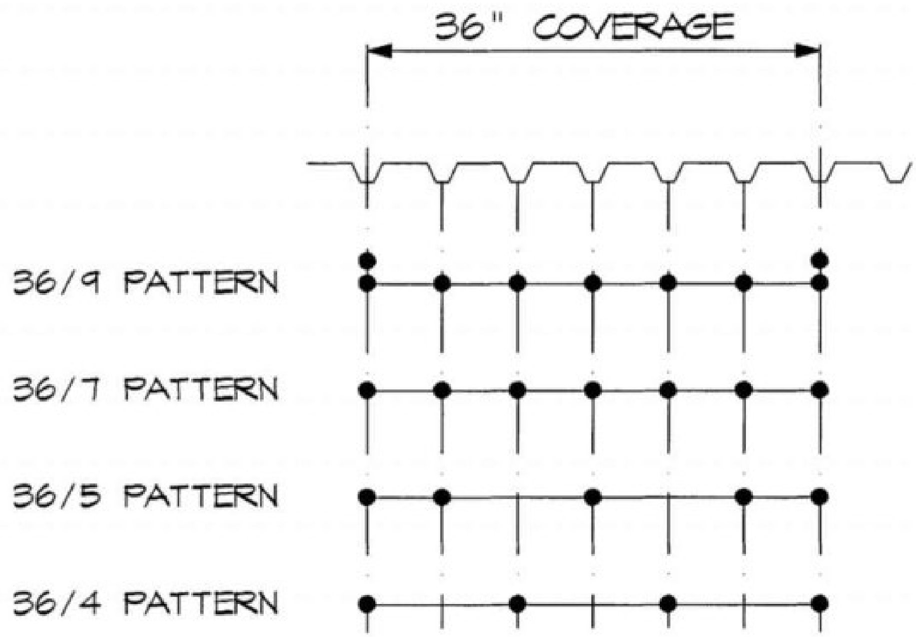


TYPICAL ROOF DECK ATTACHMENT DETAIL

NOT TO SCALE (DETAIL T5-RDIA2)

ROOF DECK CONNECTION SCHEDULE

ZONE	DECK CONNECTION
ZONE 1 (DIAPH. CAPACITY= 470#/LF)	WELDS IN 36/7 PATTERN (6)-#10 TEKS @ SIDELAPS
ZONE 2 (DIAPH. CAPACITY= 290#/LF)	WELDS IN 36/7 PATTERN (3)-#10 TEKS @ SIDELAPS
ZONE 3 (DIAPH. CAPACITY= 290#/LF)	WELDS IN 36/4 PATTERN (3)-#10 TEKS @ SIDELAPS

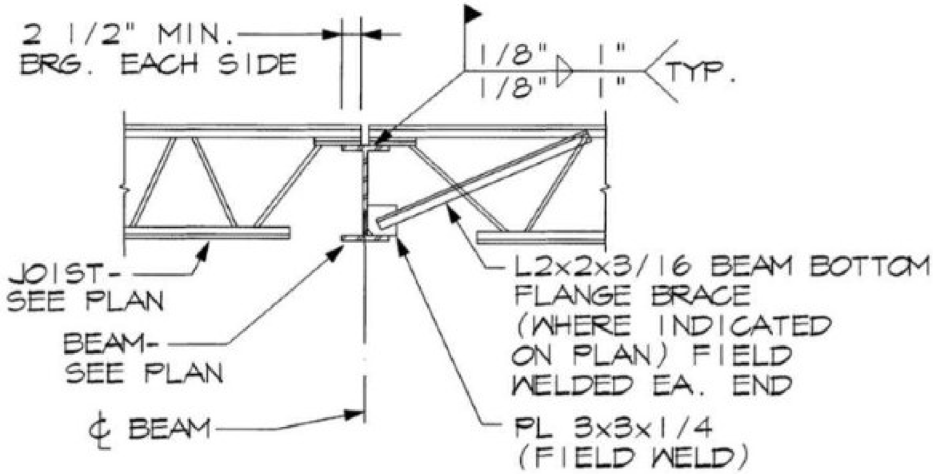


ROOF DECK ATTACHMENT PATTERNS

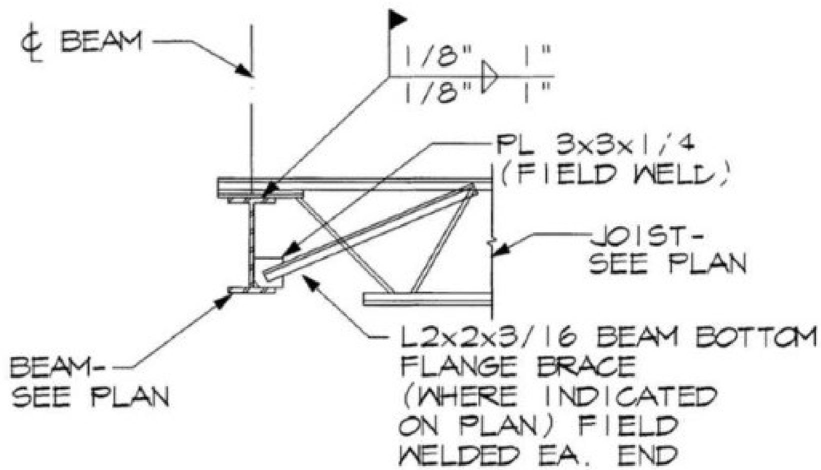
NOT TO SCALE (DETAIL T5-RDIA4)

NOTES:

1. AT JOIST NEAREST (OR ON) COLUMN CL CONNECT JOISTS WITH FIELD BOLTS PER S.J.I. REQUIREMENTS.
2. OFFSET JOISTS IF BEAM FLANGE IS LESS THAN 5" AND PROVIDE MINIMUM OF 2 1/2" JOIST BEARING.



AT INTERIOR BEAMS

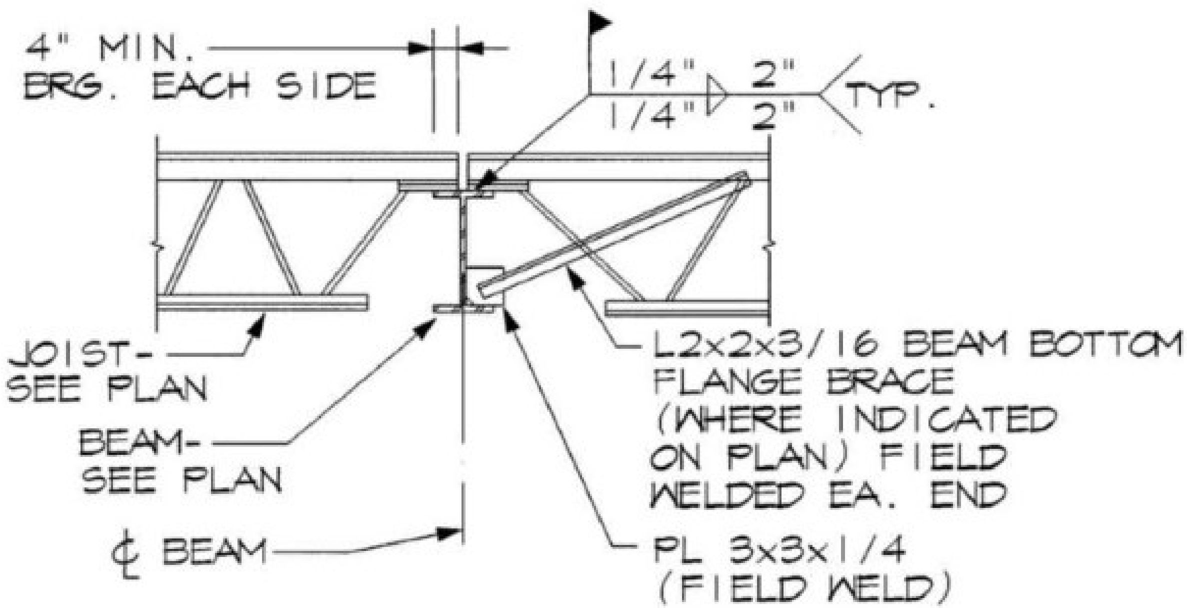


AT PERIMETER BEAMS

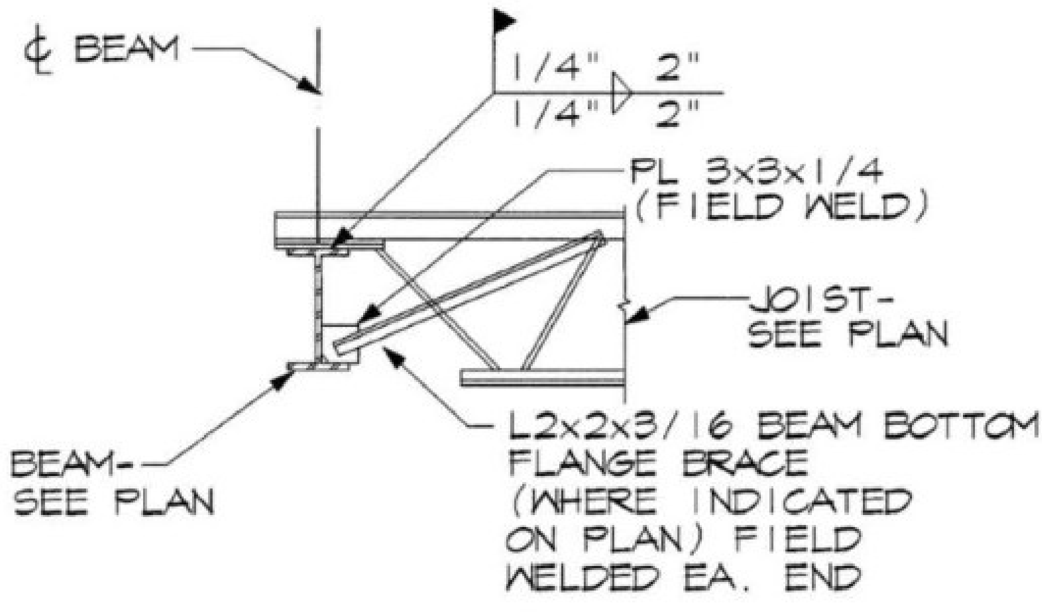
TYP. K-JOIST CONNECTION DETAILS

NOT TO SCALE (DETAIL T5-J1)

NOTES:
1. AT JOIST NEAREST (OR ON) COLUMN CL, CONNECT JOISTS WITH FIELD BOLTS PER S.J.I. REQUIREMENTS.
2. OFFSET JOISTS IF BEAM FLANGE IS LESS THAN 8" AND PROVIDE MINIMUM OF 4" JOIST BEARING.



AT INTERIOR BEAMS



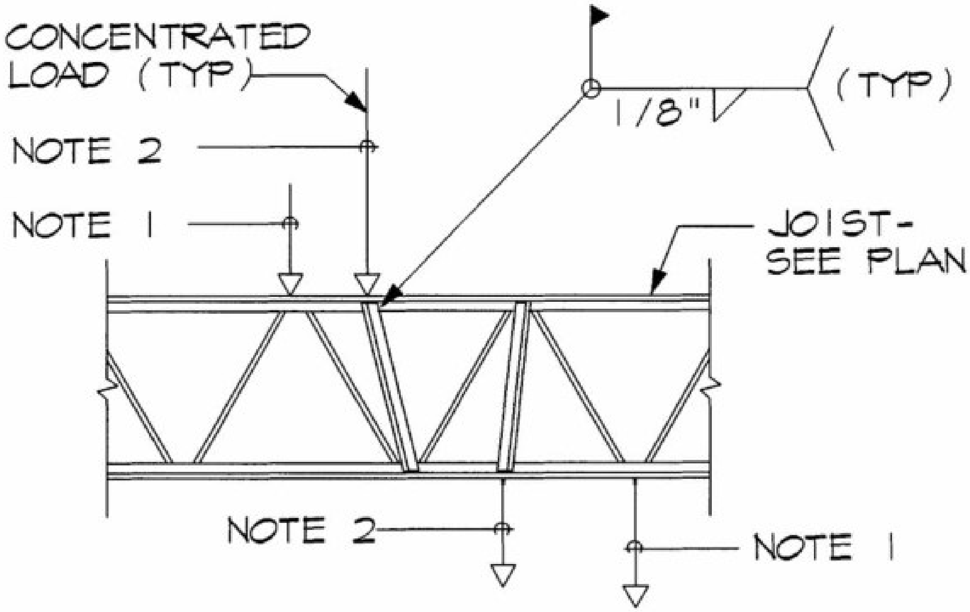
AT PERIMETER BEAMS

TYP. LH-JOIST CONNECTION DETAILS

NOT TO SCALE (DETAIL T5-J2)

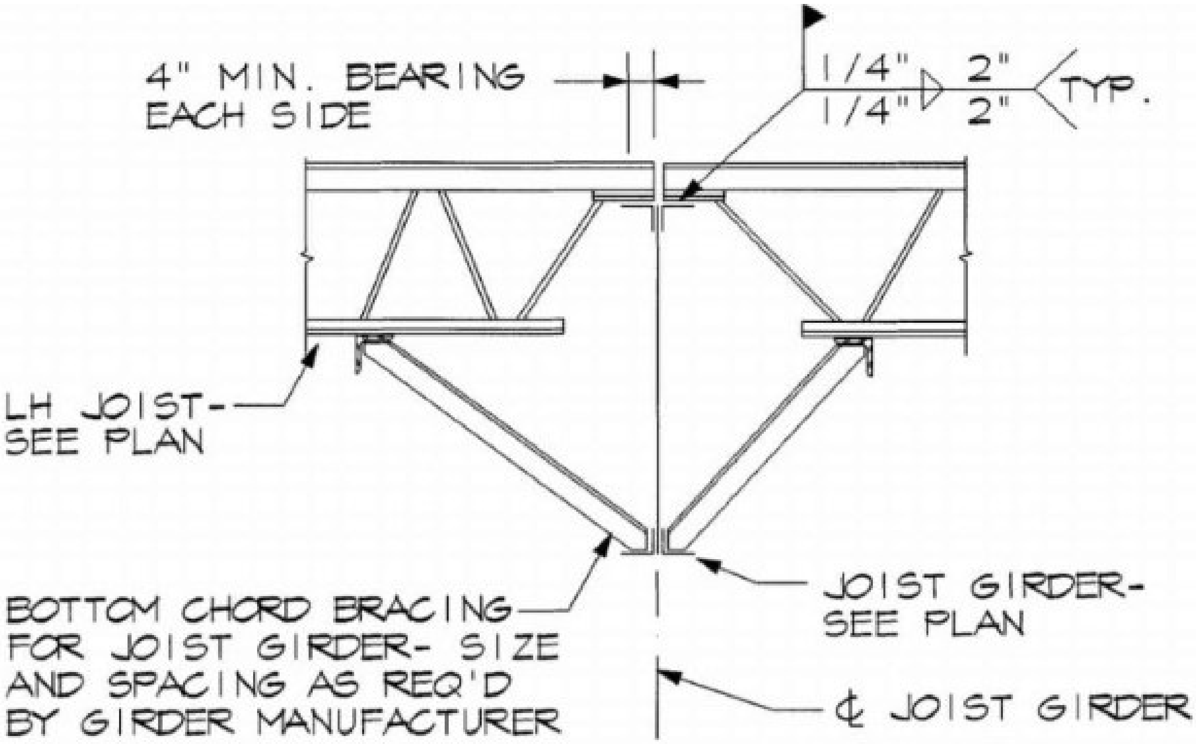
NOTES:

1. CONCENTRATED LOAD LOCATED AT JOIST PANEL POINT LOCATION - NO ADDITIONAL ANGLES REQUIRED.
2. CONCENTRATED LOAD (100 LBS. OR HEAVIER) NOT LOCATED AT JOIST PANEL POINT LOCATION - PROVIDE JL1x1x1/8 TO PANEL POINT AS SHOWN.

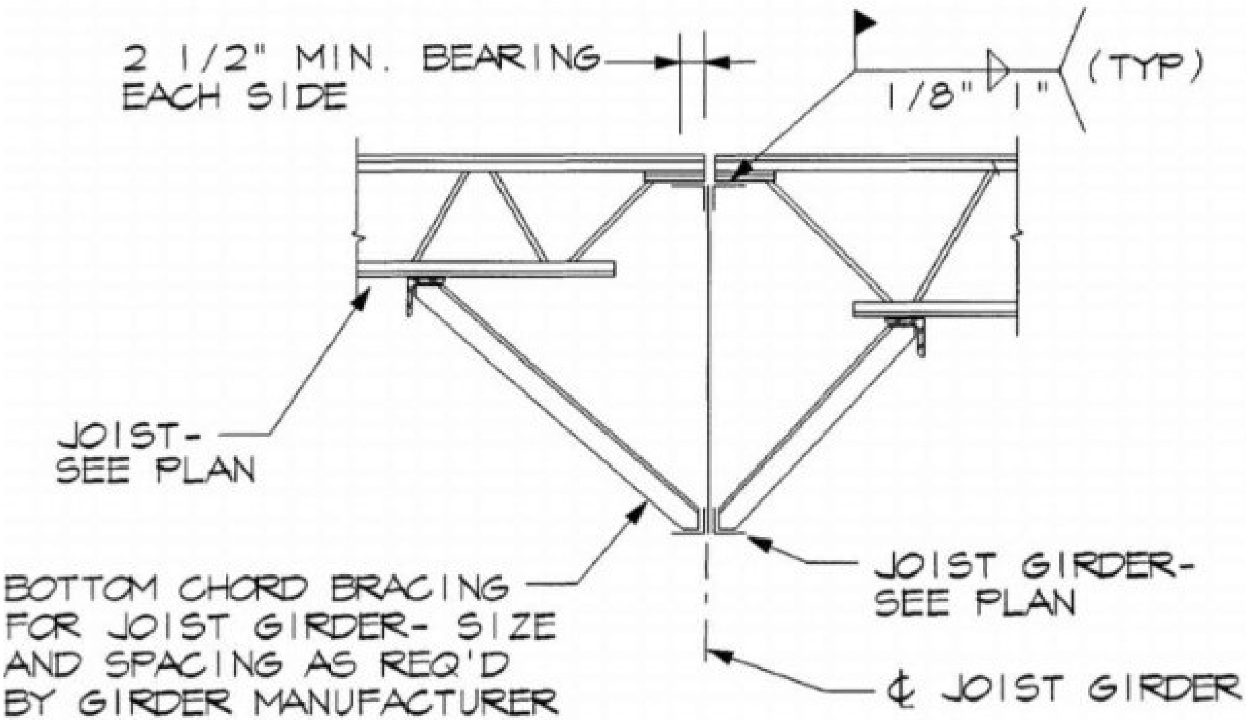


TYP. CONCENTRATED LOAD DETAIL

NOT TO SCALE (DETAIL T5–J5)



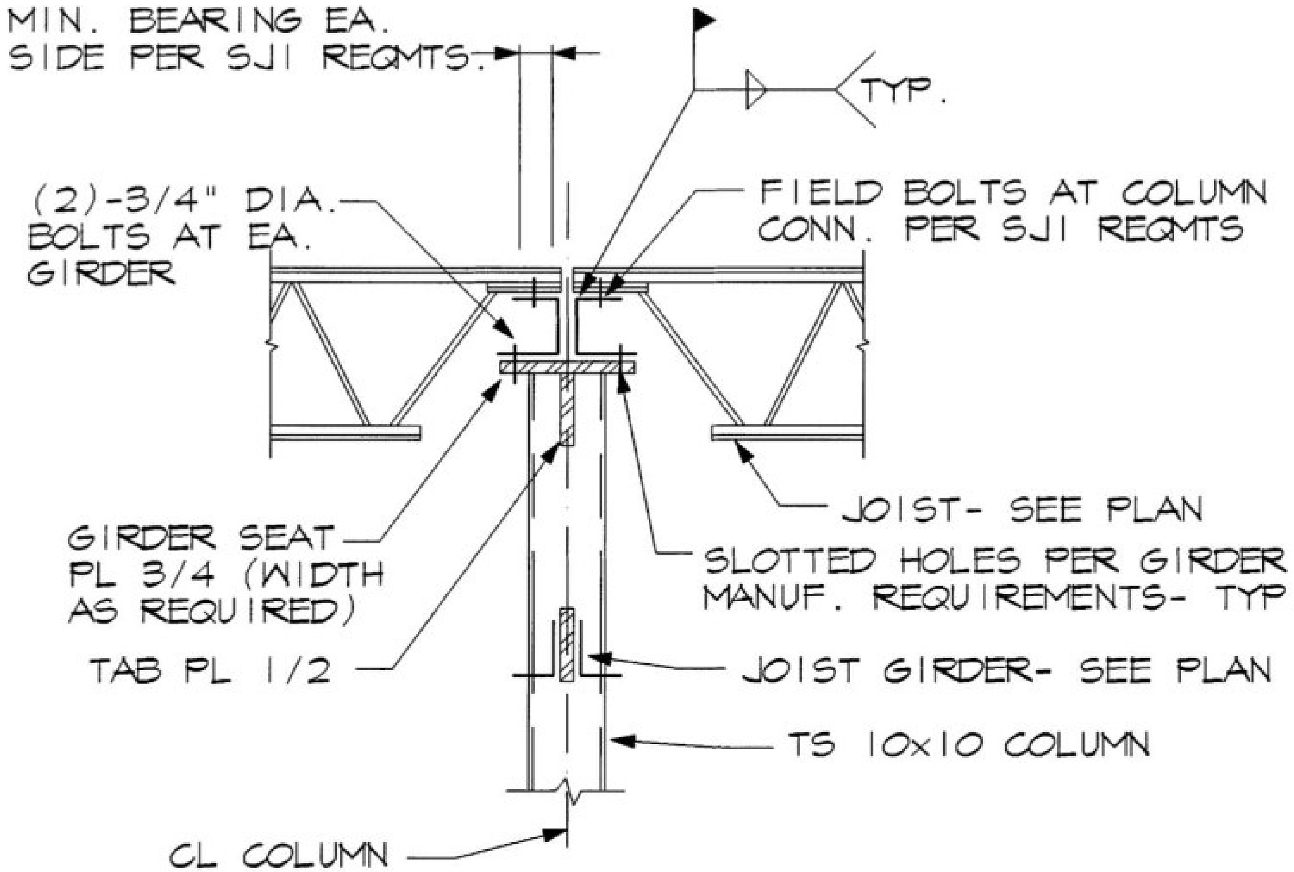
LH–SERIES JOISTS



K– SERIES JOISTS

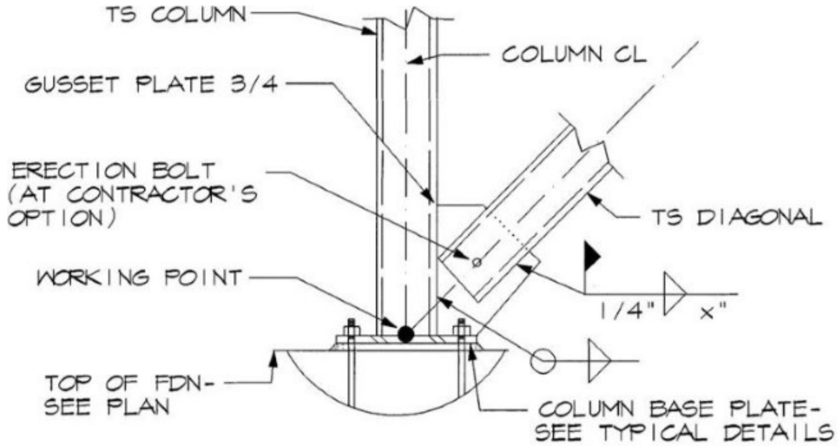
TYP. JOIST TO GIRDER CONNECTION

NOT TO SCALE (DETAIL T5–JG1)

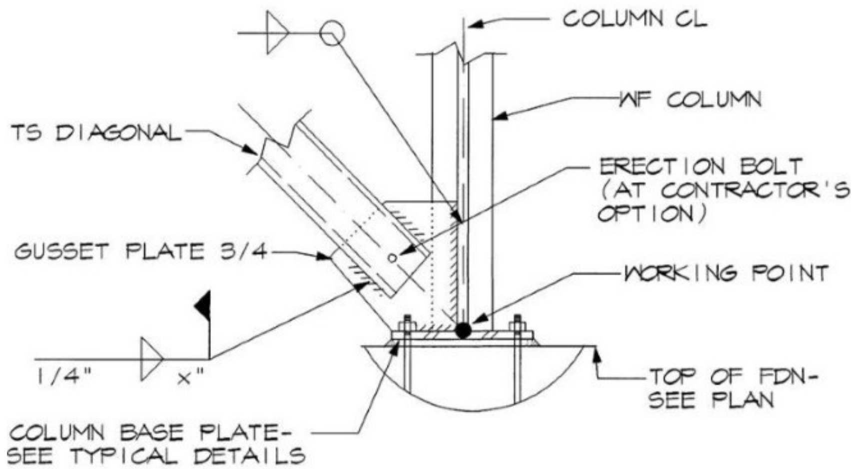


TYP. DETAIL AT INTERIOR COLUMN CL

NOT TO SCALE (DETAIL T5–JG2)



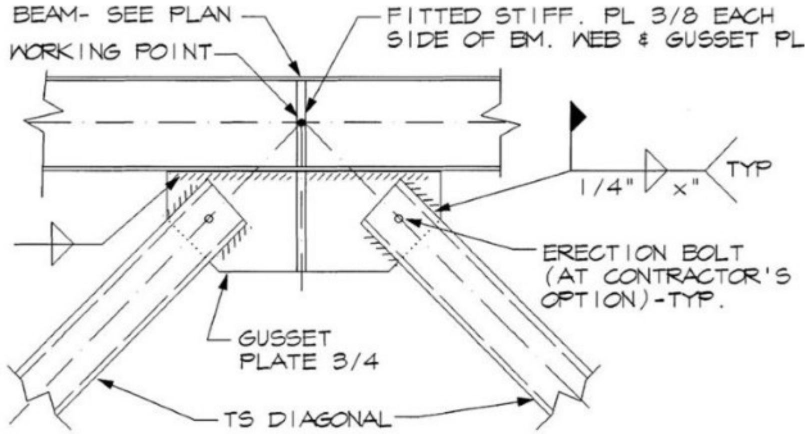
DETAIL A, AT TS COLUMN



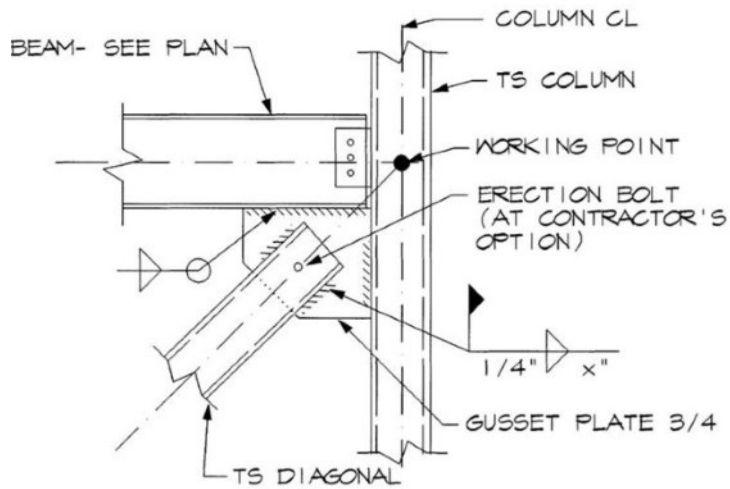
DETAIL A, AT WF COLUMN

BRACED BAY DETAILS

NOT TO SCALE (DETAIL T5-BAYD1)



DETAIL B

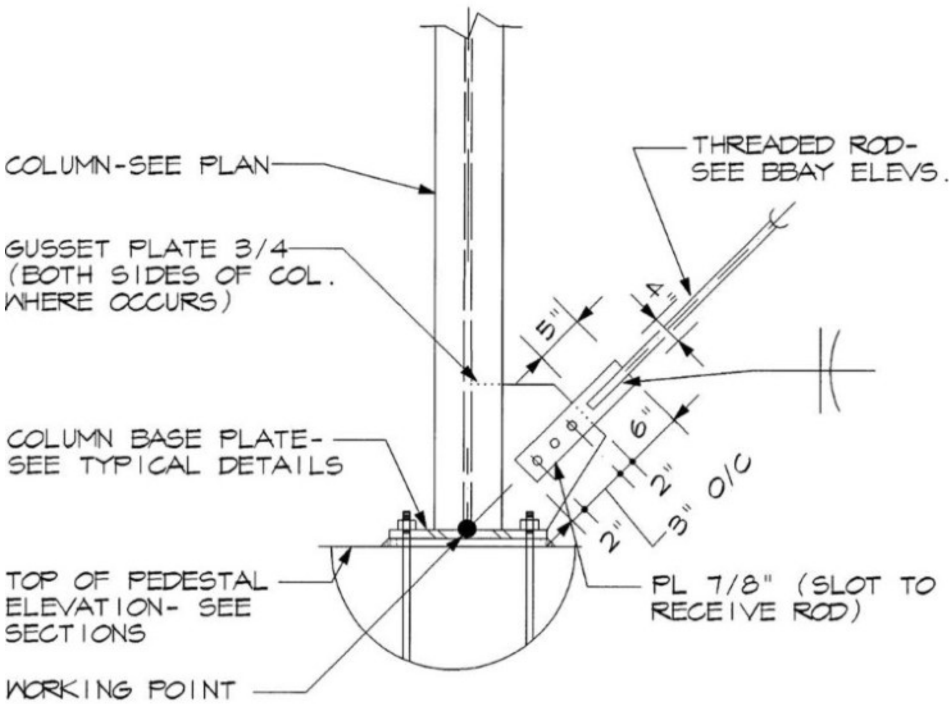


DETAIL C

BRACED BAY DETAILS

NOT TO SCALE (DETAIL T5-BAYD2)

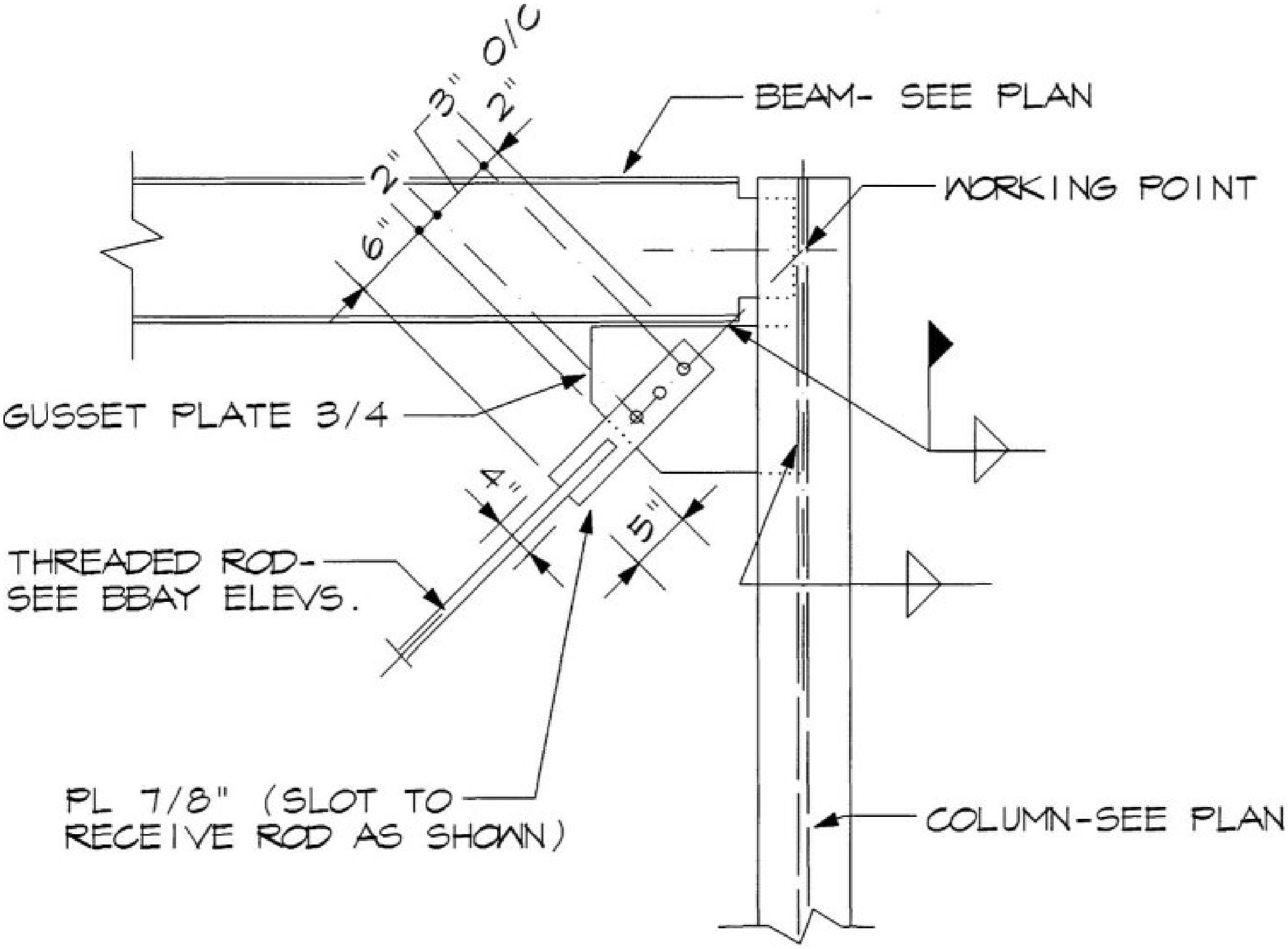
NOTE: PROVIDE BOLTED CONNECTION BETWEEN PL 7/8" AND THE 3/4" GUSSET PER THE FORCES INDICATED ON THE BRACED BAY ELEVATIONS



DETAIL A, USING THREADED RODS

BRACED BAY DETAILS

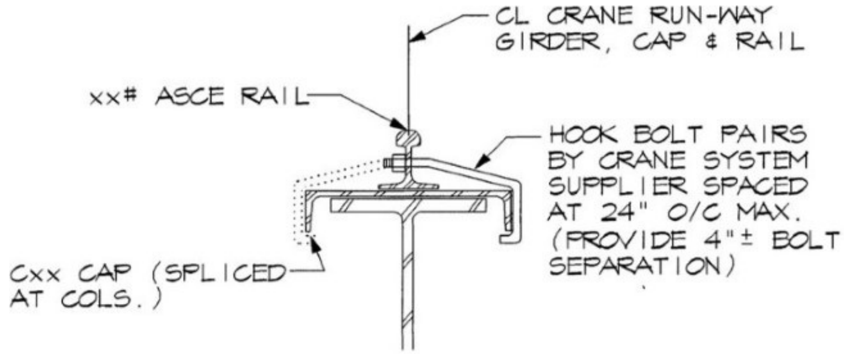
NOT TO SCALE (DETAIL T5–BAYD4)



DETAIL C, USING THREADED RODS

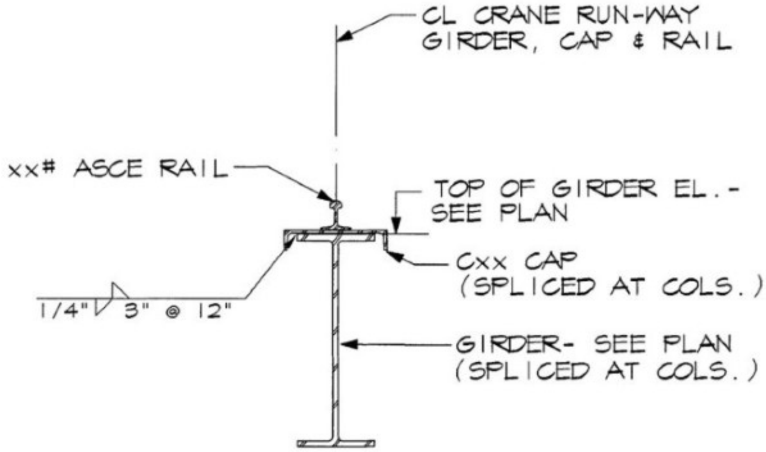
BRACED BAY DETAILS

NOT TO SCALE (DETAIL T5-BAYD5)



TYPICAL CRANE RAIL CONNECTION

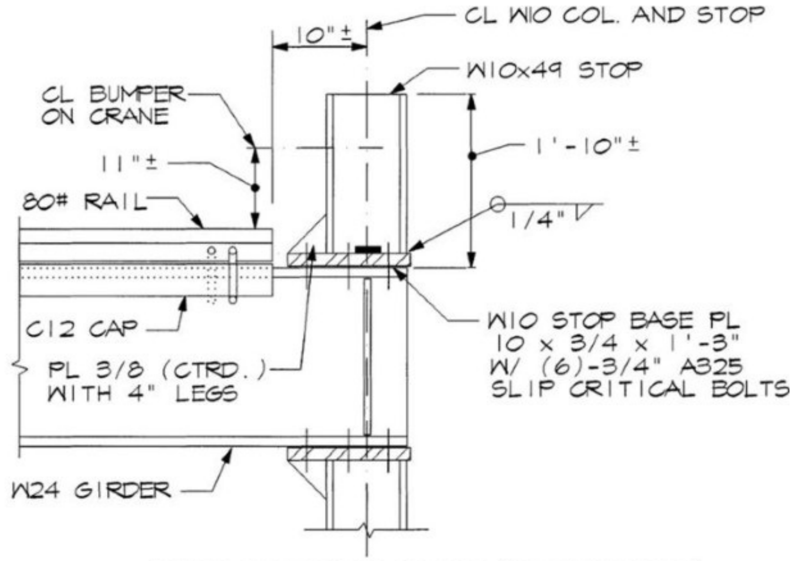
NOT TO SCALE



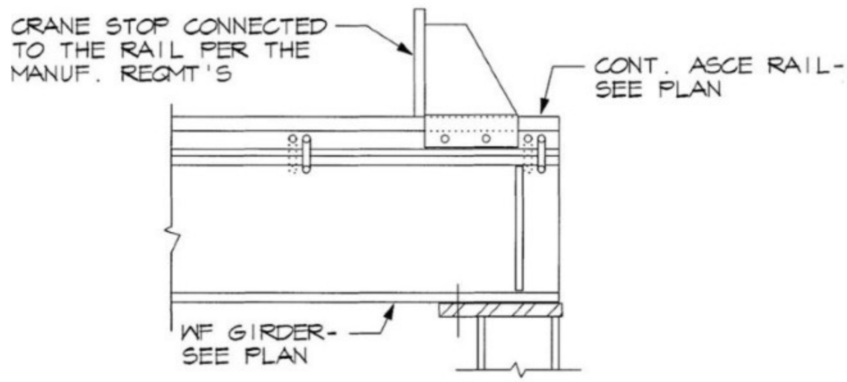
TYPICAL RUN WAY GIRDER DETAIL

NOT TO SCALE

(DETAIL T5–CRAN1)



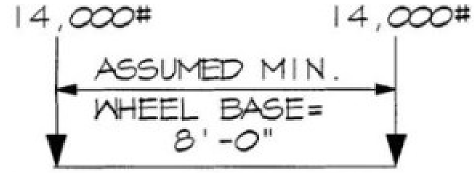
HIGH CAPACITY CRANE (30 TON MAX.)



LOW CAPACITY CRANE (5 TON MAX.)

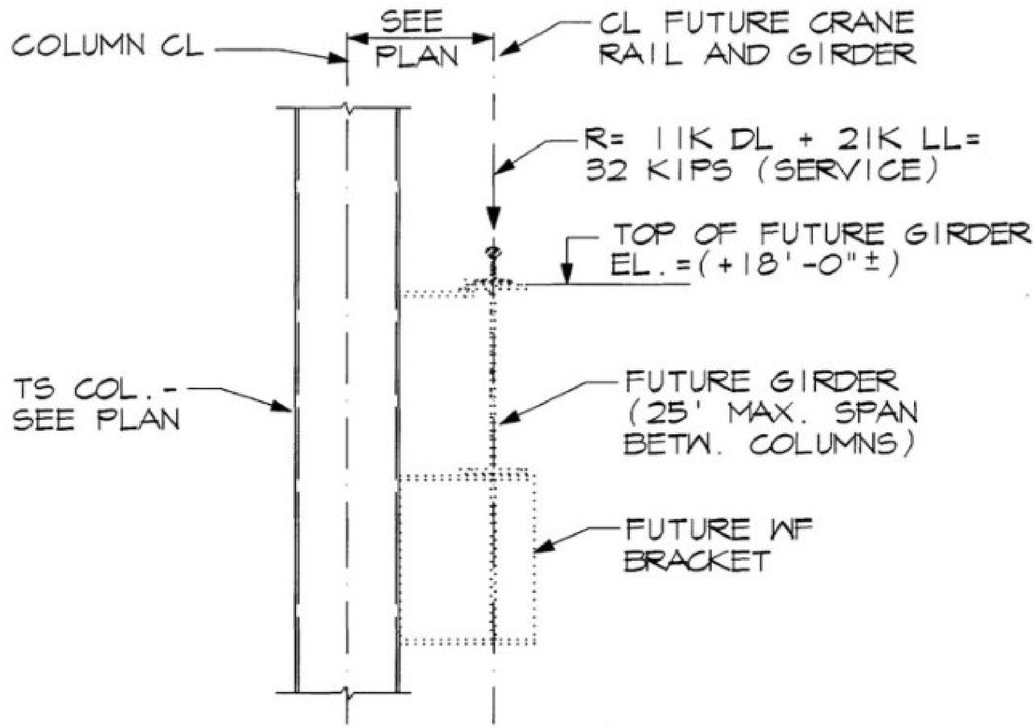
TYPICAL CRANE STOP DETAILS

NOT TO SCALE (DETAIL T5-CRAN4)



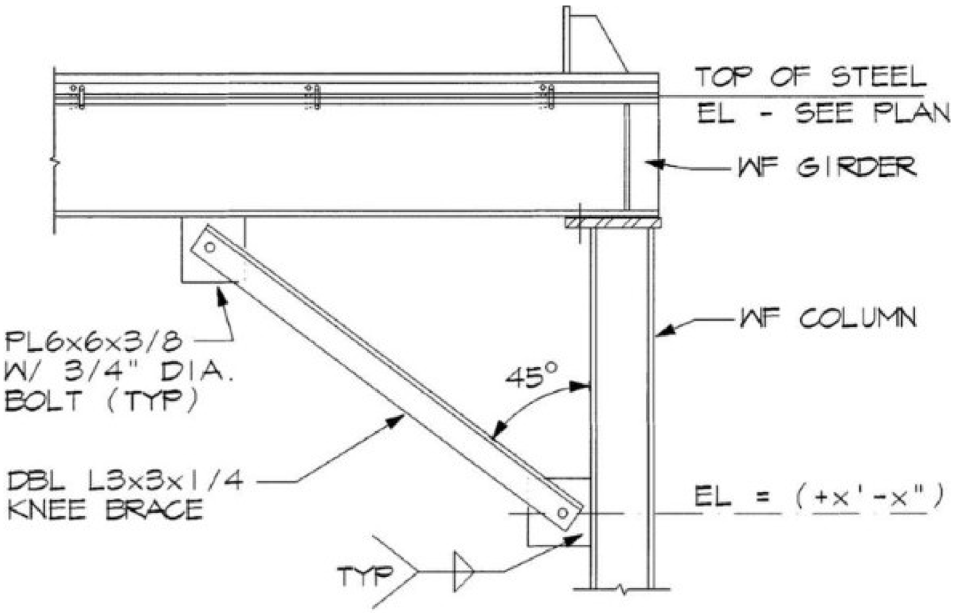
HOIST & TROLLEY WT.=2,200#
CRANE BRIDGE WT.=6,600#

ASSUMED CRANE WHEEL LOADINGS
(2 TON CRANE, 2 WHEELS PER END TRUCK)

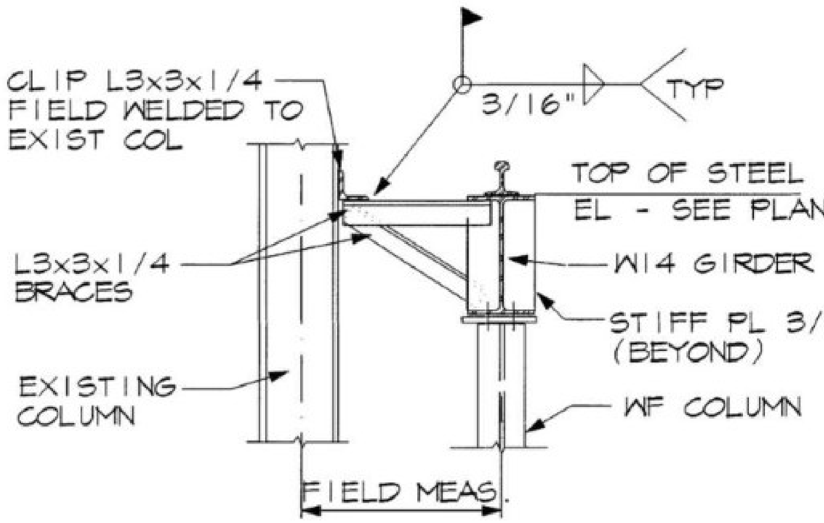


ASSUMED FUTURE CRANE CONDITION

NOT TO SCALE (DETAIL T5-CRAN8)



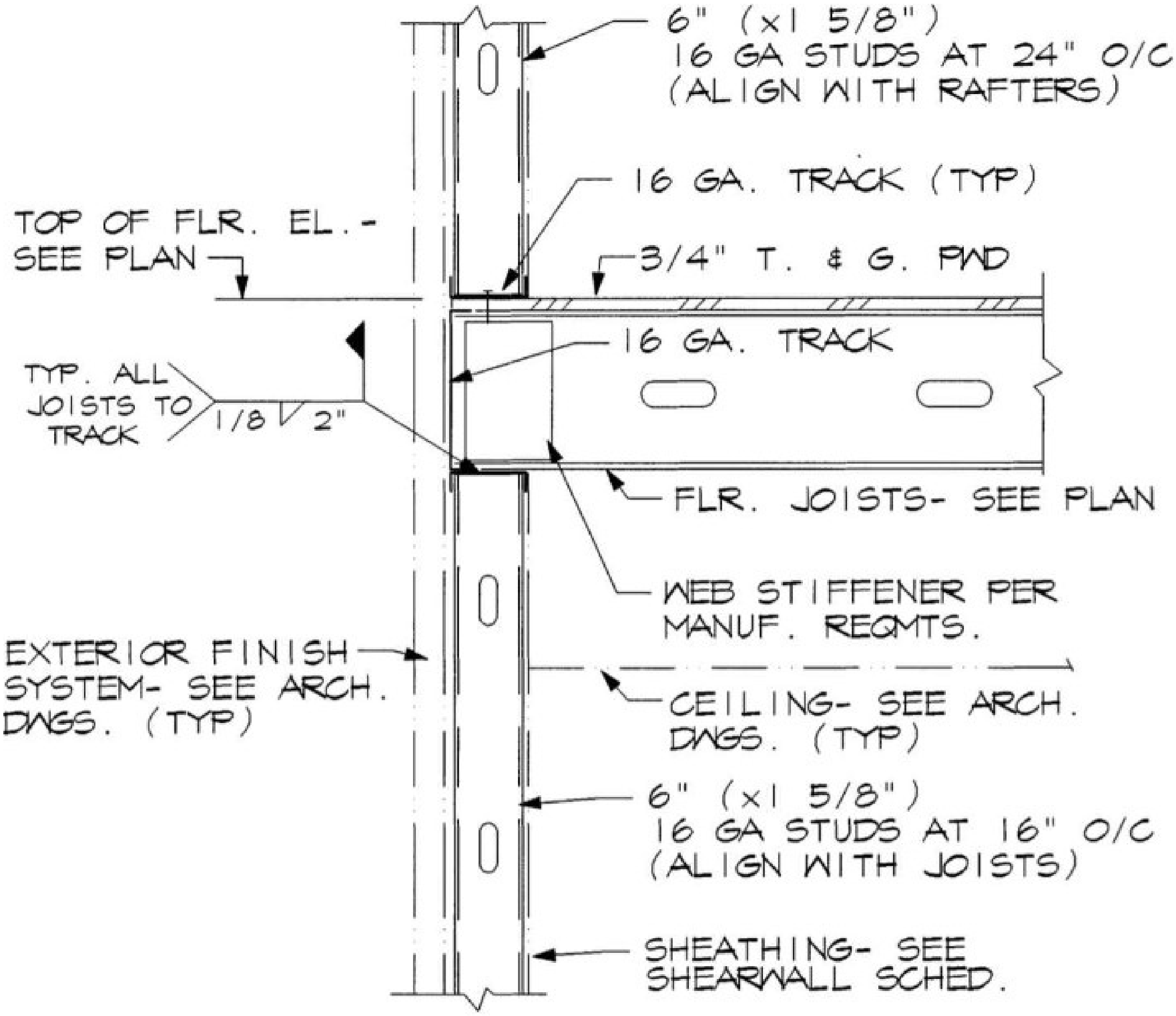
TYPICAL KNEE BRACE DETAIL



TYPICAL GIRDER BRACE DETAIL

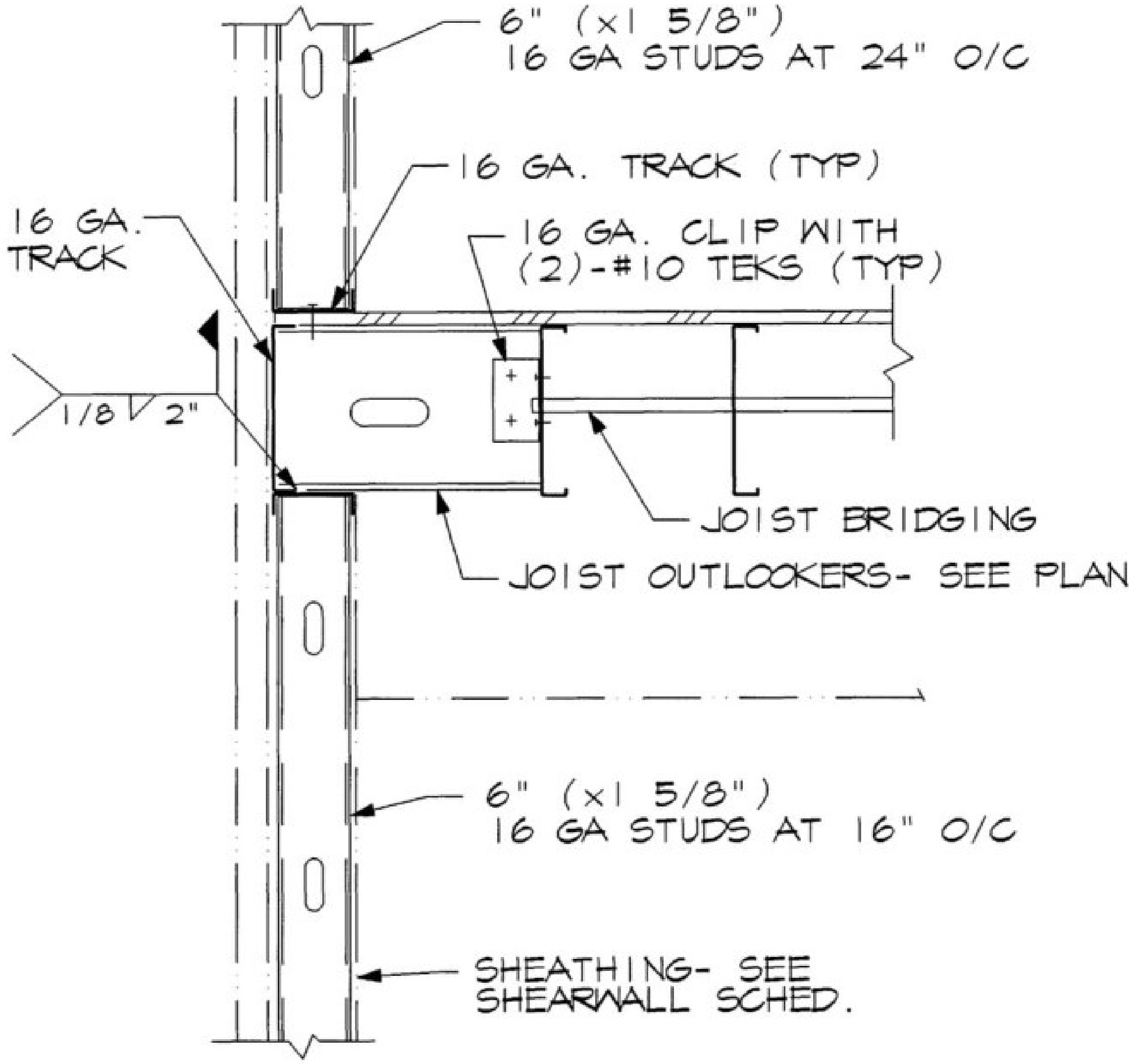
MISC. CRANE FRAMING DETAILS

NOT TO SCALE (DETAIL T5–CRAN9)



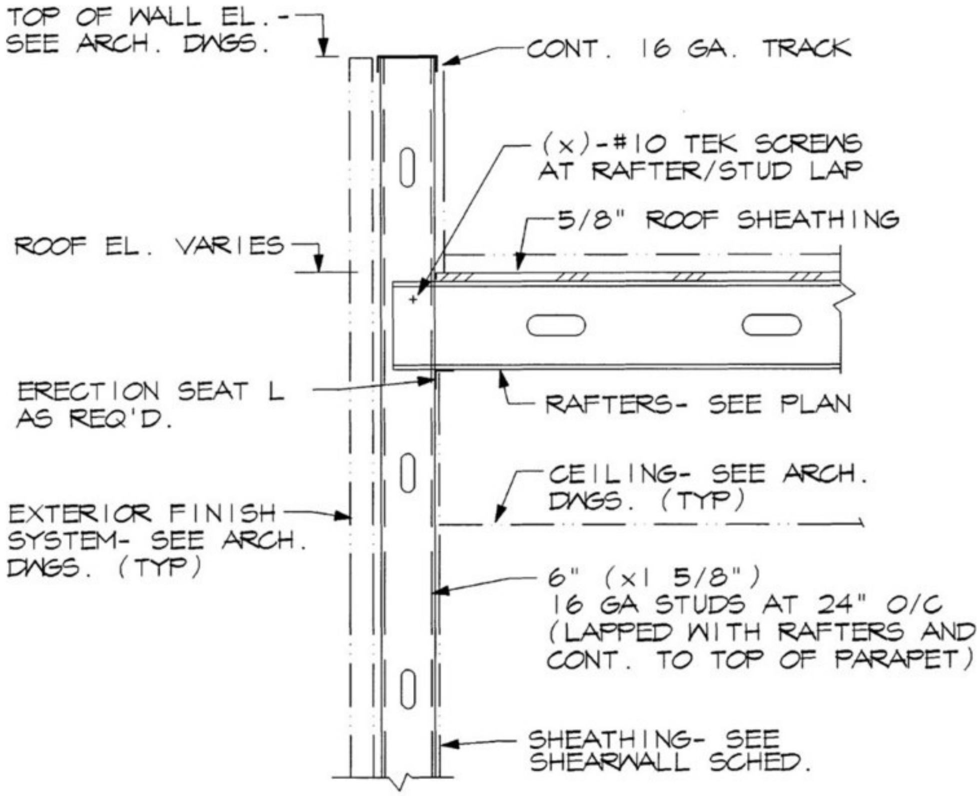
TYPICAL FLOOR JOIST BEARING DETAIL

NOT TO SCALE (DETAIL T5-CF1)



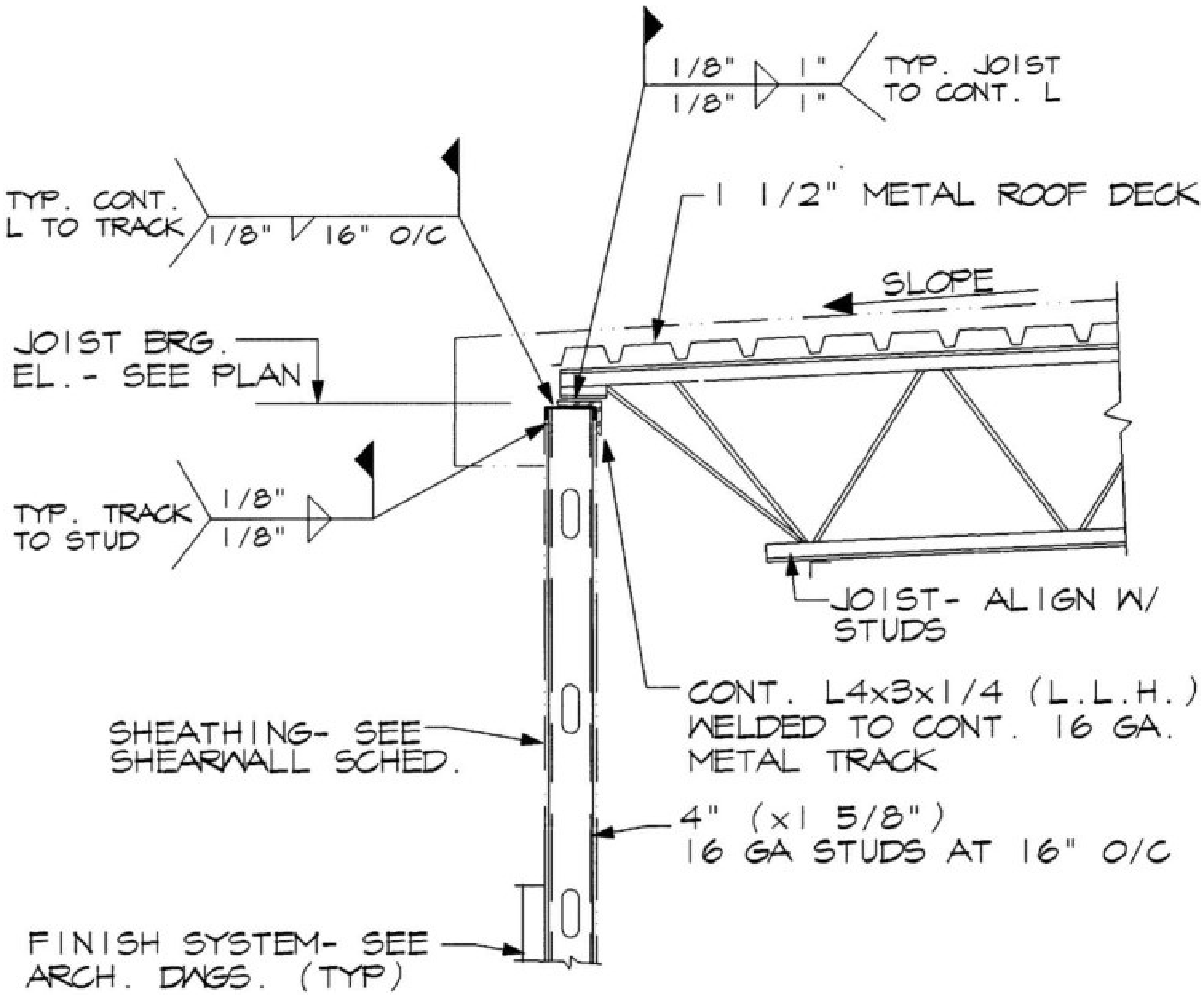
TYP. FLOOR JOIST NONBEARING DETAIL

NOT TO SCALE (DETAIL T5–CF2)



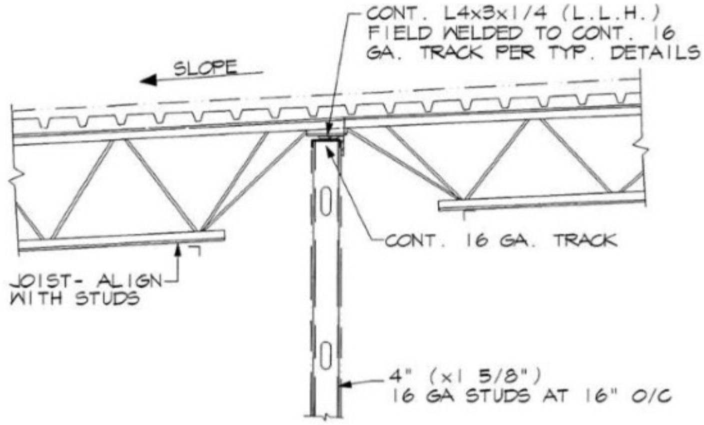
TYPICAL ROOF JOIST BEARING DETAIL

NOT TO SCALE (DETAIL T5–CF3)



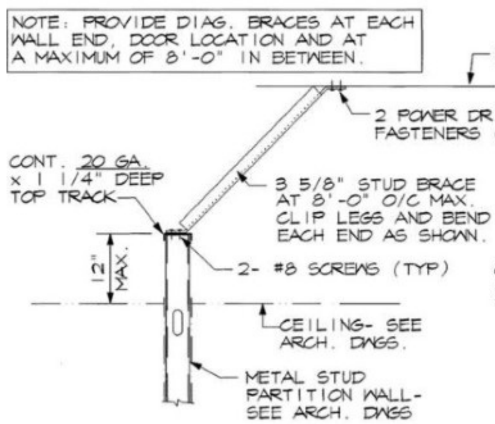
TYP. BAR JOIST ROOF BEARING DETAIL

NOT TO SCALE (DETAIL T5–CF7)

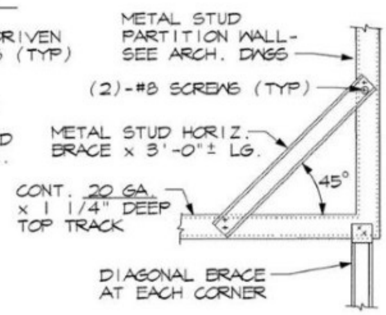


TYP. INTERIOR JOIST BEARING DETAIL

NOT TO SCALE (DETAIL T5-CF8)



ELEVATION – TYP. DIAGONAL BRACE

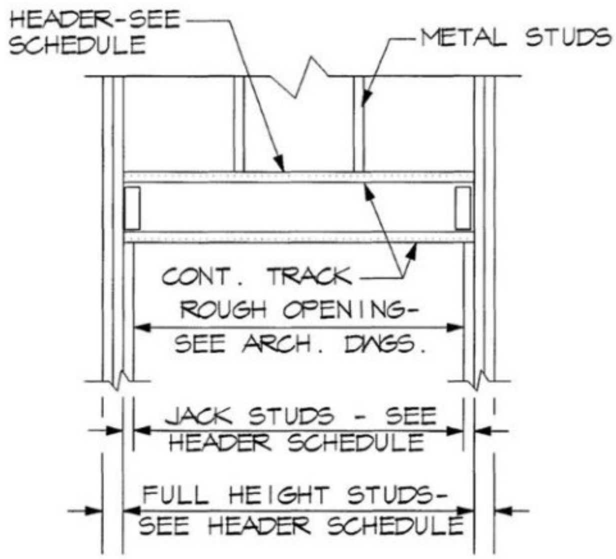


PLAN – TYP. DIAGONAL BRACE

TYPICAL PARTITION WALL BRACING DETAILS

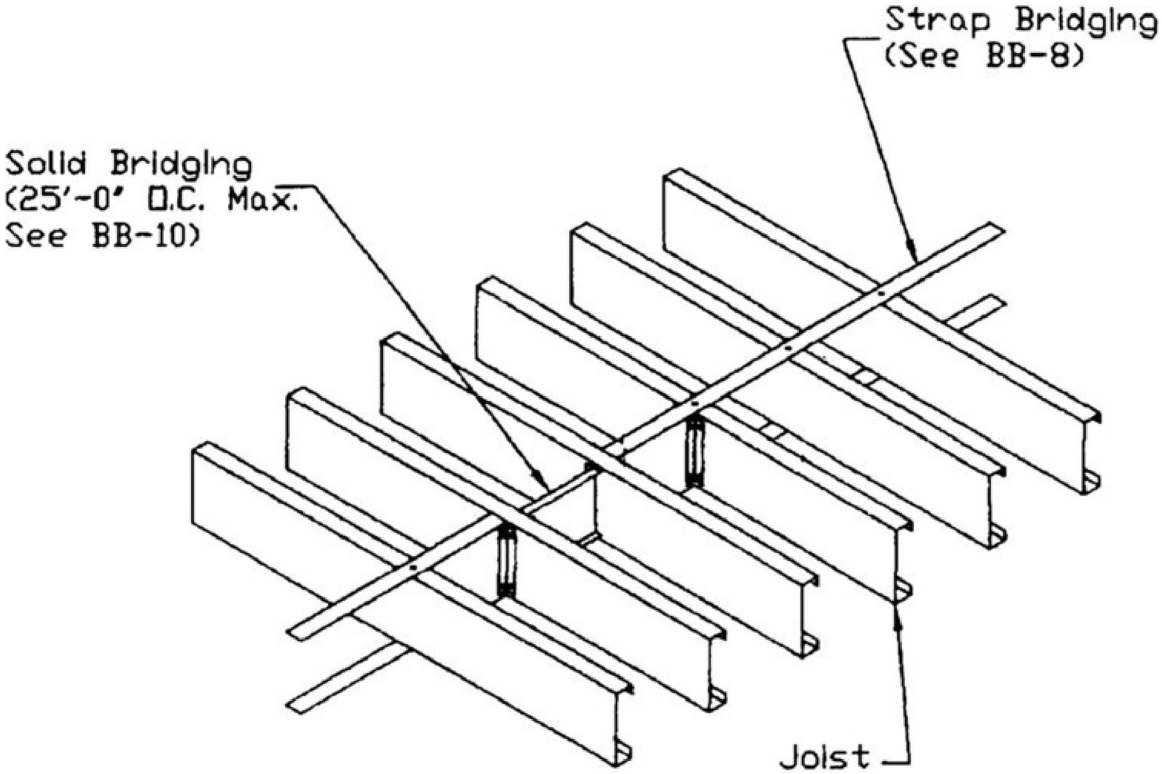
NOT TO SCALE (DETAIL T5-CFWAL)

COLD-FORMED METAL HEADER SCHEDULE					
MARK	SECTION	DESCRIPTION	GAUGE	JAMB STUDS	
				JACK	FULL-HT.
H-1	][	(2)- 6"(x1 5/8")	16 GA.	SINGLE	SINGLE
H-2	][	(2)- 8"(x1 5/8")	16 GA.	SINGLE	DOUBLE

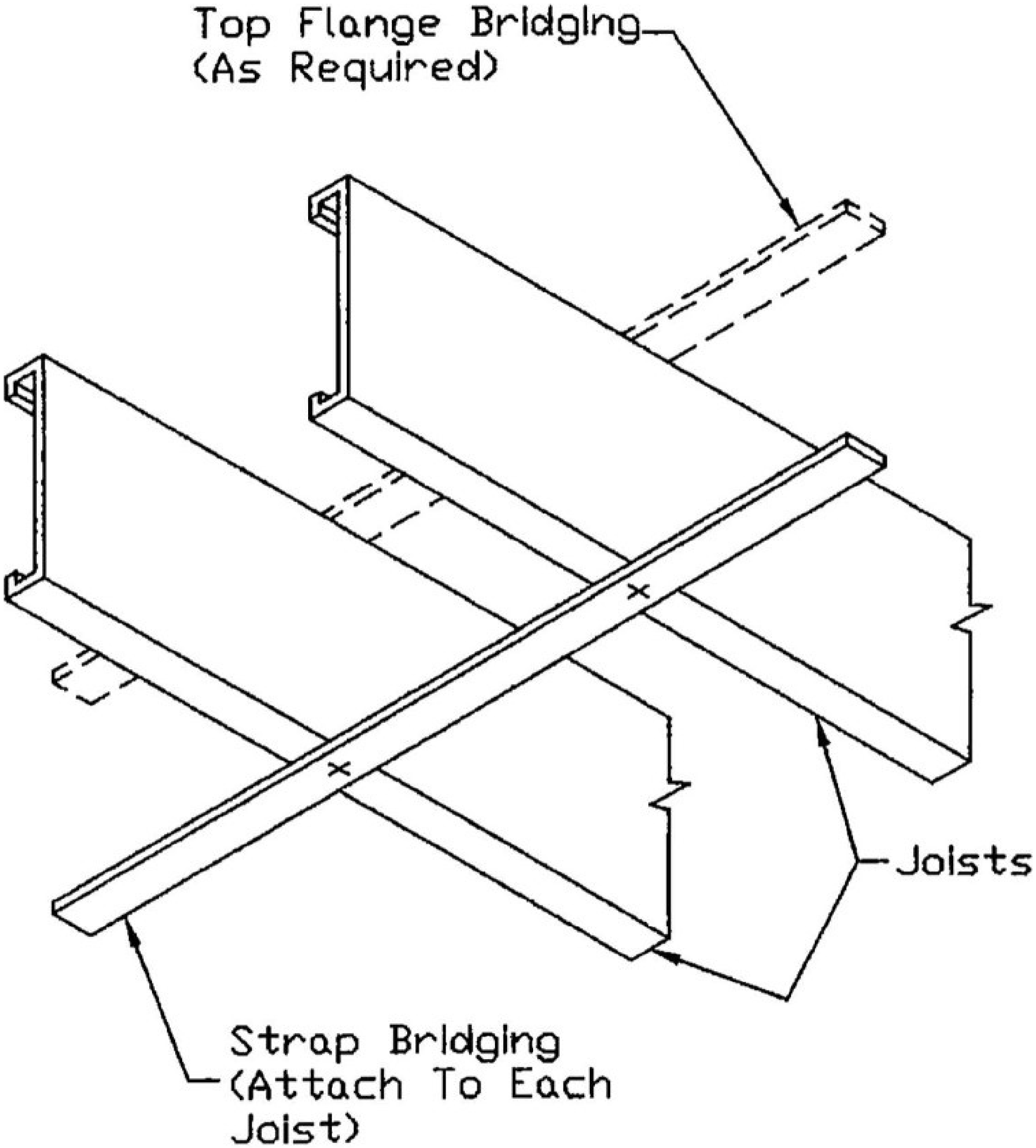


TYPICAL FRAMED OPENING DETAIL

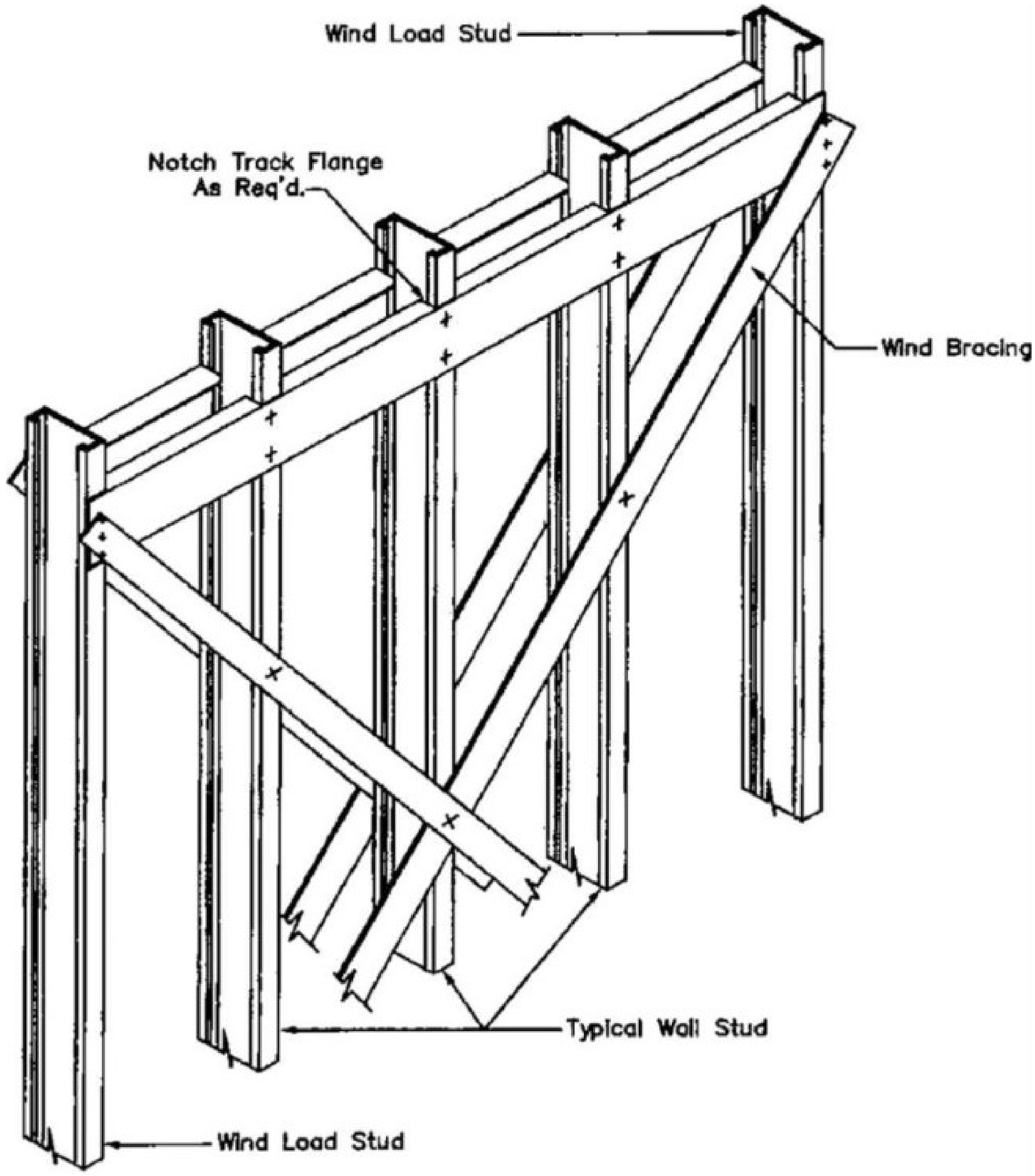
NOT TO SCALE (DETAIL T5-CFHDR)



BRIDGING AND BRACING – FLOOR BRIDGING

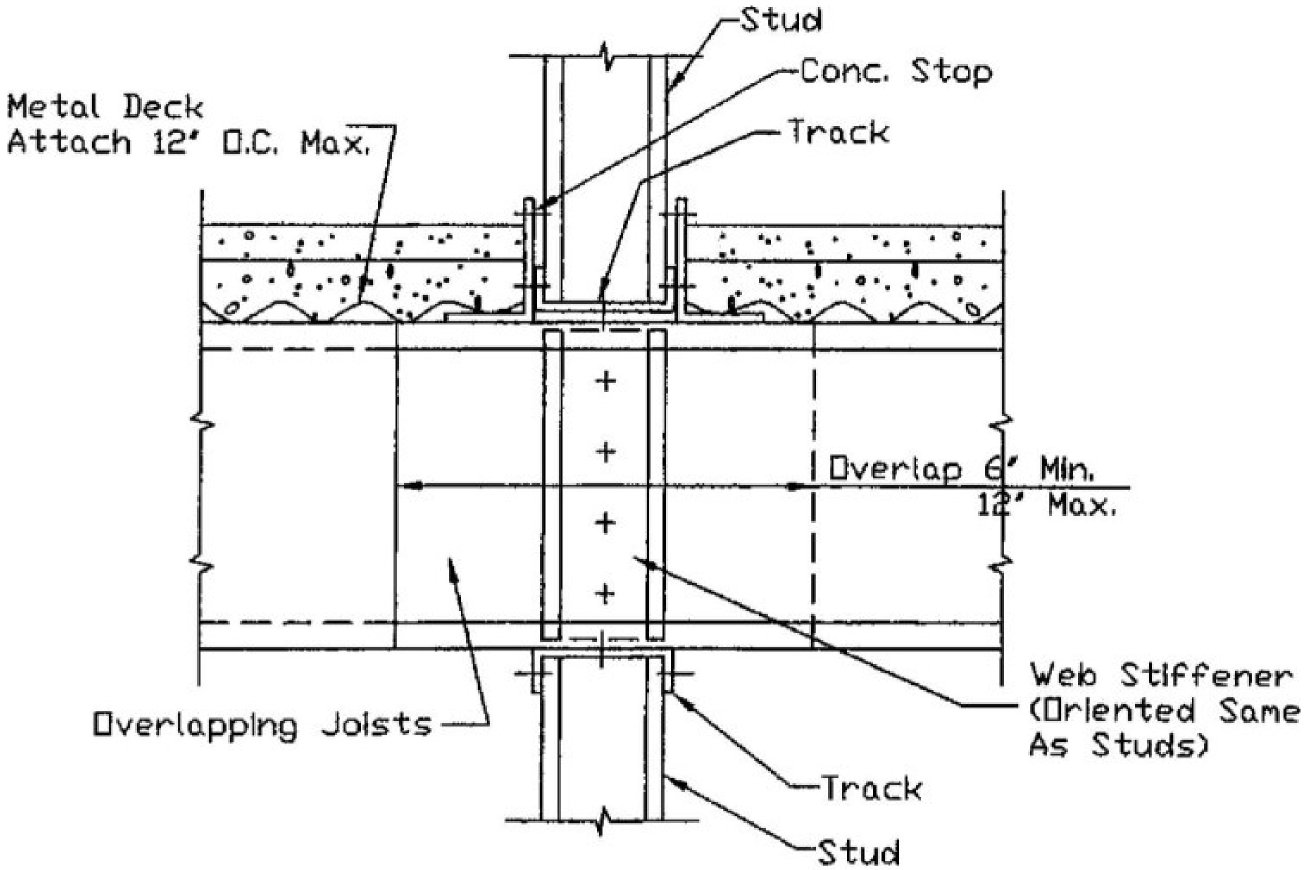


BRIDGING AND BRACING – STRAP BRIDGING



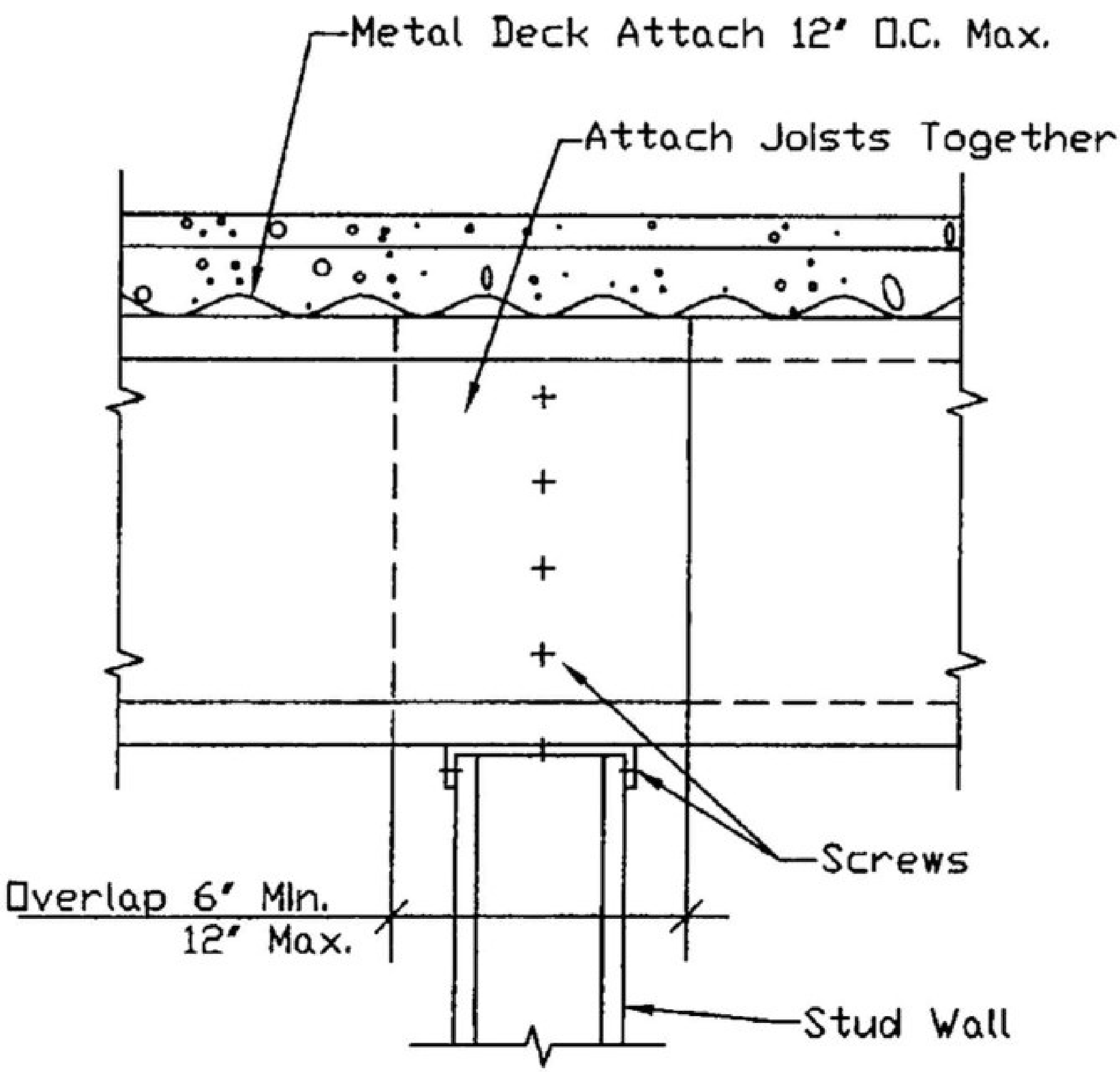
NOTE: NO. OF FASTENERS WILL VARY WITH STRENGTH REQUIRED

BRIDGING AND BRACING
Mid–Span Connection



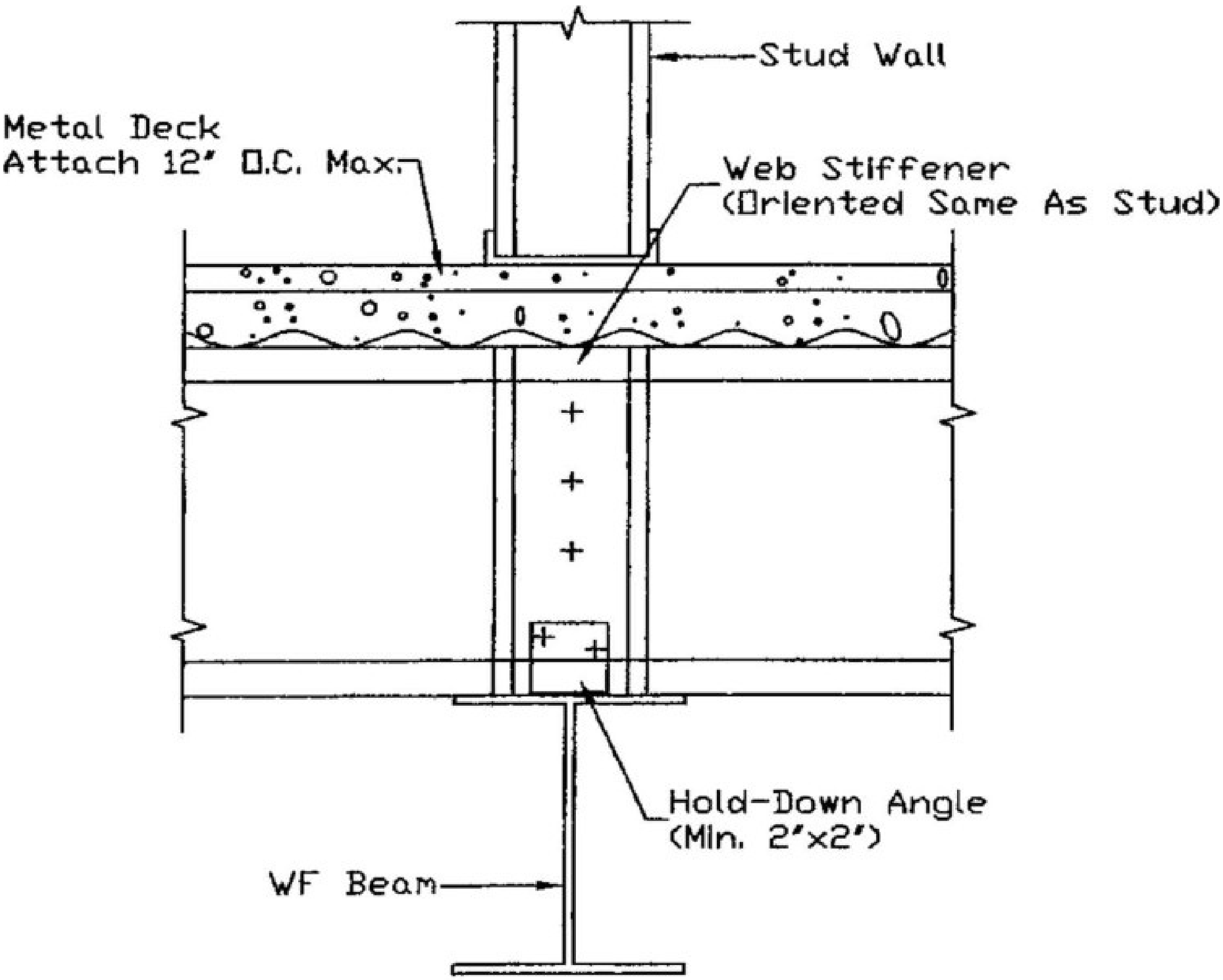
NOTE: ALIGN WEBS OF ALL MEMBERS

FLOOR SYSTEMS - CENTER BEARING ON STUDS



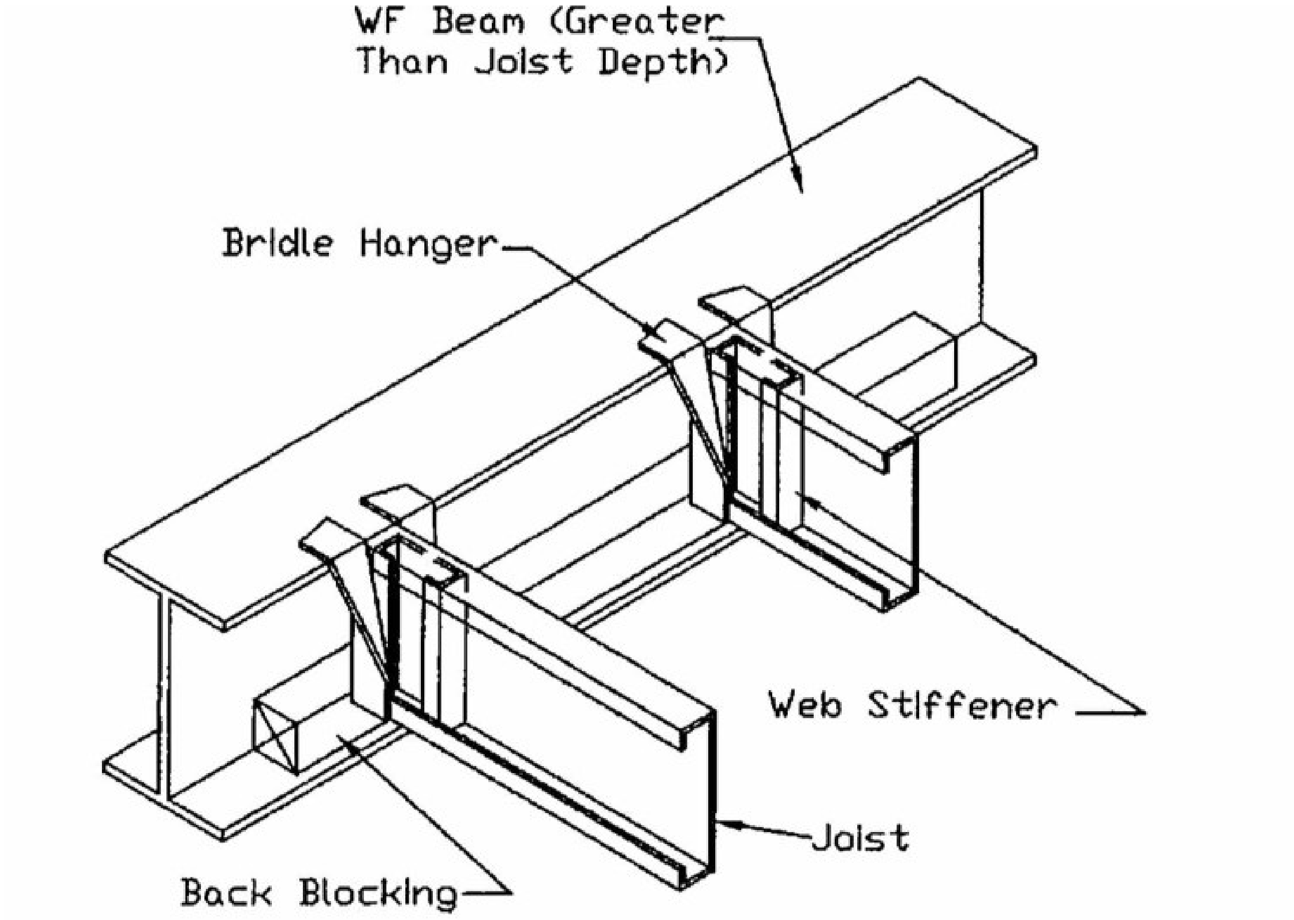
NOTE: 1. NO. OF SCREWS WILL VARY WITH DEPTH OF JOIST
2. ALIGN WEBS OF ALL MEMBERS
3. WEB STIFFENER MAY BE REQUIRED

FLOOR SYSTEMS - OVERLAPPING JOISTS



NOTE: 1. NO. OF SCREWS WILL VARY WITH DEPTH OF JOIST
2. ALIGN WEBS OF ALL MEMBERS

FLOOR SYSTEMS - BEARING ON STEEL BEAM



NOTE: 1. WELD, SCREW, OR P.A.F. ATTACH BRIDLE HANGER TO BEAM
2. ATTACH BRIDLE HANGER TO WEB OF JOIST

FLOOR SYSTEMS - CONNECTION TO WF BEAM

REFERENCE

Williams, David R., *Structural Details Manual*, McGraw-Hill, 1998.

CHAPTER 7
CONNECTION DESIGN FOR SPECIAL STRUCTURES

Lawrence A. Kloiber *LeJeune Steel, Minneapolis, Minnesota*

7.1 INTRODUCTION

Design drawings are intended to convey the engineer of record's (EOR) concept of the structure to the builder. As in any communication, there are always ample opportunities for misinterpretations or even a failure to communicate important information. The chance of a communication failure increases when constraints, such as time or budget, impact the drawing preparation and when the structural system involves unique, complex, or heavy members. Typically most of the engineer's design efforts involve laying out the structural system, structural analysis, and designing the members. Connections are often a last-minute addition to the drawing. They are usually communicated by use of schedules and standard details or, in the case of unique connections, a representative detail. In a complex structure it is almost impossible for the designer's details to show the variations required to accommodate all of the loads, member sizes, and geometry required for special connections.

Traditionally, the structural engineer establishes the strength and stiffness requirements for all connections on the design drawings along with the preferred method of force transfer. The fabricator's engineer/detailer then develops connections that comply with these guidelines. The scope of this work may vary from only establishing detail dimensions to selecting the type of connection and sizing the connection material. When designing connections for special structures it is often necessary to develop connections that involve nonstandard details or at least a modification of standard details. It is important to clearly show load-transfer requirements and to work with the fabricator to design connections that can be economically fabricated and erected and still meet all structural requirements.

Unique connections for structures, such as long-span truss connections, space-frame connections, heavy-plate connections, and splices in heavy shapes with flanges more than 2 in thick present special problems. Standard connections have been refined over the years and the problems are known. Every time you develop connections for new systems you have to be on the alert for unforeseen problems. Long-span truss connections must carry large forces while allowing for mill and fabrication tolerances and still provide easy assembly in the field. Heavy connections may have material and structural compatibility problems. Space-frame connections have access and dimensional tolerance problems. All of these may involve economic and constructability issues that require input from the fabricator and erector.

The completion of structural design drawings marks the close of only part of the total connection design process when designing special structures. Connections often need to be modified for reasons of constructibility and economy during the detailing phase. With special connections the need for modification may even arise during the fabrication or erection phases of construction when constructability problems are discovered. Special connections as mentioned previously have not been subjected to the trials of repeated use and unforeseen problems sometimes occur during construction.

Converting these design drawings into a structure requires a partnership between the EOR and the fabricator. Each has a role, the EOR as the designer and the fabricator as the builder. While they may assist each other, they remain solely responsible for the separate duties. The fabricator may size connections and propose changes in details and material but this is done as a builder not a designer. The engineer may help with construction by providing dimensional information or checking construction loads for the erector but the fabricator/erector remains responsible for the fit and constructability of the structure. This design-build approach to developing connections works well for special structures.

The EOR may elect to show representative connections, the type of load transfer that is needed, along with the required connection design forces rather than attempting to dimension each connection.

It is important when sizing members for tension forces or tension combined with moment forces that EOR review the net section requirements on representative connections to avoid expensive reinforcement at the connection. It may be necessary to increase the member size anywhere from 5 to 20 percent depending on the size of the holes, the size of the members and the strength of the material used. This will, in most cases, be more economical then welding reinforcement at the joint.

The fabricator using the forces given then selects the type and sizes connection material based on these requirements and provides all of the detail dimensioning of the connection. The EOR can then review and verify that the connections are adequate for his or her design. This method of developing connection details utilizes the knowledge and experience of both the EOR and the fabricator in the most efficient way.

The detailing phase should start with a predetail conference with the EOR, fabricator, and where necessary the erector and general contractor or construction manager attending. Preliminary sketches and schedules of some connections may be submitted at this time. This initial meeting is an effort by the entire construction team to understand the structural concept, verify whether all the needed information is shown, and determine if there are any obvious constructability problems.

Connection design does not stop with the approval of shop drawings by the EOR. The beginning of shop fabrication often presents additional challenges. Material ordered for the project may not conform to specifications, fabrication errors may occur, and unforeseen constructability problems might be discovered. The fabricator must evaluate each problem to determine if a modification or repair is necessary. Even though shop supervision or quality control personnel may identify the problem, it is important that the fabricator's engineer review and document any modification. If it is determined that the connection, even after repair or modification, will not meet the original standards, the proposed action must be submitted to the EOR to determine if the connection as fabricated will be fit for its intended purpose.

The erection of the steel frame serves as a check of the fabricator's efforts to detail and fabricate connections that fit perfectly. If the erector cannot put the bolt in the hole, it may be necessary to modify the connection. Most minor fit-up problems can be resolved with reaming, slotting, or shimming. Larger-dimensional errors or other constructability problems may require the fabricator to develop a new connection detail that requires the approval of the engineer. Again, it may not be feasible to provide a connection that meets the original design standard and the EOR will be called on to make a fitness for purpose determination.

It is very important that the EOR be made aware of, and carefully review, any connection modifications during the entire detailing, fabrication, and erection of special structures. The load transfer is often so complex that only the EOR can evaluate the effect of any modification on the service and strength of the structure.

7.2 LATERAL LOAD SYSTEMS

Bracing systems usually involve some of the most complex shop details, require the most labor to fabricate, and are the members most likely to have field fit-up problems. These members, however, are often shown with the least detail on the design drawing. Typical bracing elevations should members sizes, the location of the work points along with the complete load path for the forces. The connection designer must be able to determine how the loads accumulate and are transferred from the origin of the force to the foundation in order to design all of the connections for the appropriate forces. This includes knowing diaphragm shears and chord forces, collector forces, and pass-through forces at bracing connections. The designer’s failure to provide a complete load path may result in critical connections not being able to deliver the design forces to the bracing system.

Diaphragm chords and drag struts often serve as gravity load members in addition to being part of the bracing system. It is important to design the connections for these members for both shear and axial load. Wide-flange beams with heavy framing angles can transfer axial force of the amount found in drag struts for most bracing systems. See Chap. 2 of this book for more detail on how to design this type of connection and the limits on capacity while still maintaining a flexible-type connection. When these struts are joist or joist girders, special connection details are required. Joist and joist girder end connections typically are able to transmit only a few kips because of the eccentricity between the seated end connection and the axis of the top chord. A field-welded tie consisting of a plate or pair of angles near the neutral axis of the top chord angles is the preferred method of passing axial forces across these types of joints. Drag struts with very large transfer forces occur when it becomes necessary to transfer the entire horizontal force of a brace to a brace in a nearby bay. Members with large axial forces will usually require heavy connections that will be rigid. Consideration should be given to designing these members with fully restrained connections.

The use of concentric work points at bracing joints makes the analysis of the frame and the design of members easier but may subject the connection material to eccentric loads or result in awkward, uneconomical details. This usually occurs when bracing slopes are extreme or member sizes vary substantially in size. It is important when designing bracing to determine if the work points chosen will result in efficient use of connection material. The connection of the diagonal bracing member to the strut beam and column can be efficiently designed using procedures such as the uniform force method found in Chap. 2 (Fig. 7.1).

FIGURE 7.1 Large gusset plate required due to concentric work point for brace.

Shear-wall systems are simpler to detail and normally involve knowing only drag strut forces or diaphragm forces. These force transfer systems must be clearly shown and detailed. When the structure depends on shear walls, precast panels, or horizontal diaphragms for lateral stability, it is important that the general contractor and erector know this. The erector, by standard practice, provides erection bracing only for lateral loads on the bare frame. The construction sequence may require the general contractor or erector to provide additional bracing because the permanent lateral load system is not complete.

Moment frame systems are often very conservatively shown with notes calling for connections with a strength equal to the full section. While this may be required in seismic zones, lateral load systems are often designed for wind and the members are often sized for stiffness. There can be a substantial savings if the connections are sized for the actual design forces rather than the member strength. It is important when designing connections for wind frames to know the size of the moment in each direction. Typically, the maximum tension at the bottom flange is substantially less than at the top flange due to the combination of wind and gravity moments. When designing moment connections and checking column stiffener requirements, the use of these reduced tension loads may provide simpler, more economical connections.

7.3 LONG-SPAN TRUSSES

Long-span trusses can be divided into three general types based on the methods of fabrication and erection. Trusses up to approximately 16 ft deep and 100 ft in length can be shop-fabricated and shipped to the field in one piece. When these trusses are more than 100 ft, they can be shop-assembled in sections and shipped to the field in sections for assembly and erection. Trusses more than 16 ft deep are generally fabricated and shipped as individual members for assembly and erection in the field. The first two types usually have standard connections that are discussed in other chapters. The third type, because of the size and loads carried, has special connection design requirements.

Deep long-span trusses typically use wide-flange shapes with all of the flanges in the same plane as the truss. If all of the members are approximately the same depth, connections can be made using gusset plates that lap both sides at the panel joints. When designing web members, it is important to look at the actual depth of the chord member rather than the nominal depth. For example, when using a W14 × 311 for a chord, where the actual depth is more than 17 in, it would be better to use a W16 × 67 than a W14 × 61 for a web member. The W14 × 61 would require the use of 2⅛-in fills on each side. Truss panels should be approximately square for the most efficient layout of gusset-plate connections when using Pratt-type configurations. When using Warren-style panel, it may help to increase the slope of the web member to make a more compact joint while reducing the force and length of the compression diagonal. Chords should, where possible, splice at panel points in order to use the gusset plates and bolts already there as part of the splice material. This also makes it possible to provide for camber or roof slope by allowing a change in alignment at a braced point. The gusset plates on a Pratt-type truss will be extended on the diagonal side to allow for bolt placement in this member. For this reason, the gusset plates should be first sized to accommodate the web member connections and the chord splice placed near the center of the plate rather than at the actual panel point intersection. The plate size is then checked for chord splice requirements.

This type of truss typically uses high-strength bolts for all connections. Traditionally, these have been bearing-type connections in standard holes. This requires either very precise computer numerical controlled (CNC) drilling or full shop assembly with reaming or drilling from the solid. Until recently with CNC equipment it has been very difficult with heavy members to obtain the tolerances needed for reliable field fit-up when using standard 1/16-in hole clearances. Shop assembly and reaming or drilling from solid is very expensive and because of the overall truss size it may not be possible for some fabricators to do this. Recent improvements in CNC hole making equipment combined with improved accuracy achieved downloading machine instructions directly from a model along with the change in AISC Specifications to allow ⅛-in hole clearances for high strength bolts 1 in diameter or greater should permit a greater use of bearing type of bolts.

There has been a trend in recent years to use oversize holes with slip-critical bolts to allow tolerances that are readily achievable by most drill lines. While this increases the number of bolts and gusset-plate sizes, this can be offset by using larger A490 bolts. Bolt material costs are approximately proportional to the strength provided. For example, while the cost of a 1-in-diameter A490 bolt is more than a ⅞-in A325, the number of bolts required is substantially less. While the cost of the bolt material required does not change as the size and grade increase, the cost of plate, hole making, and installation costs decrease so larger diameter higher-strength bolts are usually cost effective.

Bolt size selection is also dependent upon the magnitude of force to be transmitted, the net section requirements of the members, and the tightening methods to be used. Generally for the loads and member sizes used in this type of truss, a 1⅛-in diameter A490 bolt is an efficient choice. The AISC specification provisions that use yield on the gross section and fracture on the net section as previously mentioned should be checked when sizing tension members. Chord splices where double gage lines are sometimes used it maybe necessary to use two or three rows of bolts at a single gage as lead-in bolts (Fig. 7.2). It is also important to check for shear lag using the net section provisions of the AISC specification when connecting only to the flanges of members. If possible, all bolts should be designed for single shear. This is especially true at splices that change slope since any splice plate on the inside will have to accommodate the change in slope by skewing the holes in a relatively narrow width plate. It may, however, be necessary to use bolts in double shear at tension chord splices to limit splice length. Compression chord splices should generally be designed as finished to bear type joints with bolts sized for half of the design force or for the application of a transverse force of 2% of the member force. Since these bolts will be slip-critical in oversize holes, it may require the use of mild steel shims in the joint to achieve the detailed chord dimension. Oversize holes should generally be detailed in all plies of material. This will allow the use of full-size drift pins to fair up the hole and make it easier to align the truss. Slip-critical bolts require special procedures to properly tension the bolts and must be inspected to ensure the required tension is achieved. While a slip into bearing is a service failure and not a collapse, it is important to establish a quality program that will ensure the work meets the design requirements.

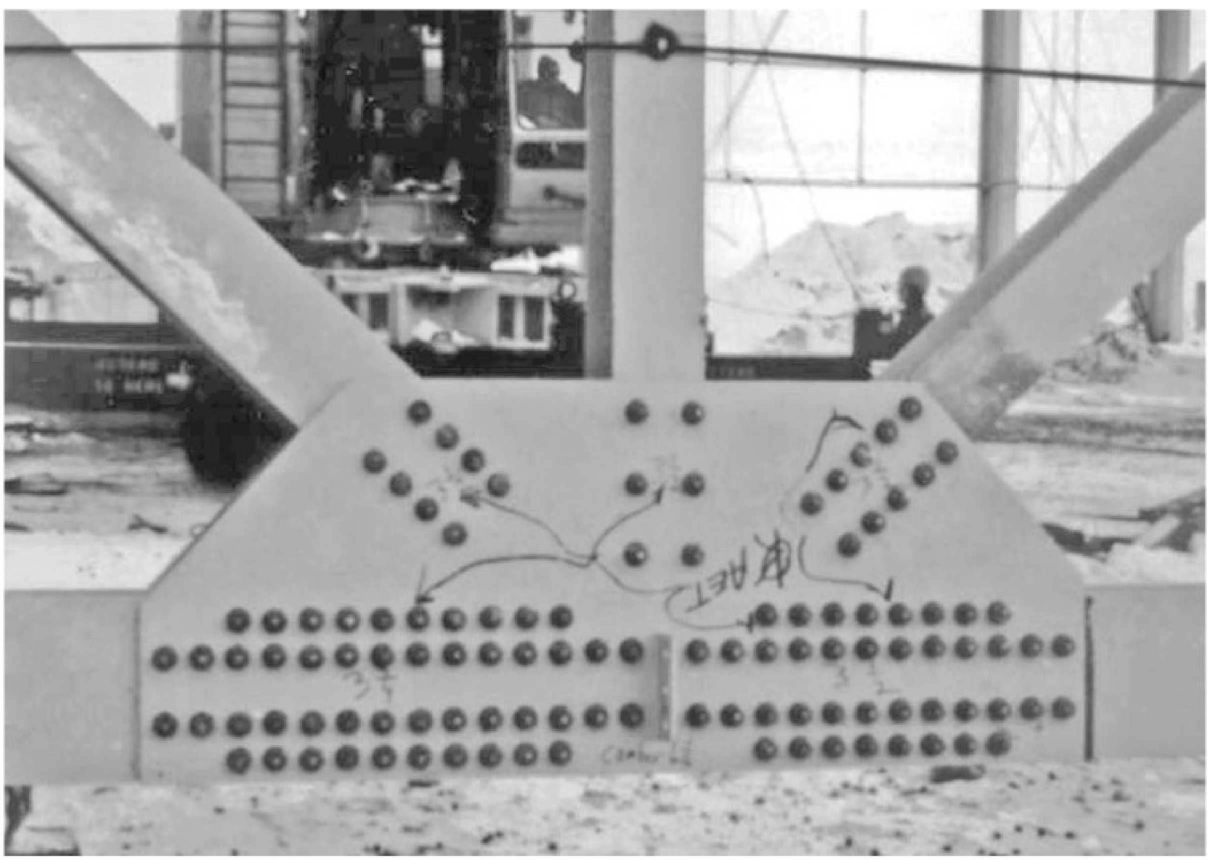


FIGURE 7.2 Large truss gusset plate with lead-in bolts at splice.

Since most trusses will be assembled in large sections on the ground, it is important to design the major gusset-plate connections so all of the bolts, except the splices between sections, can be tensioned and inspected on the ground where they are easier to install and inspect. Secondary framing connections should be made with plates shop-welded to the gusset plates rather than using some of the truss connection bolts for both connections (Fig. 7.3).

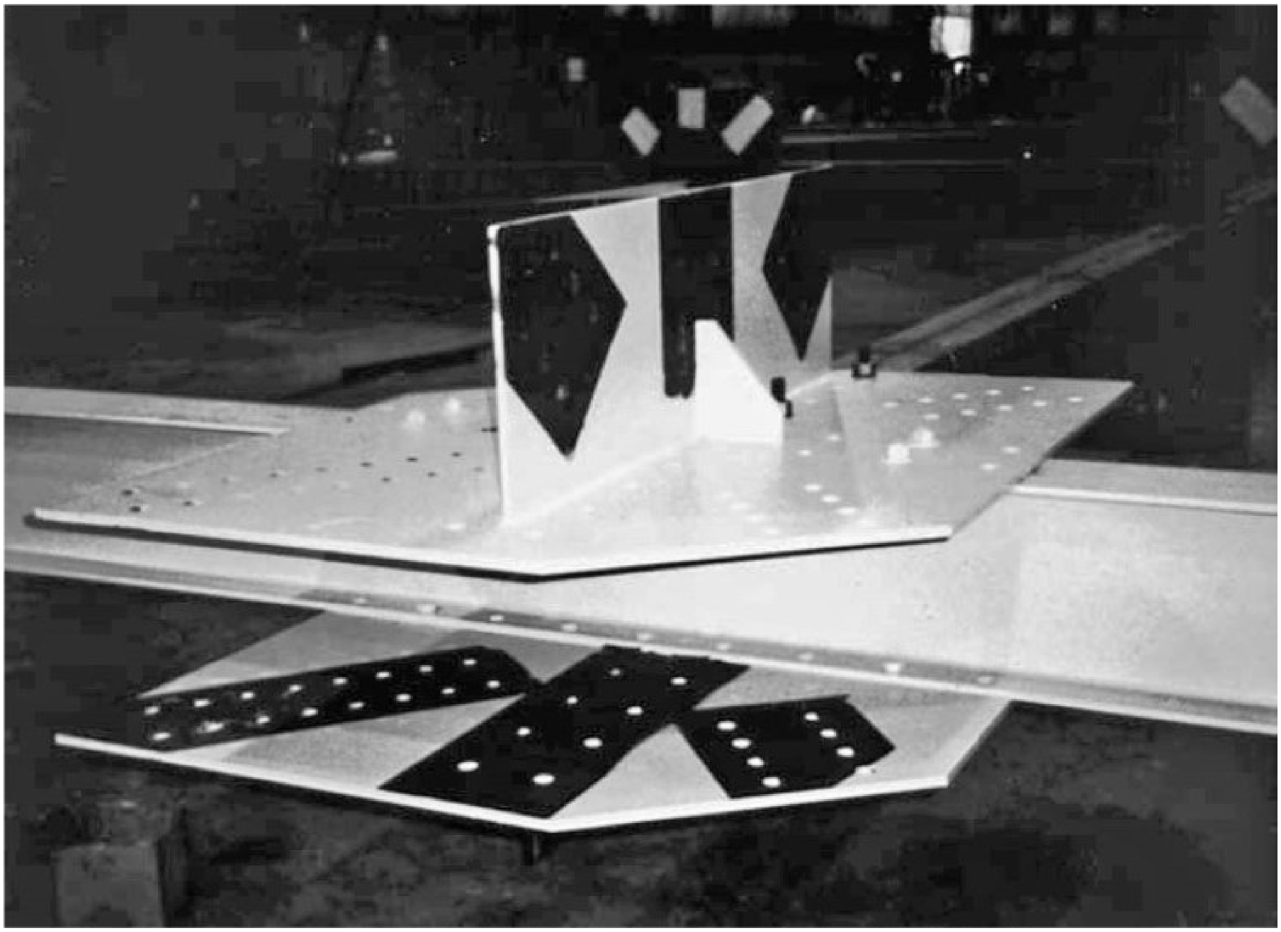


FIGURE 7.3 Truss gusset plate designed with bolts independent of secondary framing connections.

A trial joint should be assembled, tensioned, and inspected with the fabricator, erector, bolt supplier, independent testing laboratory, and the engineer of record present. Written procedures for both bolt installation and inspection for the project should be developed and agreed upon by all parties (Fig. 7.4).

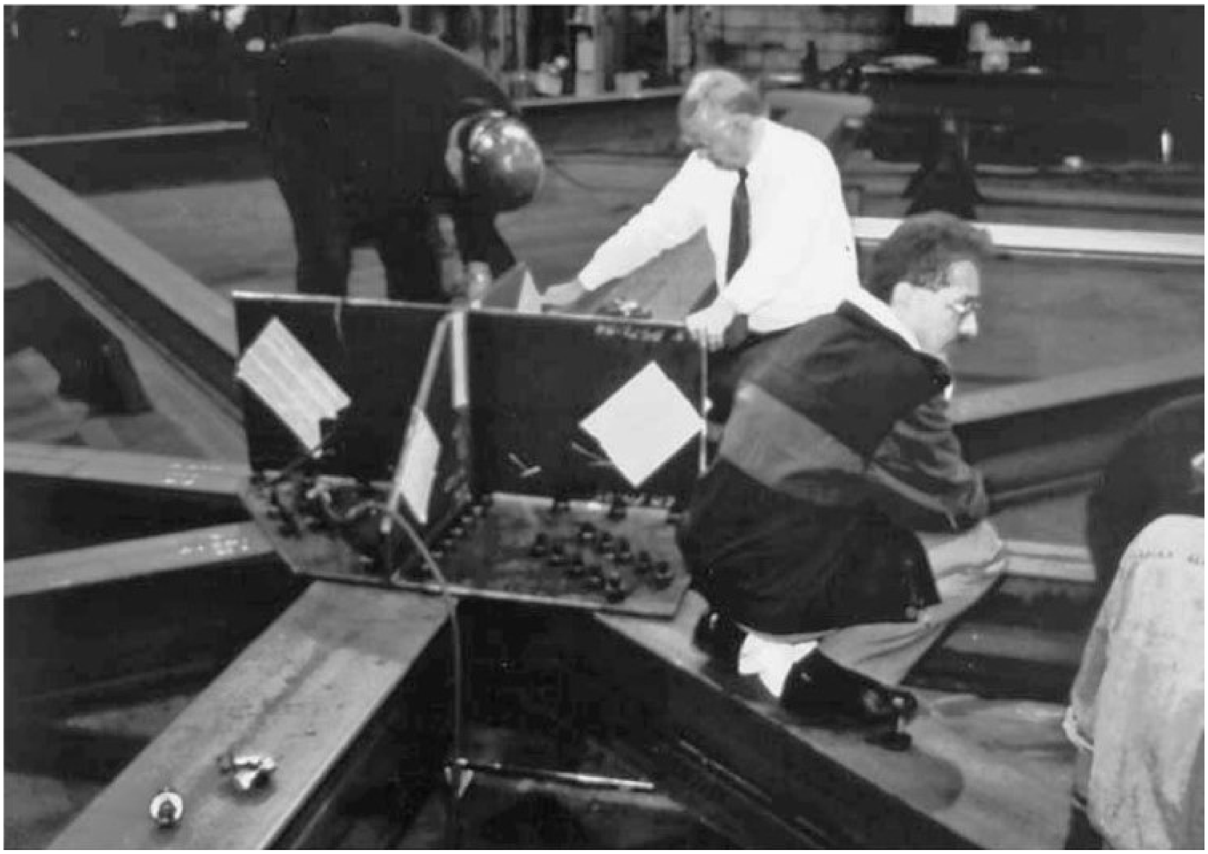


FIGURE 7.4 Trial joint assembly to establish bolt tensioning and inspection procedures—NWA Hangar, S.E. McClier.

7.4 SPACE-FRAME STRUCTURES

The space truss form is often selected for either architectural appearance or because of depth limitations. Since one-way long-span trusses are easier to fabricate and erect, they will almost always be more economical than space trusses even though they will weigh more.

Space-frame structures have connections that must transfer forces on all three axes. They have access, dimensional tolerance, and through-thickness strength problems. Because of the complexity of these joints, it is important to try and develop some type of universal connector that can be reliably fabricated or to design the structure with large shop-welded assemblies that can be connected in the field. There are patented space frames that use special steel connectors. These are typically lighter structures with somewhat limited configurations which are not covered here.

Connectors for field assembly of structural steel space frames can be designed using a through plate for the major chord force and intersecting plates for members in the other planes. These intersecting plates are generally complete-joint-penetration welded to the primary plate. When the geometry is especially complex, it may be necessary to use a center connection piece, usually a round member, to provide access to weld the joint. In either case, the through-thickness strains due to welding make it advisable to use a low-sulfur steel with a good through-thickness ductility. This material is expensive and not readily available so it is important to standardize on as few plate thicknesses as possible and use this only where needed. The welding procedure and filler metal should be evaluated to determine if it is adequate for both the design and fabrication requirements (Figs. 7.5 and 7.6). The attachment of the truss members to these connectors in the field can either be by welding or bolting. Shop-welding and field-bolting may provide better quality control but this system generally requires two connections. Field-welding typically requires only one connection and generally provides more fit-up tolerance. If field-welding is used, it is important to try to use primarily fillet welds and, if possible, limit the out-of-position welding. Because some type of erection connection or shoring will be required until the structural weld is made, space trusses should, where possible, be designed so they can be ground-assembled. Their inherent stiffness allows them to be hoisted or jacked into final position after full assembly (Fig. 7.7). This is very important for economy, quality, and safety.

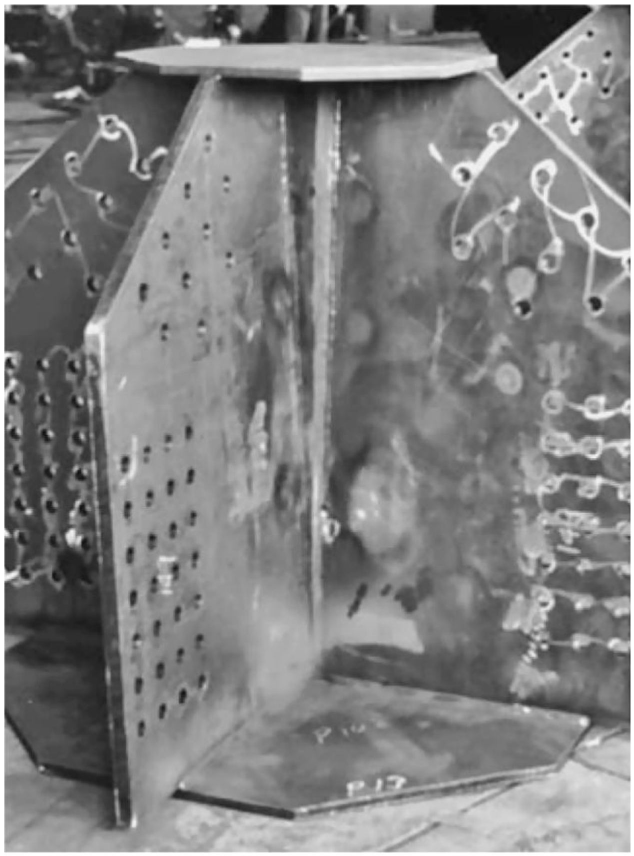


FIGURE 7.5 Space-frame connector—Carver-Hawkeye Arena, S.E. Geiger—Berger.

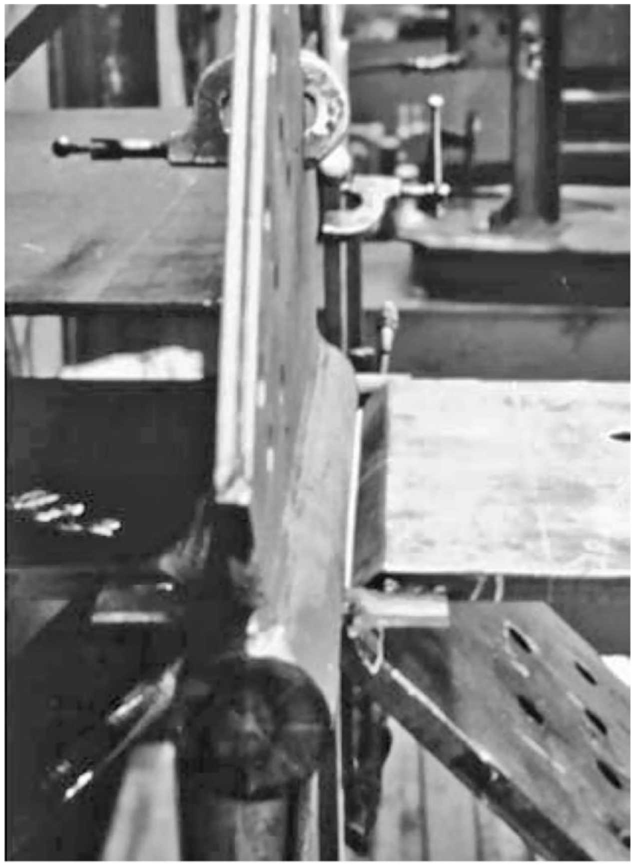


FIGURE 7.6 Space-frame connector in welding fixture—Carver-Hawkeye Arena.

FIGURE 7.7 Space-frame module 42 × 126 ft being lifted into place—Carver-Hawkeye Arena.

Space trusses can also be designed so they can be shop-welded into panels of a size that can be shipped, thereby reducing the number of field connections. Shipping limitations will normally limit these panels to about 15 ft deep and about the same width. This size will allow the shop to rotate the panel and position it for efficient welding (Fig. 7.8).

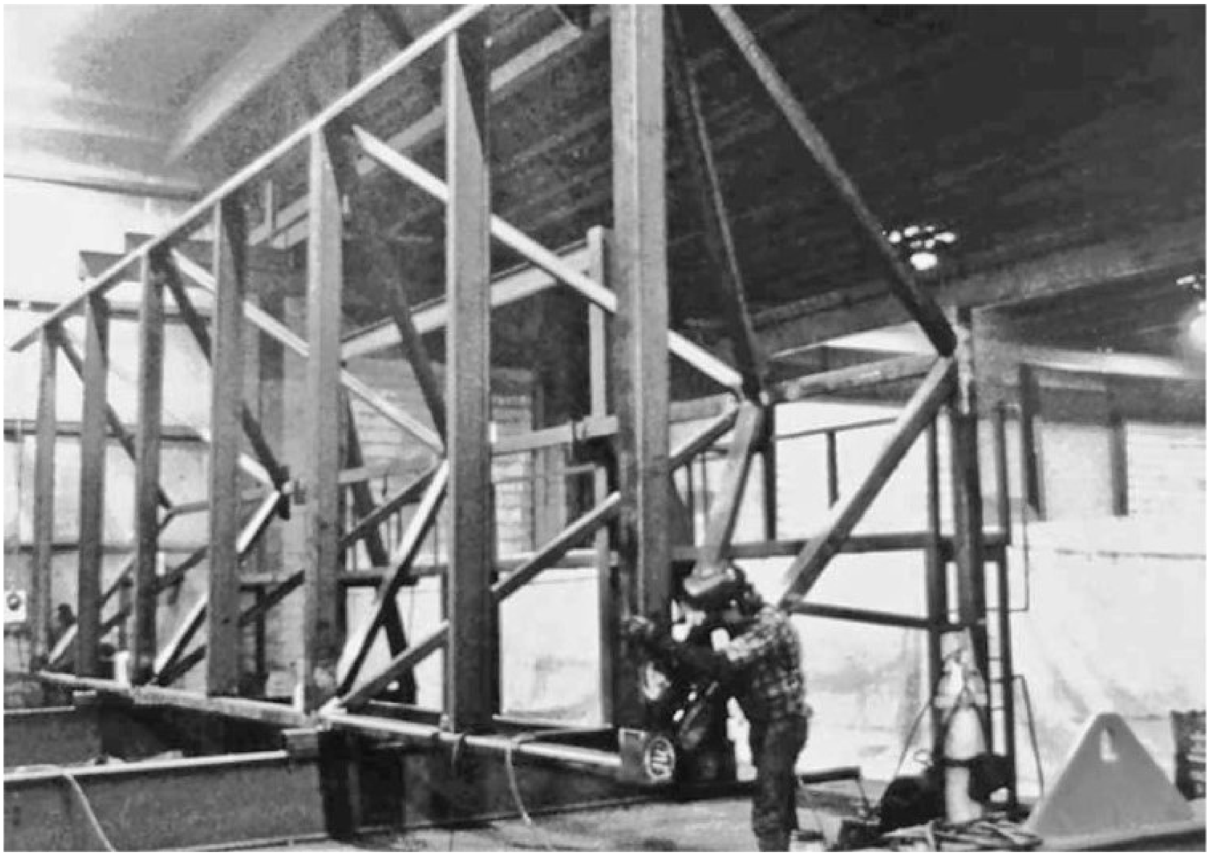


FIGURE 7.8 Space-frame module 15 × 60 ft rotated for shop welding—Minneapolis Convention Center, S.E. Skilling, Ward, Magnusson and Barkshire.

Hollow structural sections (HSS) are often used for truss members because of their appearance and axial-load capacity. Connections of direct-welded HSS require special design procedures. The connection limit state can be various modes of wall failure in addition to weld rupture due to stress concentrations. These stress concentrations are caused by the difference in the relative flexibility of the chord wall when compared to the axial stiffness of the web member. The chord wall thickness required for connections is an important factor when designing members. It may be necessary to increase wall thickness or insert a heavy section at the branch to transmit the design forces (Fig. 7.9).

FIGURE 7.9 Solid 6-in^2 reinforcement for HSS joint—Minneapolis Convention Center.

Welds for HSS-to-HSS connections should be sized to ensure adequate ductility to prevent rupture at design loads. This can be easily accomplished using the effective length concepts given in the *Specification for Structural Steel Buildings*, ChapterJ (ANSI/AISC 360-10)[1] or the AWS/*Structural Welding Code* (ANSI/AWS D1.1-2015).[2] A more conservative procedure would be to use a weld with an effective throat 1.1 times the wall thickness of the web or branch member. This is intended to make sure the wall of the web member will yield and redistribute stress before the weld ruptures. The ratio is based on E70XX electrodes and A500 grade B material. Direct-welded HSS connections of the T, Y, and K type should, where possible, utilize fillet or partial-penetration welds. Unbacked complete-joint-penetration welds that must be made from one side require special welder certification and are very difficult to make and inspect. Butt splices in HSS may require complete-joint-penetration welds. This type of weld should be made using steel backing to allow the use of standard weld procedures and welder certifications. For more detailed information on these connections, see Refs. 1 to 4.

7.5 EXAMPLES OF CONNECTIONS FOR SPECIAL STRUCTURES

Examples of connections developed for special structures can be helpful to illustrate the types of problems that are encountered and some idea of how connections can be adapted to meet special requirements.

The first project is a 42-story office building that uses a perimeter moment frame coupled with a braced core as the lateral load-resisting system. When a free-body diagram of the connection forces for the brace members in the core was prepared prior to developing connections, it was discovered that the axial loads for the horizontal struts given in the connection schedule were substantially less than the horizontal component of the brace diagonal. The EOR reviewed the lateral load analysis and discovered that when the structure was modeled, a stiffness factor was assigned to the floor diaphragm to provide for the interaction between the moment frame and the braced core. In the model the floor was carrying part of the brace force. While the brace loads may actually be transmitted in this manner, the EOR decided to follow conventional practice and size the steel for the full brace force rather than rely on this type of composite action. All of the horizontal struts were resized and connections were then developed for the full horizontal component of the diagonal force. Diagonal braces were wide-flange sections using claw angle-type connections with 1-in-diameter A325 SC bolts in oversize holes.

The second project is a sports arena using a skewed chord space truss supported on eight columns with the roof located at the bottom chord of the trusses (Fig. 7.10). Each type of connection was clearly shown on the design drawing along with the forces to be used to determine the number of bolts and welds required for each connection. While reviewing the forces given for the bottom chord, it was noted that the bottom chord members had been modeled as axially loaded pin-ended members with vertical end reactions due to the roof dead and live loads on the bottom chord. A check of the actual connection that consisted of a plate on each side of the web that was welded between the flanges of W27 sections indicated the connection was rigid. After reviewing this compatibility concern, the EOR decided to size the connections for the axial forces and vertical end reactions given using *N*-type values for all of the bolts. A check of the connections using *X*-type values for the bolts indicated adequate reserve strength for possible end moments.

FIGURE 7.10 Bottom chord connection for space truss—Carver-Hawkeye Arena.

The exposed top chord, diagonals, and connectors were all made of ASTM A588 material left unpainted so it could weather. While the fabricator was detailing these connections, the EOR became aware of a study[5] that indicated, under certain conditions where moisture had access to the inside of a joint, the expansive pressure of the continuing corrosion could overstress the bolts and lead to failure. Connection details were modified to make sure the recommendations on minimum plate thickness and maximum bolt spacing were complied with. Special restrictive fabrication tolerances were established for connection material flatness in order to ensure the connection bolts would be able to clamp the full surface together. The fabricator, by using techniques such as prebending plate prior to welding and using heat-straightening after welding, was able to eliminate almost all distortion due to shop welds (Fig. 7.11). The high-strength bolts were able to pull the plates together so there were no gaps in the connections (Fig. 7.12).

FIGURE 7.11 Heat straightening of connection plates after welding—Carver-Hawkeye Arena.

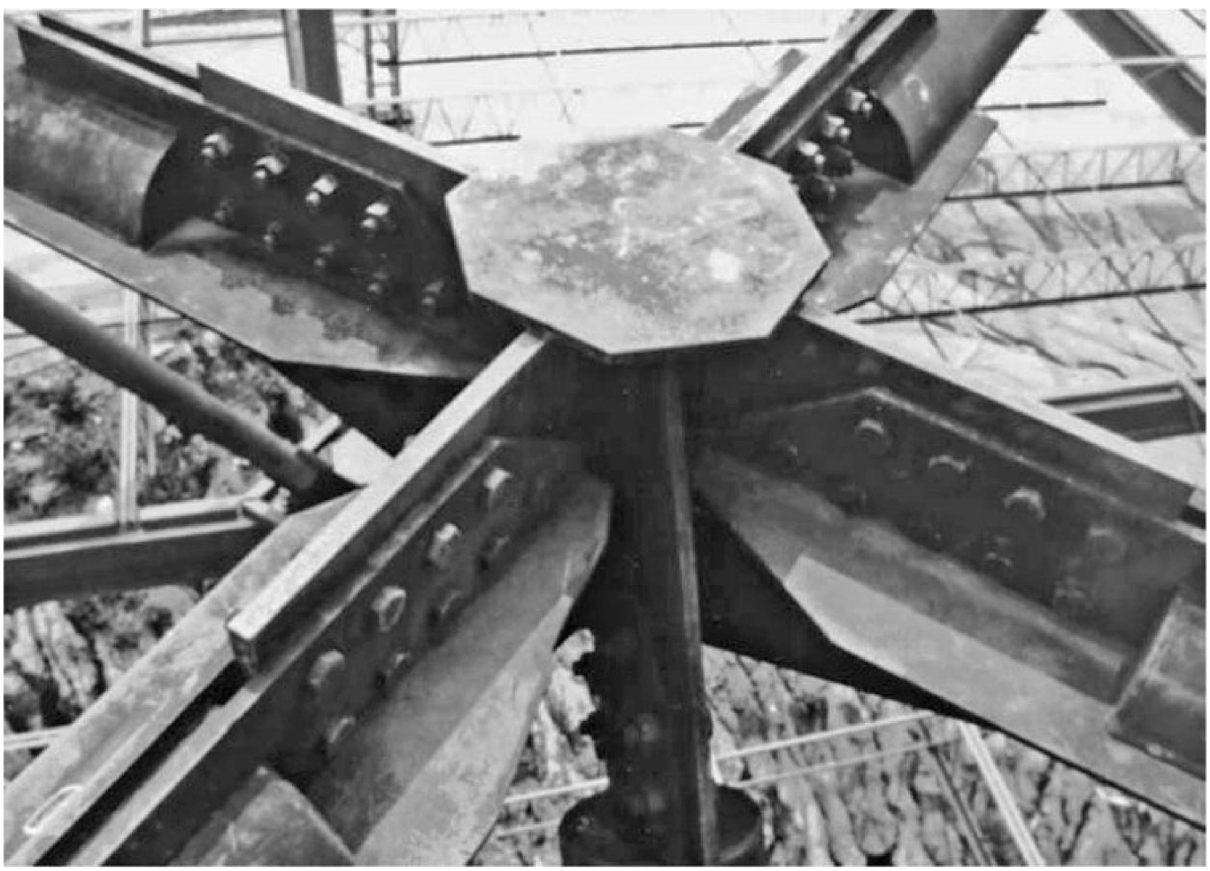


FIGURE 7.12 Exposed top chord connection showing fit-up after bolting—Carver-Hawkeye Arena.

The third project is a 57-story office building that uses a unique lateral load system. The wind in the longitudinal direction is resisted by five-story bands of vierendeel trusses that span 97 ft between concrete super columns. The vierendeel trusses were designed as horizontal tree girders with verticals spliced at midheight between floors (Fig. 7.13).

FIGURE 7.13 Vierendeel framing system—Norwest Financial Center, S.E. CBM Inc.

These splice connections were first designed with partial-joint-penetration field welds. The shop connections of these W24 verticals to the horizontal girder were complete-joint-penetration welds. The combination of weld shrinkage due to these shop welds along with the distortion of the girder due to welding and the rolling tolerance of the vertical section made it almost impossible to achieve the proper fit-up of the field-welded joint without a lot of expensive remedial work. Since the field splice was at the inflection point of the vertical, there were only axial loads and shears to be transmitted through the connection. It was decided to use an end-plate–type connection with slip-critical bolts in oversize holes to accommodate the fabrication and rolling tolerances. In addition, the members were detailed short and a 3/8-in shim pack was provided to bring each joint to the proper elevation. The modification of this connection was one of the keys to the early completion of the erection of this structure (Figs. 7.14 and 7.15).

FIGURE 7.14 Shimming of field splice of vierendeel verticals—Norwest Financial Center.

FIGURE 7.15 Splice of vierendeel verticals showing alignment—Norwest Financial Center.

The fourth project is a 37-story mixed-use structure that uses a megatruss bracing system for wind loads. The bracing truss has nodes at five-story intervals and uses wide-flange members for stiffness. The connections of the truss at the nodes were designed as partial-joint-penetration groove-welded butt splices. Because of past experience with poor fit-up the EOR specified that joint fit-up had to comply with AWS D1.1 prequalified joint requirements with no build-out permitted.

The combination of mill, fabrication, and erection tolerances would have made it impossible to achieve this type of fit on these heavy W14 members. It was decided to add a field splice in all of the diagonals midway between nodes using a lap-plate–type splice. This allowed the erector to position the lower half tight to the node and then jack the upper half tight to the upper node. The brace members had all been sized for axial stiffness and the design forces were typically less than half of the member capacity. The lap plates were designed and fillet welded for the actual brace force (Figs. 7.16 and 7.17).

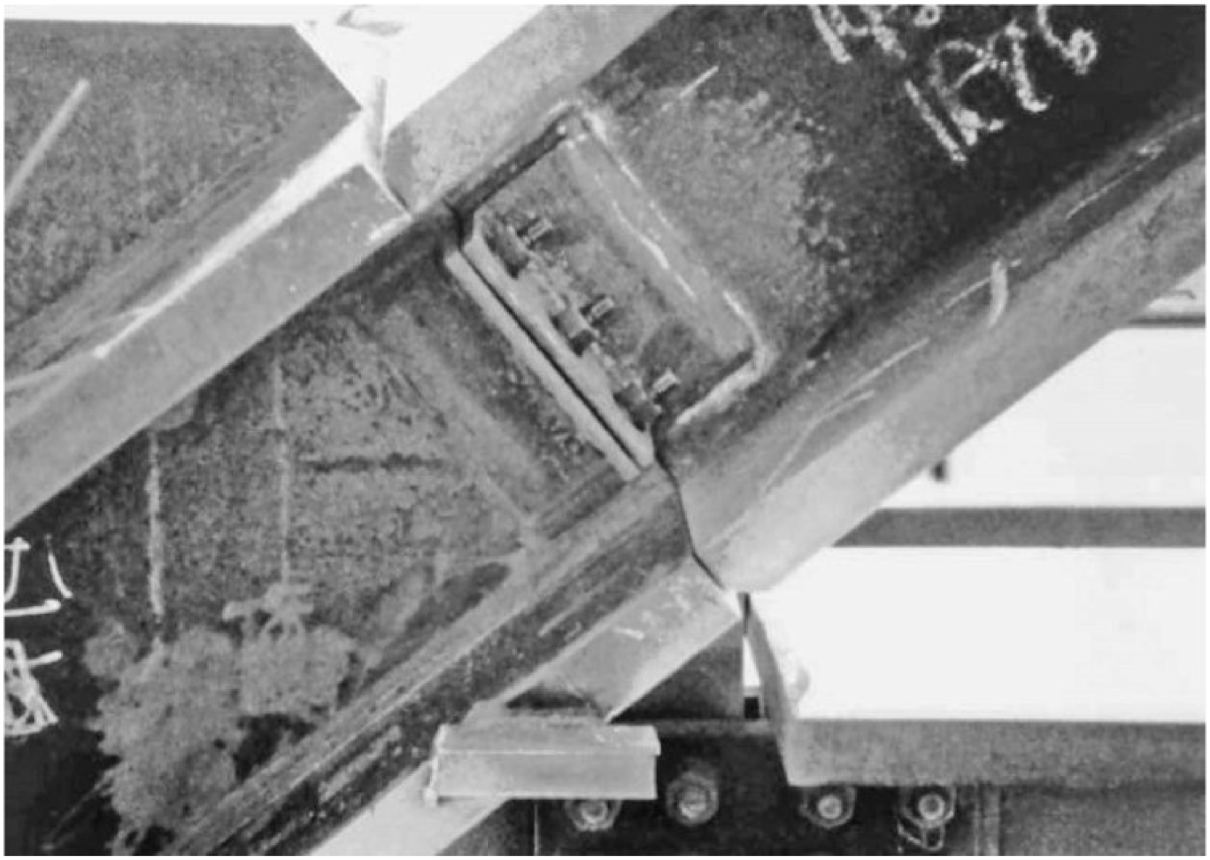


FIGURE 7.16 Bracing node connection showing fit-up—Plaza Seven, S.E. CBM Inc.

FIGURE 7.17 Adjustable midheight splice of bracing diagonal—Plaza Seven.

The fifth project is an exhibition hall consisting of three lamella domes 210 ft in diameter surrounded by a 60-ft-wide delta-type space truss made of hollow structural sections (Fig. 7.18). Each dome is supported by a series of sloping pipe struts from four columns. The domes vertically support the inside of the space truss and the space truss laterally constrains the domes (Fig. 7.19). The total structure is approximately 900 ft long without an expansion joint. The EOR laid out the space truss so modular units could be shop-fabricated in units 15 ft wide and 60 ft long. The top and bottom chords of these units were offset and oriented in the 60-ft direction to minimize splicing (Fig. 7.20). Each unit had two top chords and one bottom chord. This resulted in double top chords at the splice between units. These chords were connected by flare V-groove field welds at the panel points. The bottom chords were detailed with a short connector stub to which a section of cross-chord was butt-welded in the field.

FIGURE 7.18 Lamella dome and delta space frame—Minneapolis Convention Center.

FIGURE 7.19 Dome and space-frame column and pipe supports—Minneapolis Convention Center.

FIGURE 7.20 Top-chord field splice of delta space frame—Minneapolis Convention Center.

The diagonals of the delta truss intersect the bottom chords at 45° to the vertical and the chords. These members were typically 6-in^2 HSS and would have overlapped at the panel point. To avoid this the EOR detailed a connector consisting of intersecting vertical plates on top of each chord. Initially it was planned to provide complete-joint-penetration welds for these diagonal connections. However, when weld procedures were developed, it became apparent that the restricted access to these joints would make both welding and inspection very difficult. The connection was redesigned using partial-joint-penetration and fillet welds sized for the actual loads in the members with allowances as required for uneven load distribution (Figs. 7.21 and 7.22).

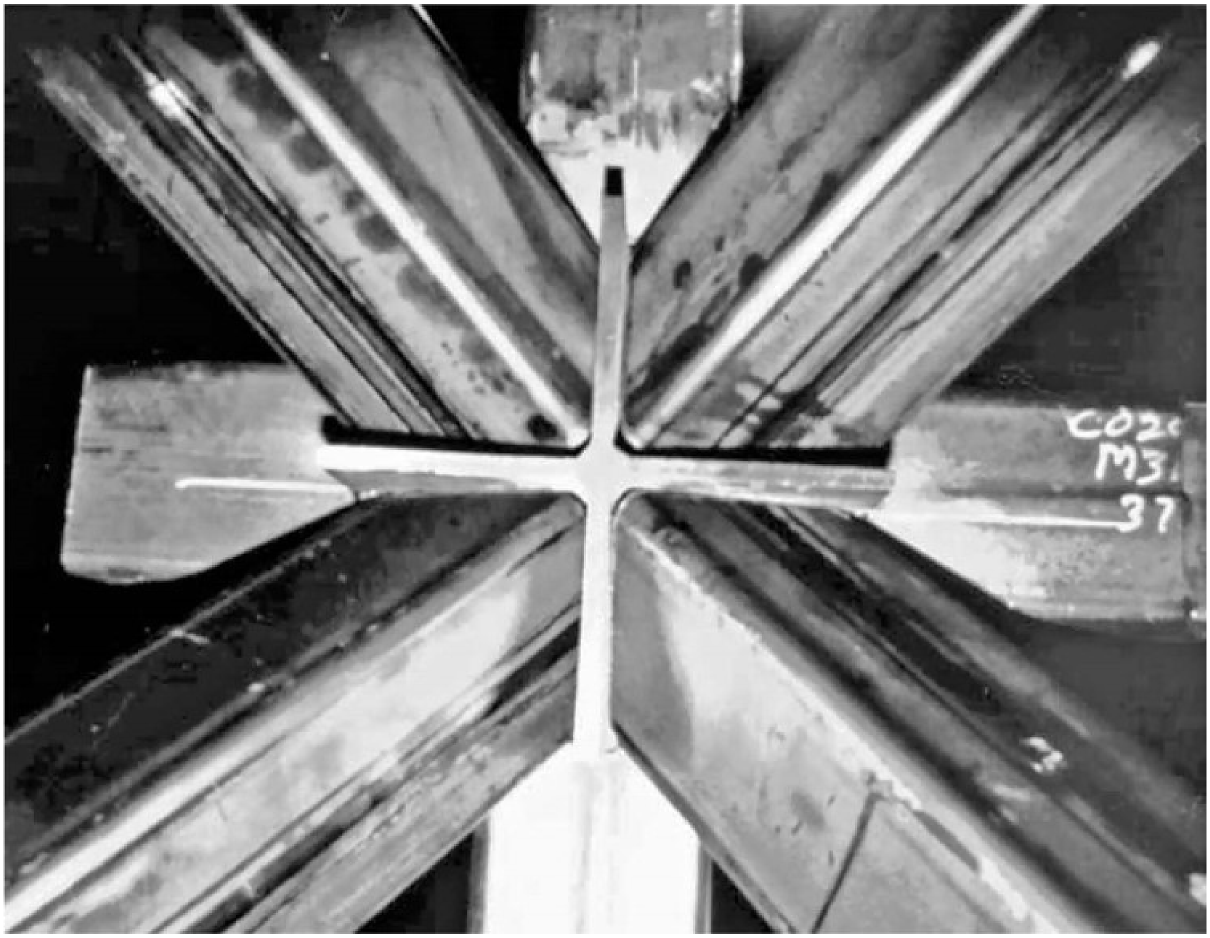


FIGURE 7.21 Space-frame bottom-chord connection showing fit-up—Minneapolis Convention Center.

FIGURE 7.22 Space-frame bottom-chord showing weld—Minneapolis Convention Center.

All of the butt splices in the chord were detailed as complete-joint-penetration welds using internal steel backing so a standard V-groove weld could be used (Fig. 7.23).

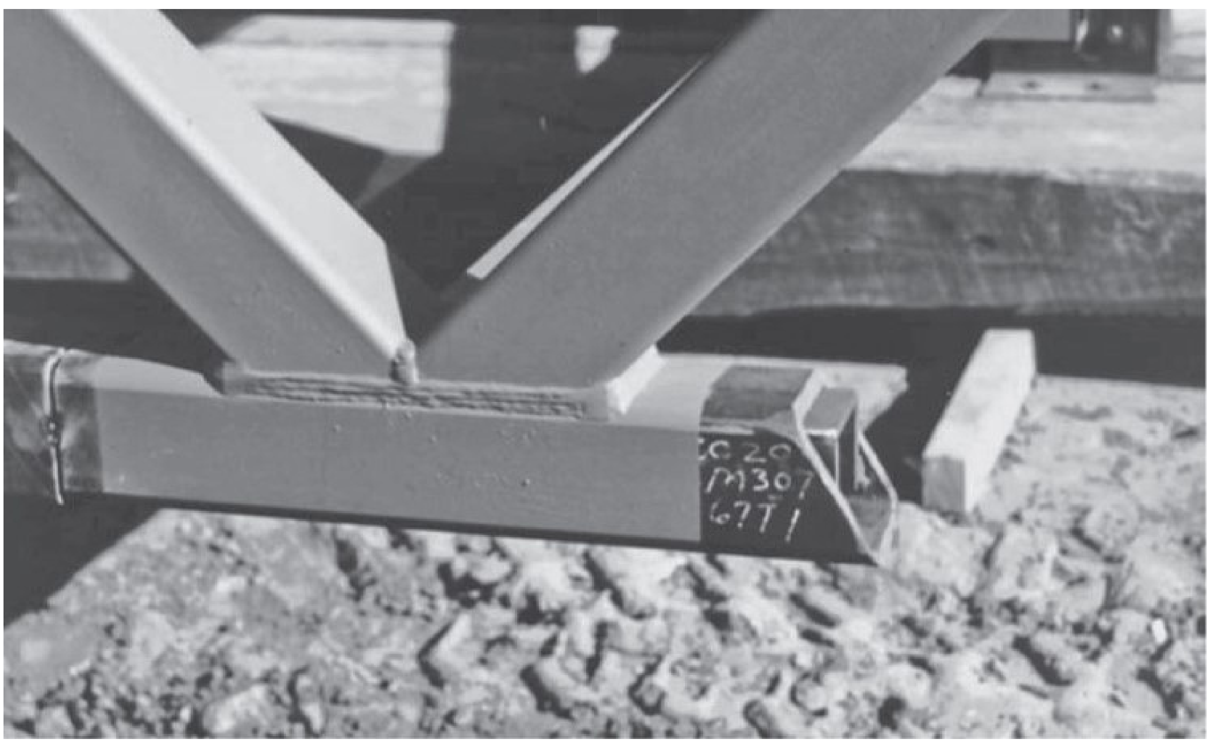


FIGURE 7.23 Space-frame bottom-chord splice connection—Minneapolis Convention Center.

The connection of the sloping pipes to bottom ring of the dome consisted of a series of radial plates that were complete-joint-penetration welded to a 6-in-thick connector plate on the ring (Fig. 7.24). The EOR was concerned about possible brittle fracture of these heavy welded plate connections and specified material ductility using standard Charpy V-notch testing. When orders were placed, the material supplier informed the fabricator that the standard longitudinal Charpy test would not measure the through-thickness properties needed to accommodate the weld strains. The design and construction team consulted with metallurgists and fracture mechanics experts to develop a specification and testing procedure that would ensure adequate through-thickness ductility. The testing procedure called for through-thickness samples to be taken near the center of the plate. A minimum through-thickness reduction in area of 20%, along with a minimum Charpy value of 15 ft-lb at 70°F in all three axes, was specified. While the through-thickness Charpy test is not a reliable indicator of ductility, it was decided to do this test as a general comparison with the properties in the other two directions. The producer supplied a low-sulfur, vacuum-degassed, and normalized material with inclusion shape control. All material was 100 percent ultrasonically inspected at the mill. There were no through-thickness problems due to welding strains. Since this project was built, several mills have developed proprietary low-sulfur materials with excellent through-thickness properties and ASTM now has a specification, A770, for through-thickness testing.

FIGURE 7.24 Gusset-plate connections for pipe struts to dome—Minneapolis Convention Center.

The lamella domes were designed using wide-flange shapes shop-welded into diamond patterns (Fig. 7.25). Since the fabricator was nearly even, the 24-ft-wide diamonds at bottom ring were shop-fabricated and delivered to the site. A bolted web splice was provided between diamonds and for the ring beams to the diamonds. This provided both an erection splice and was adequate for any out-of-plane loads. The flanges of these members were complete-joint-penetration welded.

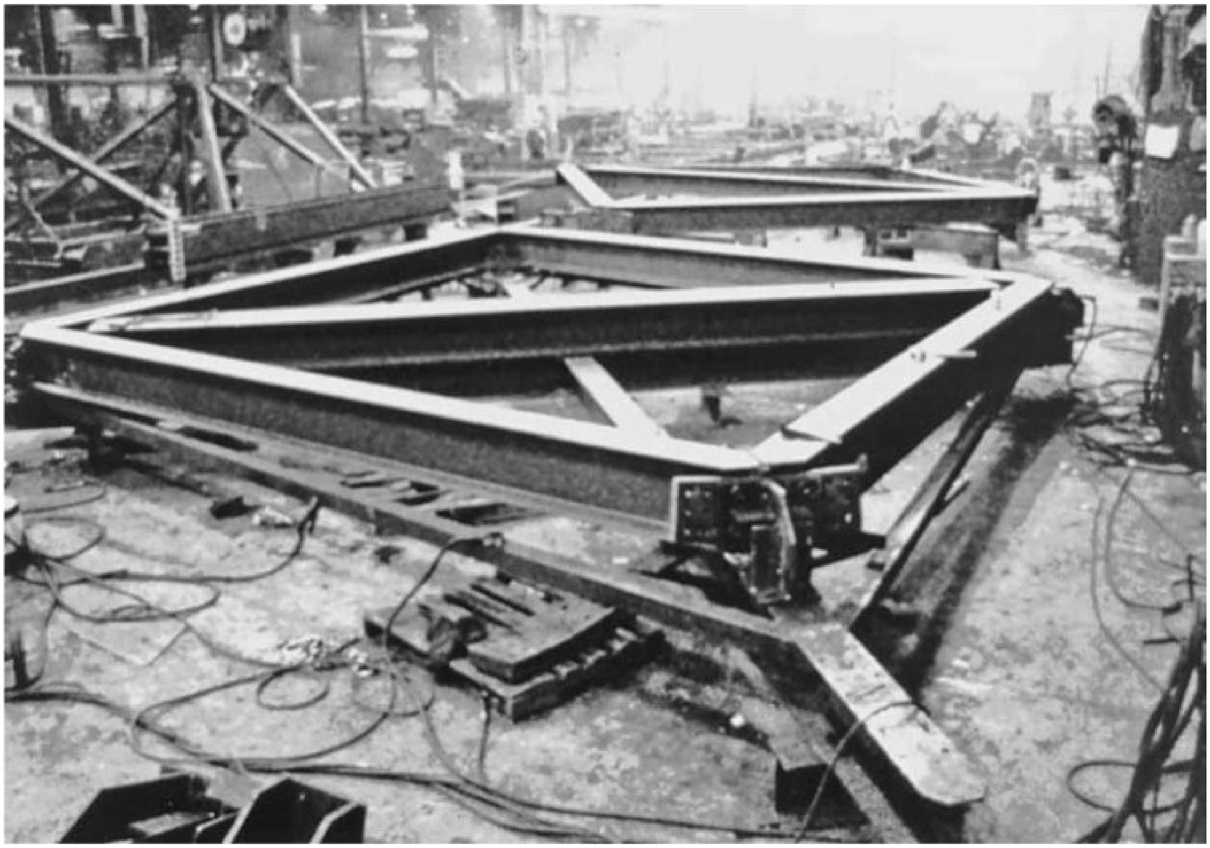


FIGURE 7.25 Lamella dome module in fabrication—Minneapolis Convention Center.

The entire space frame project was ground-assembled. The space trusses were assembled in units 60 × 75 ft and hoisted by crane onto shoring towers and perimeter columns. The dome was assembled ring by ring on the ground using shores as required (Fig. 7.26). The dome assembly, including the deck, was then jacked into place (Fig. 7.27). When the slotted pipe supports were slipped over and welded to the gusset plates on the heavy weldments described here, a new concern arose. The misalignment of the gusset plates due to the angular distortion caused by the one-sided groove welds along with the erection tolerances of the structure resulted in some bowing of the connection plates. The EOR reviewed the forces and added stiffeners, where required, to prevent buckling due to any misalignment of the plates in compression.

FIGURE 7.26 Start of ground assembly of dome—Minneapolis Convention Center.

FIGURE 7.27 Jacking rods in position for lifting dome—Minneapolis Convention Center.

The sixth project is a multiuse sports and events center. The roof framing consists of 26-ft-deep trusses spanning 206 ft that are framed at one end to a jack truss spanning 185 ft. All of the truss members are W14 sections oriented with the flanges in the vertical plane and connected with lap-type gusset plates on each flange. All of the typical truss connections used slip-critical high-strength bolts in oversized holes. Chords were spliced at panel points and W16 sections were used for web members as recommended previously.

The connection of the 206-ft trusses to the jack truss presented a special problem because of the large reaction that had to be carried by the framing angles (Fig. 7.28). Originally it was planned to intersect the work points of the end connection at the center of the jack truss top chord. A free-body diagram of the connection, however, showed the bolts would have to develop a moment of 465 ft kips in addition to carrying a shear of 450 kips. Even if it was possible to get enough bolts in the plate, the angles could not develop the moment. By moving the work point to the face of the truss it was possible to eliminate the bending in the outstanding leg of the angle and to reduce the eccentricity to the bolt group in the other leg to 4.5 in. The eccentric reaction on the jack truss was easily balanced by adding a 14-kip axial connection at the bottom chord. The bending stress in the jack truss vertical was checked and found to be acceptable.

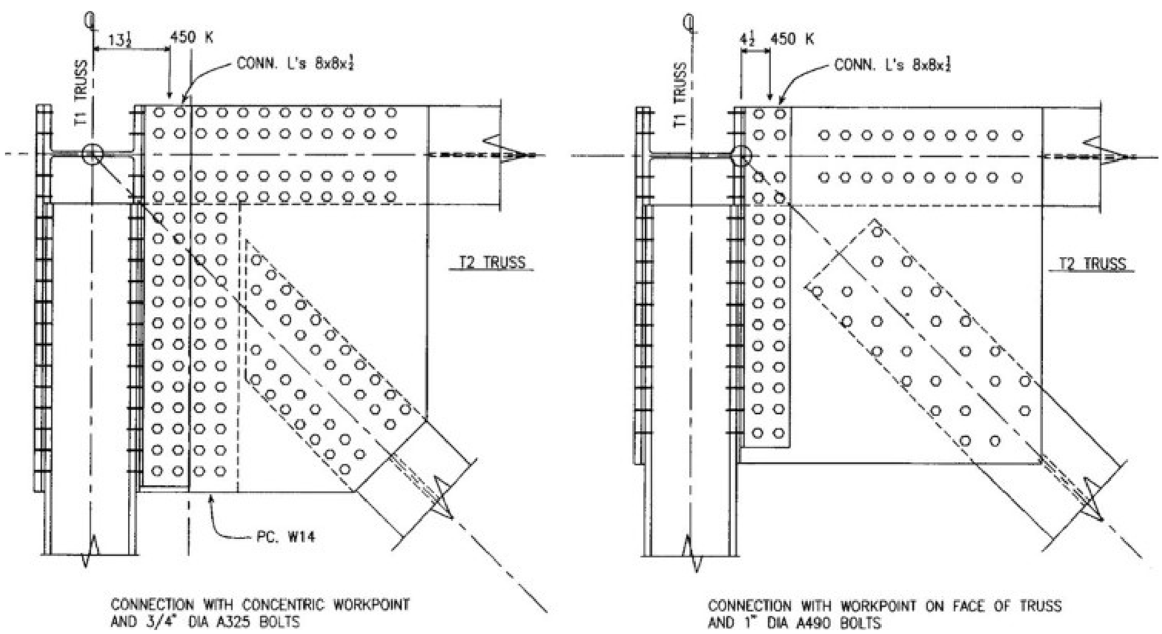


FIGURE 7.28 Connection roof truss to jack truss—Mankato Civic Center.

7.6 BUILDING INFORMATION MODEL

The acronym, BIM (Building Information Model), describes what has become a paradigm shift in the method of designing and constructing complex steel structures that continues to evolve as this is written. Historically the design and construction of a building has relied upon drawings and specifications as described in Sec. 7.1 to define a building. Multiple views were required to depict an object often along with some type of written description of the size and other requirements. This process also requires each contractor to interpret the data and reproduce much of the same information before adding their product. Coordination between various systems such as concrete, structural steel, precast, and mechanical systems required time and a great deal of effort and was subject to error.

A BIM differs from a two-dimensional (2D) and some three-dimensional (3D) CAD models, which are typically nothing more than electronic drawings. Instead of representing members as lines with labels, a BIM is an intelligent model that consists of a series of objects with their geometry and attributes. These objects or building elements can be displayed in multiple views, as well as having their nongraphic attributes assigned to them. If the particular BIM software has interoperability with other programs, the various design and construction team members with the help of other softwares can use the BIM information to develop their own model and their work can be input back to the BIM. This provides for ease and accuracy in coordinating the design and construction of the various building elements.

There are several levels or versions of BIM. When it involves steel fabrication and erection. Mark Howland, chief engineer of Paxton Vierling Steel uses the terms “Big BIM” and “Little BIM.” *Big BIM* is where the design team models the structure and uses it for their design needs and then passes it to the entire construction team for their use and input. This method is most cost-effective from the overall project standpoint. It allows the designer to better visualize each of the elements and avoid clashes between various systems. Doing this up front in the design stage saves both time and money. Design firms have expressed concern about the cost and possible legal liability for providing this information. Owners need to realize there is a substantial value in providing this service up front and should compensate the designers accordingly. *Little BIM* is where the design team provides 2D drawings or CAD files and one member of the construction team, typically the steel fabricator prepares a BIM model for steel fabrication and erection. This model may in turn be used by other members of the construction team to coordinate their work.

Two examples where the model was developed by the design team are the Adaptive Reuse of Soldier Field in Chicago, Illinois, and the New York Mets Stadium—Citi Field in Queens, New York.

The Soldier Field project involved an existing stadium built in the 1920s that featured classical colonnades designed by architect Holabird and Roche as a memorial to the American Soldier. Because this was an existing facility currently in use, the construction schedule was limited to 20 months. Thornton-Tomasetti, the structural engineer, elected to perform the structural analysis for the main frame using SAP2000 and RAM for the floor framing. Thornton-Tomasetti in joint decision with the project team proceeded to model the project in Tekla Corporation’s Xsteel 3D-modeling software. The 3D models were generated for each of the four quadrants and documentation was added showing beams sizes, forces, and camber along with column and brace information.

The steel fabricator, Hirchfeld Steel Co. received the 3D models and used them for connection-design information and preparation of shop drawings. This model with the connection information added was then submitted for review by Thornton-Tomasetti. Because the review process only required the examination of the connections the review took 5 days instead of the usual 10 days and saved valuable schedule time. After the review process Hirchfeld Steel Co. used the information in the model to prepare computer numeric control (CNC) instructions for download to the machines to cut, punch, and drill material along with the preparation of 2D shop drawings. The model was also used to coordinate the detailing and erection of the secondary framing for the complex cladding system. A more complete description of the project can be in seen in Ref. 6.

The New York Mets Stadium—Citi Field is another example where the design team prepared the BIM for use by the construction team. WSP Cantor Seinuk decided to use AutoDesk’s Revit software for this project. The project architect, HOK Sport, took full advantage of the 3D-modeling features of Revit along with its compatibility with other AutoDesk products. The architectural and structural models were combined to provide coordination of the complex geometry and construction features. At this stage a decision was made to convert the Revit BIM model to Tekla Corporation’s Xsteel to facilitate the steel-detailing process. As a result of an AISC initiative several years ago the industry established a digital standard for electronic communication, CIMsteel, Integration Standards/Version 2 (CIS/2). This provided interoperability between these software systems. What was unusual for this project was that the Xsteel program was furnished to the bidders to save them time and expense in bidding the project. The final Xsteel model was furnished to the selected steel supplier thereby saving considerable time and expense in preparing shop drawings and avoided numerous Requests for Information (RFIs). For a paper covering this project along with more information on BIM practices, see Ref. 7.

LeJeune Steel Company has fabricated projects with what could be called Little BIMs with mixed results. The major problem with developing a BIM using 2D drawings is that depending on the accuracy of the data furnished by the contract documents it can be very time consuming often requiring numerous RFIs. While clashes or coordination issues are discovered before construction starts they still cause delays and added cost to resolve at this stage. A BIM developed for a project from conventional 2D drawings for the building structure and an AutoCad wire frame model for the facade framing found clashes between the facade framing and a number of building elements. While it was important to find these problems in the detailing stage it would have saved time and money to have found and solved the problems in the design stage.

The new University of Minnesota Football Stadium—TCF Field—utilized a somewhat different process to produce a BIM. The general contractor, M. A. Mortenson (MAM), worked with the design team to develop a model of the stadium geometry with all of the control points and their elevations using Nevis Works software. They supplied this information to the steel fabricator, LeJeune Steel Co. (LSC). LSC and its detailer, LTC Consultants, then built a Xsteel model with all of the structural steel members and added all of the connection details (Fig. 7.29).

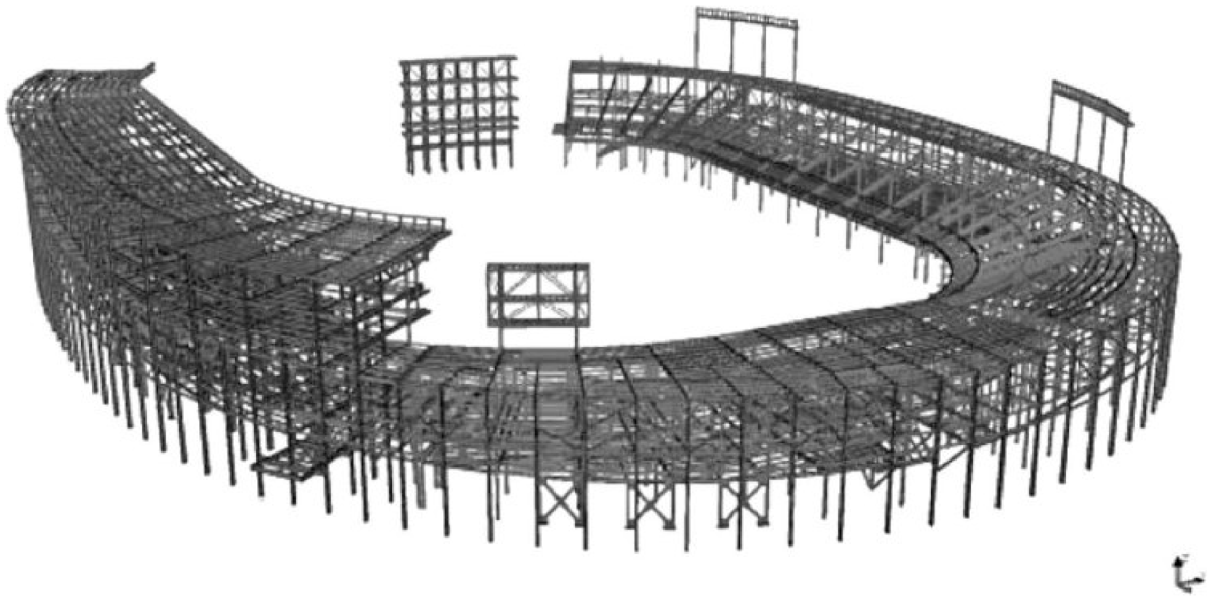


FIGURE 7.29 Model of the structural steel for the University of Minnesota, TCF Stadium, S. E. Magnusson—Klemencic Associates.

The interoperability between the two systems allowed all of the information to be transferred seamlessly. The interoperability also allowed LSC to subcontract a portion of the structural steel to American Structural Metals (ASM) who used the MAM information to build a Design Data SDS-2 model. LTC was able to import this information to check coordination between the steel packages and provide a full model of all of the steel to MAM.

MAM also built a model of the precast seating including the connection points to the steel. LSC incorporated this information into the Xsteel model and used it to determine the location of all of the seat support points on the steel seating rakers (Figs. 7.30 and 7.31).

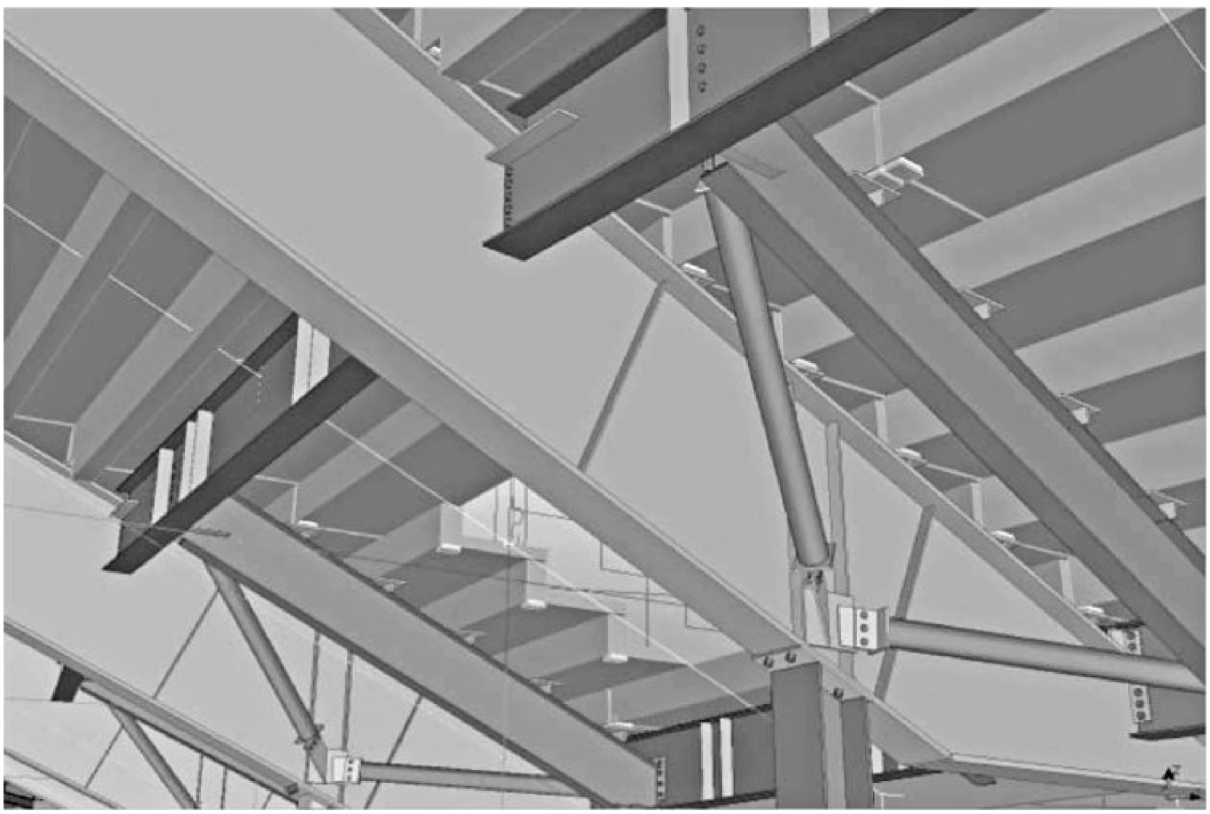


FIGURE 7.30 Model of steel rakers with precast seating—University of Minnesota, TCF Stadium.

FIGURE 7.31 Steel raker with precast seating erected—University of Minnesota, TCF Stadium.

This not only saved time, it ensured that even with the complex radial geometry, the two materials would fit together without a problem. Initially there were some clashes but with all of the information available in one model, the problems were easily resolved in a few working sessions.

MAM also used the BIM to help with scheduling access to critical areas. There were areas under the cantilevered skyboxes where the precast erector and the steel erector had to take turns. The BIM allow the trades to see exactly what the access would be at various points in the construction process.

MAM also used the Xsteel model to coordinate the layout of the MEP duct work through the numerous brace elevations (Fig. 7.32). This prevented clashes with the braces and the large gusset plates needed for their structural connections. Other subcontractors such as the architectural precast contractor and even the window-washing equipment contractor used the model in laying out their work.

FIGURE 7.32 Model showing bracing members and connections—University of Minnesota, TCF Stadium.

A more recent example of what can be accomplished by the combination of a BIM and the accuracy of the new hole making equipment is the retrofit of the three span Washington Avenue bridge over the Mississippi River in Minneapolis for light rail traffic.

The retrofit involved fabricating four 3-span continuous trusses to be installed between four existing continuous plate girder spans shown in Fig. 7.33. The trusses were designed by AECOM, the EOR with box chords and wide flange diagonals connected with lap plate gussets as shown in Fig. 7.34. A BIM model was built from the standard 2D drawings. Project specifications called for standard holes for all bolts to be either drilled with members assembled or predrilled using CNC equipment that could demonstrate the required accuracy. All of the members were drilled prior to assembly. Because this bridge had to be assembled in the air over the river each truss was shop assembled after being drilled to verify the accuracy. When a rare fit-up issue occurred in the assembly such as shown in Fig. 7.35, it found there was an inaccuracy in the geometry of the laydown that was corrected by moving the assembly so the bolts fit.

FIGURE 7.33 Two of four continuous trusses in place—Washington Ave Bridge, Minneapolis, MN.

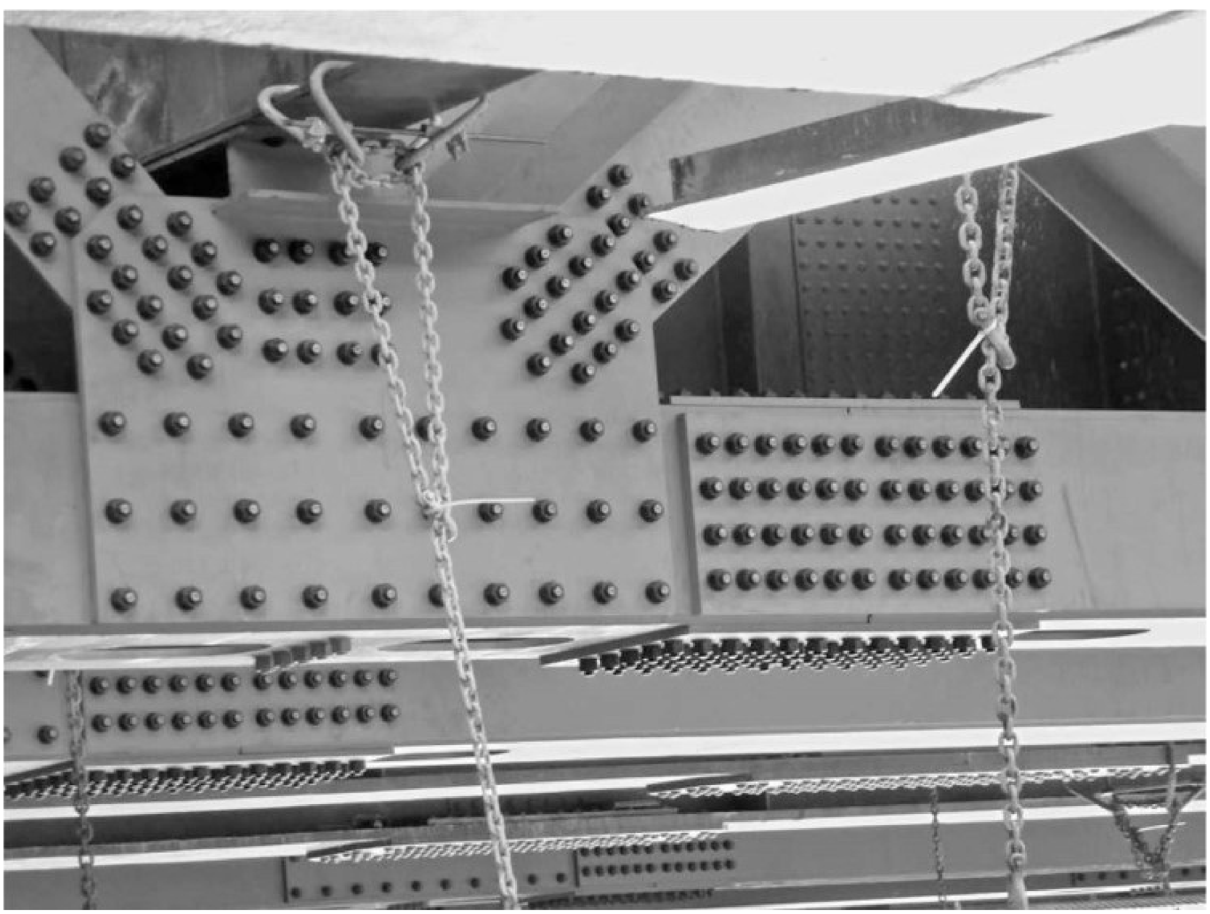


FIGURE 7.34 Typical bottom chord splice fully bolted—Washington Ave Bridge, Minneapolis, MN.

FIGURE 7.35 Truss section in shop assembly—Washington Ave Bridge, Minneapolis, MN, AECOM S.E.

Two major design-build projects, Deep Space for EPIC Software and U.S. Bank Stadium for the MN Vikings have utilized a cloud based model that was shared by the project team who worked together with Thornton Tomasetti, the EOR, to develop a fully connected model.

The EOR provided a structural model with all of the members and the required geometry. The fabricator, erector, and general contractor developed fabrication and erection sequences and schedules based on the overall project schedule and these were entered into the model.

The fabricator was able to establish basic parameters for typical connections along with locations for splices so advance mill order lengths could be extracted from the model. The mill order references were retained in the model for use by the fabricator when preparing shop cutting instructions along with the basic detailing information like major piece marks.

Using the fabricators recommended connections concepts the EOR provided connection design by reference in the model to detail sketches and connection standards. Regular Internet Go-To meetings for the team provided the opportunity to review the design and allowed the team exchange information and solve any constructability problems.

When the design on each unit was complete it was released for detailing to the fabricator to finalize the details by adding weld details, checking bolt clearances and preparing shop drawings with the necessary information for machine instructions for the shop. The shop drawings were submitted electronically and with all of the connection information readily available in the model the EOR could quickly review and approve the shop drawings.

The Deep Space project with J.P. Cullen as the general contractor and erector was a radial truss layout that was assembled at grade and the mechanical, electrical, lighting systems and catwalks all installed before lifting the almost 5000-ton assembly (Figs. 7.36 and 7.37). The BIM made it possible to monitor all the critical layout points while at grade and while lifting to make sure the entire structure was performing as designed and would fit to the support structure that was already in place. (*See Ref. 8 for narrated slide presentation of project*.)

FIGURE 7.36 Long span roof with jacking strands connected and ready for lift—Deep Space, Epic Software, Verona, WI, Thornton Tomasetti S.E. Photo: Vakaris Renetskis/Thornton Tomasetti.

FIGURE 7.37 Roof in over lift position ready for erecting transfer truss supporting columns—Deep Space—Epic Software,

Verona, WI. Photo: Vakaris Renetskis/Thornton Tomasetti.

The U.S. Bank Stadium for the MN Viking football team is a long span roof truss roof structure shown in Fig. 7.38 that is over 250 ft above grade at each joint with geometry and access problems that would not have been possible to solve without a BIM. The project team of Thornton Tomasetti the SE, M.A. Mortenson the general contractor, DCCI the erector, LSC the detailer, and LeJeune Steel the fabricator were able develop connections and member assemblies that would be fabricated and safely erected on a very critical schedule. It was important to both shop fabricate and ground assemble as large a section as possible. The BIM was used to check shipping clearances for shop assemblies during the connection design and detailing to permit the complex queen post-truss boxes to be shipped in 60 to 80 ft lengths fully painted and ready to be combined into units over 300 ft long. The large box truss for the ridge that was more than 50 ft deep and 990 ft long was ground assembled in sections weighing up to 600 tons. The BIM was used to provide check dimensions to verify the ground assembled units would fit when hoisted in the air.

FIGURE 7.38 Long span roof structure showing sloping ridge truss with skewed queen post trusses spanning to concrete ring beam—U.S. Bank Stadium, Minneapolis, MN Thornton Tomasetti S.E.

A BIM is really the only practical way to design, detail, fabricate, and erect complex structures like those described here. During the design stage the BIM not only allows the designer to check for clashes between various materials it also allows for an interactive viewing of each connection node to check for clearances needed to fabricate and erect the complex connections. The 3D node shown in Fig. 7.39 can be viewed through 360° rotation, both horizontally and vertically, along with section cuts anywhere on the assembly. The section cut shown in Fig. 7.40 shows the arrangements of the bolts at the connection and can be used to determine if the erection clearances are adequate to stick and tension the high-strength bolts. The detail can be viewed over the Internet simultaneously by the structural engineer, fabricator, and steel detailer and any required modifications can be agreed upon. It is only necessary then to provide either an electronic or paper copy of the final design for record.

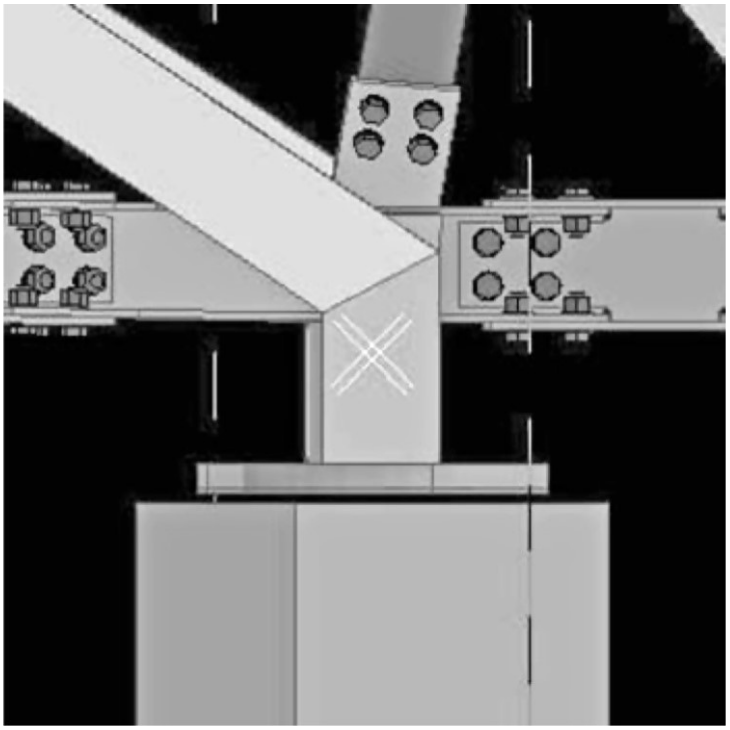


FIGURE 7.39 Front view of support 3D node—Millennium Park, Chicago, Illinois, S.E. SOM.

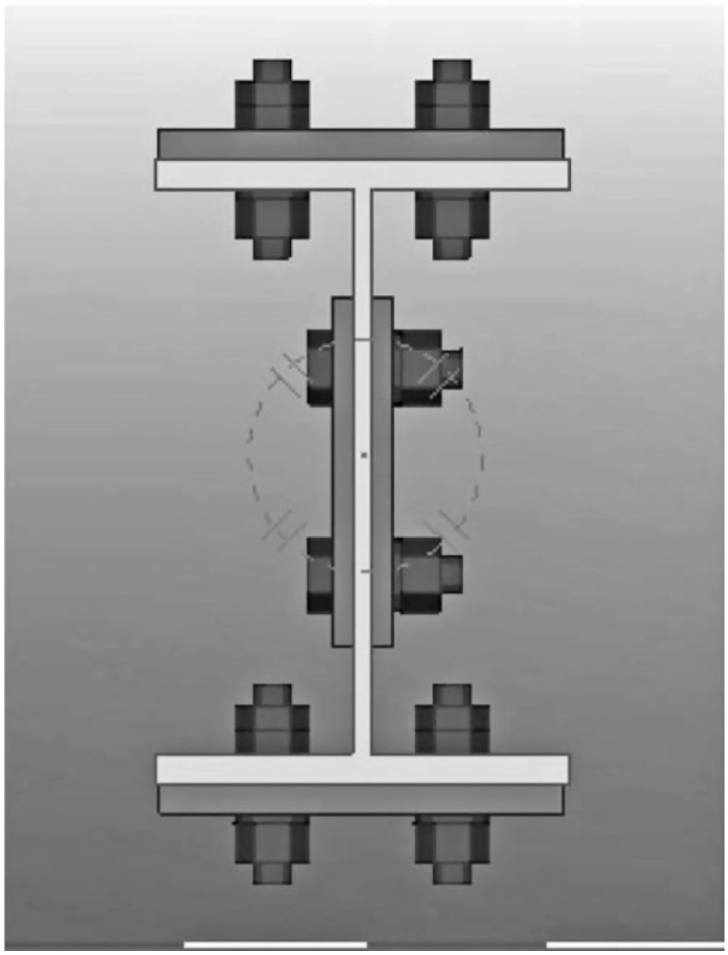


FIGURE 7.40 Section at support node—Millennium Park, Chicago, Illinois.

Most fabrication shops still require 2D shop-detail drawings to fit and weld the individual shipping pieces. Transferring all of the complex geometry can really only be done efficiently and accurately by detailing software. The rib member shown in Fig. 7.41 illustrates some of the detail drawing complexity that is possible using a 3D model.

FIGURE 7.41 Rib connection detail—Millennium Park, Chicago, Illinois.

The ability to download all of the geometry from the model to the CNC machines that the fabricator uses to cut, punch, and drill all of the detail pieces not only provides significant cost and time savings, it provides a level of accuracy that greatly improves overall fabrication and erection accuracy. Experience has shown that fit of the detail parts often controls the accuracy of the shipping piece. When assembling a complex member if the parts do not fit the reason is not the accuracy of the parts but the alignment of the main pieces. When the alignment is adjusted so that the parts fit, the member typically will meet all planned dimensions.

The information from the BIM can also be used to build fixtures for complex assemblies and to design shoring for field erection. The model will provide coordinates for any point on the surface of the assembly along with the relationship to any other point needed to control overall fit. A fixture for shop assembly of a large erection subassembly is shown in Fig. 7.42. The shoring for the erection subassemblies shown in Fig. 7.43 was designed using the geometry from the BIM.

FIGURE 7.42 Fixture for shop check of subassembly—Millennium Park, Chicago, Illinois.

FIGURE 7.43 Subassembly shoring—Millennium Park, Chicago, Illinois.

The BIM is often used in the erection process to verify access and plan the work sequence. For projects with free-form geometry like the Millennium Park project shown in Fig. 7.44 the correct positioning of each of the nodes can only be determined by using the coordinate geometry from the BIM. Coordinates are first determined for a series of targets placed on the surface of the member near the nodes. This information is then downloaded to a total station–

type surveying instrument. The total station with the target coordinate information is then used to verify the location, in space, of each node as a crane lifts the subassembly into place.

FIGURE 7.44 Using 3D coordinate system to locate steel—Millennium Park, Chicago, Illinois.

7.7 CONCLUSION

It is helpful when designing special connections to start with a freebody diagram of the connection. The free body should usually be cut at the connection face and all forces shown (see Fig. 7.45). While it may be necessary to use advanced techniques, such as finite-element analysis or yield-line theory to evaluate stiffness of elements, it is important to first try to establish the best force path. Care should be taken to make sure this is a complete path. Both sides of the connection must be able to transmit the force. An evaluation should be made for any connection eccentricity. It may be better to design the member for the eccentricity instead of the connection. A check should also be made for the flexibility of the connection if it was modeled as a pin in the analysis. It may be possible to ignore connection fixity in axially loaded members, such as trusses, as long as the members are modeled with flexible connections and all loads are applied at panel points. Preliminary connection design should be done prior to final member selection. It is impossible to effectively size members without taking into account connection requirements. Because of constructability and economic concerns, the design of special connections will almost always require input from the fabricator and erector. The EOR must either obtain this input in advance or be prepared to evaluate proposed means and methods modifications during the construction stage.

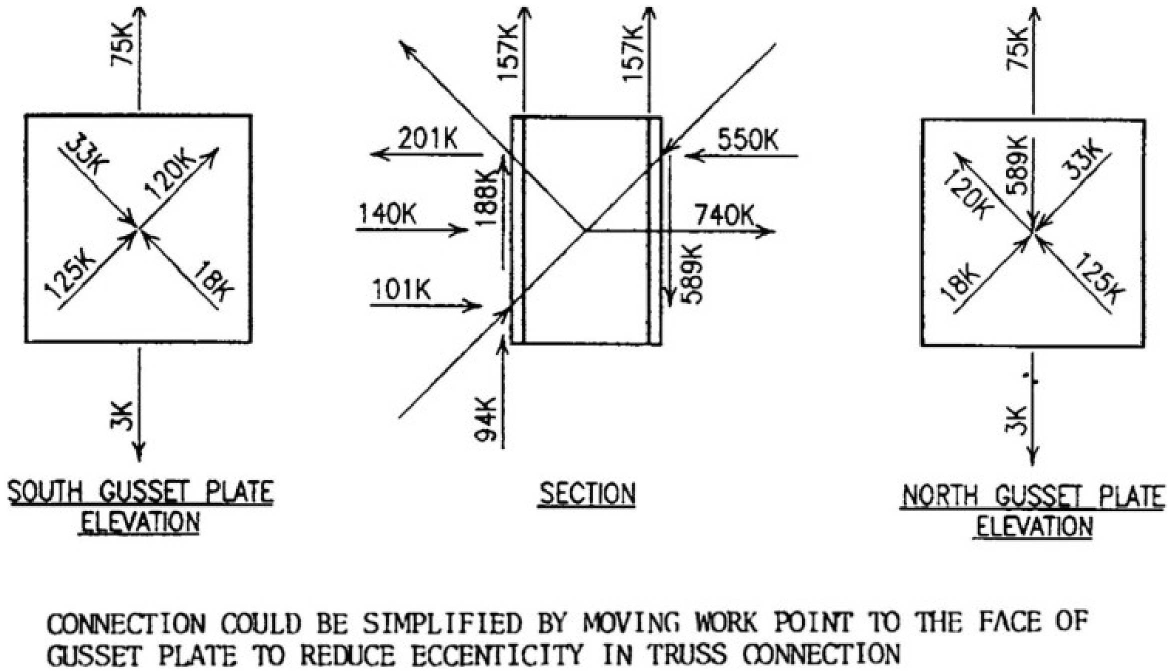


FIGURE 7.45 Free-body diagram of roof truss to box truss—NWA Hangar.

The increasing geometric complexity of special structures especially those with free-form geometry will require the use of new 3D solid–modeling programs to verify the constructability of the connections. It is important that the programs used have interoperability as outlined by the CIMsteel, Integration Standards/Version 2. This interoperability will allow for exchange of information and permit the fabricator to detail and download all of the required information for the CNC machines in his shop. This will help achieve the dimensional accuracy needed to make sure all of the members fit properly in the field.

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CHAPTER 8
QUALITY CONTROL AND QUALITY ASSURANCE

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8.1 PRINCIPLES OF QUALITY CONTROL AND QUALITY ASSURANCE

The definitions for quality control (QC) and quality assurance (QA) used in the *AISC Specification for Structural Steel Buildings* (AISC 360-16) and other AISC specifications are consistent with those used in the construction industry. Quality control is defined as “Controls and inspections implemented by the fabricator or erector, as applicable, to ensure that the material provided and work performed meet the requirements of the approved construction documents and referenced standards.” Quality assurance is defined as “Monitoring and inspection tasks to ensure that the material provided and work performed by the fabricator and erector meet the requirements of the approved construction documents and referenced standards. Quality assurance includes those tasks designated ‘special inspection’ by the applicable building code.”

QC and QA are defined differently in the AISC quality certification standards, for consistency with International Standards Organization (ISO) and American Society for Quality (ASQ) documents, and use the definition provided in ANSI/ISO/ASQ Q9000-2000. In these standards, QC is defined as “… the inspection of work. Conformity evaluation and judgment accompanied as appropriate by measuring, testing, or gauging.” QA is defined as “that part of quality management focused on providing confidence that quality requirements will be fulfilled” and is the function of the fabricator and erector.

ISO 9000 Quality Management Systems—Fundamentals and Vocabulary (ISO 9000:2015) defines quality control as “part of quality management focused on fulfilling quality requirements” and quality assurance as “part of quality management focused on providing confidence that quality requirements will be fulfilled.” Quality management can be stated as coordinated activity to direct and control an organization to such a degree that a set of inherent characteristics of an object fulfils requirements.

Quality control by the steel fabricator and erector is an AISC requirement, involving all levels of the production workforce, supervision and management. It includes monitoring those production tasks that affect quality, and measurements to verify quality in the fabricated steel or erected structure.

On the other hand, quality assurance is not an AISC requirement, but is performed by a third party when required by the authority having jurisdiction, often termed the building official, the applicable building code such as an government-adopted version of the

International Building Code, the purchaser or owner of the structural steelwork, or the engineer of record responsible for the design of the structure. It is performed when necessary to provide a level of assurance that the fabricated steel or erected structure meets the project requirements.

8.2 STANDARDS FOR QC AND QA

AISC Specification requirements for QC and QA follow the same principles for inspection as used in related steel construction standards. For bolting, the Research Council on Structural Connections—Specification for Structural Joints Using High-Strength Bolts (RCSC, 2014), hereafter referred to as the RCSC Specification, is cited and used. For welding, the American Welding Society standard AWS D1.1/D1.1M Structural Welding Code-Steel (AWS D1.1:2015), hereafter referred to AWS D1.1, is cited and used.

In addition to the AISC Specification, QC and QA are addressed in the AISC's *Seismic Provisions for Structural Steel Buildings* (AISC 341-16), hereafter referred to as AISC Seismic Provisions, and the AISC's *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC 358-16). AISC's *Code of Standard Practice for Steel Buildings and Bridges* (AISC 303-16), hereafter referred to as *AISC Code*, provides additional requirements for shop operations, field operations and inspection.

8.3 FABRICATOR'S AND ERECTOR'S QC PROGRAMS

8.3.1 Fabricator and Erector QC Activities

The *AISC Code*, Chapter 8, requires the fabricator and erector to maintain a QC program as part of their normal operations. The AISC Specification, Chapter N, does not require a specific QC system, but requires the fabricator and erector to perform certain inspections and tasks as a part of what is termed a "quality control program" defined as a "program in which the fabricator or erector, as applicable, maintains detailed fabrication or erection and inspection procedures to ensure conformance with the approved design drawings, specifications, and referenced standards." A quality control program may include practices such as policies, document control, personnel qualifications, and methods of tracking production tracking.

The fabricator's quality control program should include, as a minimum:

- Receiving and retaining material test reports for main structural steel elements, steel castings, steel forgings, anchor rods, and threaded rods
- Receiving and retaining manufacturers certifications for fasteners, headed stud anchors and welding consumables
- For welding filler metals and fluxes, availability of the manufacturer's product data sheets or catalog data that describe the product, limitations of use, recommended or typical welding parameters, and storage and exposure requirements, including baking, if

applicable

- Availability and distribution of welding procedure specifications (WPSs)
- Availability of welding procedure qualification records (PQRs) for WPSs that are not prequalified
- Availability of welding personnel performance qualification records (WPQR) and continuity records
- Material control procedures
- Nonconformance procedures
- QC inspector qualifications
- NDT personnel qualifications, if NDT is self-performed

The erector's quality program should include the same items as listed above for fabricators, but material test reports and material certifications would be needed only for those items purchased and supplied by the erector.

The fabricator's quality control program needs to include inspection procedures for the following shop operations:

- Welding
- High-strength bolting
- Connection details
- Cut and finished surfaces
- Heating for straightening, cambering and curving
- Verification of fabrication tolerances

The erector's quality control program needs to include inspection procedures for the following field operations:

- Welding, including headed steel stud anchor placement and welding
- High-strength bolting
- Connection details
- Cut surfaces
- Heating for straightening
- Verification of erection tolerances

QC inspection is done to the shop drawings and erection drawings, with the applicable referenced specifications, codes and standards. The *AISC Code* requires the transfer of relevant information from design drawings and project specifications to the shop and erection drawings, allowing QC inspection to be based upon these drawings alone, without reference back to the design drawings or project specifications.

Inspection documentation should include identification of the product inspected, the type of inspection performed, the name of the inspector, and the date(s) of inspection. Any nonconformances and the correction of those nonconformances should also be noted. Such documentation may include marks or tags on the production piece, notes placed on shop or erection drawings, production tracking records, or digital record-keeping systems.

The level of detail in the inspection record should be sufficient to provide confidence that the product is in compliance with the project requirements. It is not normally required to document or record detailed dimensions, names of production personnel who performed the work, identification of member or component heats or production lots, or other items that are controlled as a part of the fabrication or erection process.

8.3.2 QC Inspection Personnel

In accordance with the AISC Specification, Chapter N, the fabricator's and erector's QC program is to state the required qualifications of their quality control inspectors (QCIs).

Chapter N requires that QC welding inspection personnel be qualified to the requirements stated in *AWS B5.1 Standard for the Qualification of Welding Inspectors* (AWS B5.1:2013) at the level of associate welding inspectors (AWIs) or higher. Welding inspectors qualified in accordance with AWS D1.1 Clause 6.1.4.1 item (1) with "current or previous certification as an AWS Certified Welding Inspector (CWI) in conformance with the provisions of AWS QC1, Standard for AWS Certification of Welding Inspectors" are acceptable, and certified associate welding inspectors (CAWIs) and senior certified welding inspectors (SCWIs), as described in QC1, should also be considered acceptable. Likewise, welding inspectors qualified in accordance with AWS D1.1 Clause 6.1.4.1 item (2) with "current or previous qualification by the Canadian Welding Bureau in conformance with the requirements of the Canadian Standard Association Standard CSA W178.2, Certification of Welding Inspectors" are acceptable, including inspectors at levels I, II, and III.

AWS D1.1 Clause 6.1.4.1 also includes item (3) "an individual who, by training or experience, or both, in metals fabrication, inspection, and testing, is competent to perform inspection of the work." This is also considered compliant with the AISC Specification. In steel fabrication facilities and project sites with repetitive work using standard, lower-strength materials, with a limited number of welders, straightforward welding procedures, and basic joints such as fillet welds and unrestrained groove welds, a welding inspector with limited knowledge and experience may be adequate, provided the inspector is familiar with all aspects of the work being performed. The fabricator and erector should consider the inspector's experience in structural welding as a part of their QC program.

In addition to the aforementioned certifications and programs, the International Code Council offers certifications for special inspectors for a variety of structural materials, including a certification for "structural welding special inspector," designated the S2 certification by the ICC. An examination is required, but there is no minimum experience considered for this certification.

The fabricator's and erector's QC bolting inspection personnel should be qualified on the basis of documented training and experience in structural bolting inspection. Neither the AISC Specification not the RCSC Specification cites specific certifications for bolting inspectors.

The International Code Council also offers a certification for “structural steel and bolting special inspector” designated the S1 certification. Similar to the S2 certification, an examination is required, but there is no minimum experience considered for this certification.

8.3.3 Fabricator and Erector Approvals

The authority having jurisdiction (AHJ), commonly called the building official, is given the authority by most building codes to waive any building code provisions for special inspection, termed QA inspection by the AISC Specification, for fabricated structural steel. The AISC Specification provides for waiver of QA inspection for both steel fabrication and steel erection. The AHJ must approve the fabricator and/or erector to perform the work without special inspection.

For approval, the AHJ is to review the fabricator’s written procedural and quality control manuals, and there is to be periodic auditing of fabrication practices by an approved agency. Many AHJs rely upon independent certifications, such as the AISC Certification Program for Structural Steel Fabricators, using the *Standard for Steel Building Structures* (AISC 201-06), that reviews the quality management system of the fabricator and conducts periodic shop audits. Similarly, steel erectors may be certified under the AISC Certification Program for Structural Steel Erectors, using the *Standard for Structural Steel Erectors* (AISC 206-13), that reviews the quality management system of the erector and conducts periodic jobsite audits. The audits confirm that the company has the personnel, knowledge, organization, equipment, experience, capability, procedures, and commitment to produce the required quality of work for a given certification category.

In addition to the AISC certification programs, AHJs may rely upon the International Accreditation Service, an arm of the International Code Council, which offers a program entitled *AC172 Accreditation Criteria for Fabricator Inspection Programs for Structural Steel*. For metal building system fabricators, there is *AC472 Accreditation Criteria for Inspection Programs for Manufacturers of Metal Building Systems*. For metal building system erectors, there is *AC478 Accreditation Criteria for Inspection Practices of Metal Building Assemblers*.

At completion of fabrication, because the fabricator has taken full responsibility for inspection of the work with no third-party independent QA inspection, the approved fabricator is to submit a certificate of compliance to the AHJ stating that the materials supplied and work performed by the fabricator are in accordance with the construction documents. Similarly, at completion of erection, the approved erector is to submit a certificate of compliance to the AHJ stating that the materials supplied and work performed by the erector are in accordance with the construction documents.

8.4 QUALITY ASSURANCE PROGRAMS

8.4.1 QA Inspection Activities

The quality assurance inspector (QAI) is to review the material test reports and certifications for the materials used by the fabricator and erector for compliance with the construction documents. These materials include for

- Main structural steel elements, material test reports
- Steel castings and forgings, material test reports
- Fasteners, manufacturer's certifications
- Anchor rods and threaded rods, material test reports
- Welding consumables, manufacturer's certifications
- Welding filler metals and fluxes, manufacturer's product data sheets or catalog data that describe the product, limitations of use, recommended or typical welding parameters, and storage and exposure requirements, including baking, if applicable
- Headed stud anchors, manufacturer's certifications

Building codes require inspections to made using "approved construction documents," which include the design drawings and specifications submitted and approved by the building official or the authority having jurisdiction (AHJ), including any subsequent revisions that have been approved. The QAI may also use the shop drawings and erection drawings to assist in the inspection process.

The engineer of record (EOR) should evaluate what is already a part of the fabricator's or erector's QC system in determining the quality assurance needs for each project. Where the fabricator's or erector's QC system is considered adequate for the project, including compliance with any specific project needs, the special inspection or quality assurance plan may be modified to reflect this. Similarly, where additional needs are identified, supplementary requirements should be specified.

Coordination of QC and QA inspection tasks may be needed for fabricators in remote locations, distant from the jobsite, or distant from the home base of the inspector(s). Similarly, when jobsites are remotely located or distant from the home base of the inspector(s), coordination of QC and QA tasks may be needed. Coordination may be particularly helpful where QC and QA inspection tasks are redundant. The AISC Specification permits coordination of QC and QA inspections between the QCI and QAI so that the inspection functions are performed by only one party; however, when QA is to rely upon QC for QA-designated inspection functions, the approval of the EOR and the AHJ is required. This may also serve as an alternative to waiving QA inspection altogether.

Concurrent with the submittal of inspection reports and nondestructive testing reports to the AHJ, EOR and/or owner, the QA agency is to submit these reports to the fabricator and erector.

8.4.2 QA Inspection Personnel

The quality assurance agency determines the qualifications, training, and experience required for personnel who are to conduct QA inspections, with these qualification requirements documented in the QA agency's written practice. Qualification requirements may be based on the actual inspections to be performed on a particular project, and may also include experience, knowledge and physical requirements such as visual acuity examinations for welding inspectors.

In accordance with the AISC Specification, Chapter N, QA welding inspection personnel should be qualified to the requirements stated in AWS B5.1 at the level of welding inspector (WI) or senior welding inspector (SWI). An associate welding inspector (AWI) is permitted when under the direct supervision of WIs or SWIs who are on the premises and available when weld inspection is being conducted. This is similar to the permission granted in AWS D1.1 Clause 6.1.4.3 to use assistant inspectors who may perform specific inspection functions under the supervision of the inspector. Assistant inspectors must be qualified by training and experience to perform the specific functions to which they are assigned, and their work must be regularly monitored by the Inspector, generally on a daily basis.

In addition to the above, the AISC Specification permits inspections to be performed by welding inspectors qualified in accordance with AWS D1.1 Clause 6.1.4.1, as is permitted for QC inspectors. It should be noted that although QA welding inspection personnel are to be WIs or SWIs, there is no distinction made that CAWIs under AWS QC1, and level I welding inspectors under CSA W178.2, would not be permitted. In addition, as is permitted for QC welding inspectors, those individuals deemed competent by their employers are still permitted as QA welding inspectors.

QA bolting inspection personnel should be qualified on the basis of documented training and experience in structural bolting inspection, the same as QC bolting inspection personnel.

In certain locations in the United States, particularly in the West and in large metropolitan areas, building officials often require ICC special inspection certifications for structural welding and structural bolting inspection personnel.

8.4.3 Nondestructive Testing Personnel

For individuals performing only nondestructive testing (NDT) work, the inspector need not be generally qualified for welding inspection. However, the individual must be qualified using the provisions of the American Society for Nondestructive Testing's (ASNT) *Recommended Practice No. SNT-TC-1A Personnel Qualification and Certification in Nondestructive Testing*. This document provides recommendations for the training, experience and testing of NDT technicians. A suitable alternative to the *recommended practice*, although not referenced in AWS D1.1, is *ANSI/ASNT CP-189 ASNT Standard for Qualification and Certification of Nondestructive Testing Personnel*. Both reference *ANSI/ASNT CP-105 ASNT Standard Topical Outlines for Qualification of Nondestructive Testing Personnel* for technical training and knowledge.

These documents address specific subjects applicable to several areas of NDT, including

- Radiographic testing
- Magnetic particle testing
- Ultrasonic testing
- Liquid penetrant testing
- Visual testing (SNT-TC-1A only)

NDT technicians are placed into four categories. Formal definitions vary between the SNT-TC-1a and CP-189. Using the definitions of the CP-189, the level III technician has the "skills and knowledge to establish techniques; to interpret codes, standards and specifications; to designate the particular technique to be used; and to verify the adequacy of procedures." This individual is responsible for the training and testing of other NDT personnel in the individual's area of certification. The level II technician has "the skills and knowledge to set up and calibrate equipment, to conduct tests, and to interpret, evaluate, and document results in accordance with procedures approved by an NDT Level III." The level I technician has "the skills and knowledge to properly perform specific calibrations, specific tests, and with prior written approval of the Level III, perform specific interpretations and evaluations for acceptance or rejection and document the results." The trainee is a technician who works under the supervision of a level II or III, and cannot independently conduct any tests or report any test results.

Many level III technicians have taken and passed a nationally administered ASNT examination in the particular field of NDT, as required by CP-189. However, under SNT-TC-1a, it is possible for an individual to be administratively named by the employer to a level III designation, based upon his or her experience and knowledge in the field.

8.5 INSPECTION OF BOLTED CONNECTIONS

8.5.1 Scope of Inspections

The RCSC Specification defines bolting inspection requirements in terms of inspection tasks and scope of examinations. The RCSC Specification uses the term "routine observation" for inspection, hence the choice of the term "observe" for the AISC Specification. Bolting inspection tasks are prescribed in the AISC Specification, Tables N5.6-1, N5.6-2, and N5.6-3, summarized in Table 8.1. In these tables, the inspection tasks are prescribed as follows:

TABLE 8.1 Bolting Inspection Tasks

Inspection tasks prior to bolting	QC	QA
Manufacturer's certifications available for fastener materials (RCSC Sections 2.1 and 9.1)	O	P
Fasteners marked in accordance with ASTM requirements (RCSC Figure C-2.1, Section 9.1 and ASTM standards)	O	O
Correct fasteners selected for the joint detail (grade, type, bolt length if threads are to be excluded from shear plane) (RCSC Sections 2.3.2, 2.7.2, and 9.1)	O	O
Correct bolting procedure selected for joint detail (RCSC Sections 4 and 8)	O	O
Connecting elements, including the appropriate faying surface condition and hole preparation, if specified, meet applicable requirements (RCSC Sections 3, 9.1, and 9.3)	O	O
Preinstallation verification testing by installation personnel observed and documented for fastener assemblies and methods used (RCSC Sections 7 and 9.2)	P	O
Protected storage provided for bolts, nuts, washers, and other fastener components (RCSC Sections 2.2, 8, and 9.1)	O	O
Inspection tasks during bolting	**QC**	**QA**
Fastener assemblies placed in all holes and washers and nuts are positioned as required (RCSC Sections 7.1(1), 8.1, and 9.1)	O	O
Joint brought to the snug-tight condition prior to the pretensioning operation (RCSC Sections 8.1 and 9.1)	O	O
Fastener component not turned by the wrench prevented from rotating (RCSC Sections 8.2 and 9.2)	O	O
Fasteners are pretensioned in accordance with the RCSC Specification, progressing systematically from the most rigid point toward the free edges (RCSC Sections 8.2 and 9.2)	O	O
Inspection tasks after bolting	**QC**	**QA**
Document acceptance or rejection of bolted connections (not addressed by RCSC Section 9)	P	P

Observe (O): The inspector shall observe these items on a random basis. Operations need not be delayed pending these inspections.

Perform (P): These tasks shall be performed for each bolted connection.

8.5.2 Inspection prior to Bolting

Connected Materials. The size of bolt hole used for a particular size bolt may vary with the type of joint and hole selected by the engineer. The AISC Specification, Table J3.3, provides the given hole sizes for each diameter of bolt and hole type. These values are provided in Table 8.2 for convenience.

TABLE 8.2 Nominal Hole Dimensions

Bolt diameter	Standard (STD)	Oversize (OVS)	Short-slot (SSL)	Long-slot (LSL)
		inch		
½	9/16	⅝	9/16 × 11/16	9/16 × 1¼
⅝	11/16	13/16	11/16 × ⅞	11/16 × 1-9/16
¾	13/16	15/16	13/16 × 1	13/16 × 1⅞
⅞	15/16	1-1/16	15/16 × 1⅛	15/16 × 2-3/16
1	1⅛	1¼	1⅛ × 1-5/16	1⅛ × 2½
≥ 1⅛	d + ⅛	d + 5/16	(d + ⅛) × (d + ⅜)	(d + ⅛) × (2.5d)
		mm		
M16	18	20	18 × 22	18 × 40
M20	22	24	22 × 26	22 × 50
M22	24	28	24 × 30	24 × 55
M24	27	30	27 × 32	27 × 60
M27	30	35	30 × 37	30 × 67
M30	33	38	33 × 40	33 × 75
≥ M36	$d + 3$	$d + 8$	$(d + 3) \times (d + 10)$	$(d + 3) \times (2.5d)$

It should be noted that the 2016 AISC Specification increased the nominal dimension for standard (STD) holes by 1/16 in for bolts of 1-in diameter and greater, and likewise increased the width of slotted (SSL and LSL) holes for these larger diameter bolts. This was done for a number of reasons, including allowing for bolt swell, fins, size tolerances and out-of-straightness values for larger bolts that sometimes made it impossible to fit bolts into holes when only 1/16 in (1.6 mm) clearance was provided. Increasing hole size also allowed for more efficient bolt installation without the need for reaming and slotting to enlarge and align holes for larger bolts in thicker materials. As a consequence, the use of oversize (OVS) holes for such conditions is anticipated to be minimized. The change in hole dimensions for the larger inch-series bolts also brings the table into alignment with the metric hole sizes provided in the AISC Specification and those commonly used in other international standards. The 2014 RCSC Specification was published before the AISC increased the hole sizes.

Oversize holes may be used only in slip-critical joints. Slotted holes (SSL and LSL) may be used in snug-tight and pretensioned joints only when the load is approximately transverse to the direction of the slot. Slotted holes loaded in the direction of the slot may be used only in slip-critical joints.

The faying surface, the contact surface between the connected plies, should be inspected prior to assembly to ensure the surfaces are free of dirt and other foreign material. When slip-critical joints are specified, more intense inspection is needed to verify that the surfaces meets the requirements for class A surfaces (unpainted clean mill scale steel surfaces, surfaces with class A coatings on blast-cleaned steel, or hot-dipped galvanized and roughened surfaces) or class B surfaces (unpainted blast-cleaned steel surfaces, or surfaces with class B coatings on blast-cleaned steel), as defined in the AISC Specification, Section J3.8. With coated surfaces, it should also be verified that the coating thickness is within the range specified for the coating, and that the coating has completely cured prior to assembly. Slip-critical hot-dipped galvanized surfaces are to be roughened using a hand wire brush to the extent that scratch marks are visible in the zinc. Additional details are provided in the RCSC Specification, Section 3.

Fastener Materials. The quality of the fastener components begins with the manufacturer of the steel. Steel is purchased by bolt, nut, and washer manufacturers to rigid chemical specifications so that, after manufacture and heat treatment, if performed, the desired mechanical properties will be achieved. The quality of the steel is verified through the use of material test reports provided by the steel mill and reviewed by the fastener manufacturer.

The manufacturer will make several hundred to several thousand components in each production lot, depending upon the type of product and the manufacturing facility. A *production lot* is defined by ASTM F1749 as a "quantity of product of one part number that has been processed essentially under the same conditions from the same heat treatment lot and produced from one mill heat of material and submitted for inspection at one time." Testing is performed during production to verify that dimensional tolerances are met. Random sampling may be performed for physical testing, but generally this is left until the completion of the production run. Physical testing is performed following the completion of heat treatment, if performed, and following galvanizing and some other coatings application.

The type of testing required depends upon the type of product being manufactured, and are specified in the applicable ASTM Specification. Bolts are tested for strength and ductility, with additional tests for higher strength bolts and galvanized assemblies. Nuts are tested for stripping resistance and strength on a hardened threaded mandrel, with a block attempting to push the nut down the mandrel. Hardness tests are also performed to verify proper nut strength. Hardness tests are performed on washers.

Bolt strength is tested in a tensile testing machine. A wedge of either 6° or 10° is placed under the head of the bolt, then the bolt is pulled to failure. The failure must take place in the threads of the bolt, between the nut and the shank. Failure directly underneath the bolt head or in the shank, or by stripping of the threads, is unacceptable. The elongation of the bolt is also measured as tensile loading is applied. The bolt must satisfy the requirements for minimum proof load, which is established as 70% of the minimum required tensile strength for 120 ksi tensile strength bolts and 80% of the minimum required tensile strength for 150 ksi tensile strength bolts. The proof load establishes that the bolt will not yield prematurely at a lower stress level, and therefore not provide the pretension desired when installed using established techniques. When bolts are too short to fit into a tensile testing machine, they are checked using alternative tensile tests and hardness tests to establish minimum and maximum strength levels.

Bolts that are galvanized are to be supplied as an assembly, with the washers and nuts that are to be used with the bolts. An assembly rotational-capacity test is performed to verify that the effect of galvanizing and the overtapping of the nut did not adversely affect the assembly performance. The test involves deliberately overtightening the assembly in a test fixture, ensuring the bolt and nut have adequate strength, then verifying that the threads of the bolt and nut resist stripping.

For bridge work, special rotational-capacity tests are performed for both uncoated and coated assemblies. The testing includes checking the torque required for tightening the assembly, with a maximum torque value used to confirm the effectiveness of the nut lubrication, as well as verifying reserve strength after over-rotation, and thread stripping resistance.

Fastener components are physically tested using statistical sampling techniques, as prescribed by the applicable ASTM specifications. Zero defects are permitted for strength and proof-load requirements. Should the item fail a strength, proof-load, or a hardness test if used, then the entire production lot is rejected. Generally, lots rejected on the basis of strength are heat-treated again, and then retested.

Bolts may have small cracks, called *bursts*, in the head of the bolt. Bursts are acceptable provided they meet the dimensional limits of depth and width, and do not reduce the dimension across the points of the head below prescribed limits.

The AISC Specification, Table N5.6-1, includes requirements for observation of "Fasteners marked in accordance with ASTM requirements." This includes the required package marking of the fasteners and the product marking of the fastener components in accordance with the applicable ASTM standard. As an example, ASTM F3125 requires the following items for package marking: ASTM designation, grade, type and style; size; name of the manufacturer or responsible party; number of pieces; lot number; purchase order number or other distinguishing information, when required by the customer; and country of origin, when required by the customer. ASTM F3125 requires manufacturer identification and grade identification on the head of each bolt.

Manufacturer Certificates. Bolts, nuts, and washers are typically purchased as commodity items and are placed into inventory. Because the shop bolt list is not completed until the shop detail drawings are done, and the field bolt list is not done until the erection plans and shop details are done, bolts are ordered in advance using estimates of quantities and lengths.

Because the RCSC Specification requires preinstallation testing for fastener assemblies of the bolt and nut production lot combinations to be used, and the direct tension indicator (DTI) lot when used, lot identification and control of fastener components is needed, and products of one lot should not be mixed with those of other production lots. Only a few fastener manufacturers place their production lot number on the fastener itself. This is a requirement for DTIs. All others place their lot identification on the packaging only. Once removed from the container, lot identification can be maintained only through established shop or field control procedures.

Manufacturer's test reports are to be supplied by either the manufacturer or the supplier, as applicable, when requested by the purchase order, according to the ASTM standards. However, the AISC Specification and RCSC Specification require that manufacturer's certifications (test reports) be made available for all fasteners.

Protected Storage and Lubrication of Fastener Components. Bolts, nuts, washers, and direct tension indicators must be maintained in protected storage, defined by the RCSC Specification as "the continuous protection of fastener components in closed containers in a protected shelter."

All black (uncoated, or plain) bolts, nuts, and washers should have a "lubricant" present when installed. It should be noted that it is not an ASTM requirement that uncoated fastener components be lubricated. It is common industry usage to call residual oil a "lubricant." Most uncoated fasteners have a residual water-soluble oil as a result of the production operations, particularly heat treated fasteners that are quenched or tempered in this oil. If the fasteners are exposed to rain, snow, dew, condensation, or other moisture conditions, this residual oil may be washed off. This oil may also evaporate after a short period of time when left in open containers.

Uncoated fasteners should be oily to the touch prior to being installed. When compared to oily fasteners, bolts that have lost their “lubrication” may require as much as twice the torque to install them, requiring more time and more powerful tools. In addition, the bolt’s ductility (ability to stretch) is reduced because of the higher bolt torsional stress from the torque used to tighten a poorly lubricated fastener.

Should any of the bolts, nuts, or washers show significant rust, the rust should be cleaned from the surface of the fastener component and the component lubricated. Dirt, sand, grit, and other foreign material should be cleaned off the bolts prior to installation, with lubrication added when necessary. If a bolt, nut, or washer has lost its “lubrication,” it may need to be lubricated prior to installation. Note that is not permitted to add or modify lubrication on any component of a twist-off-type tension control bolt assembly.

The type of lubrication to be used is not specified, but typically a similar oil-based product, stick wax, bee’s wax, liquid wax, or spray lubricant may be used. The most effective lubrication is placed on the threads of the nut and on the inside face of the nut. Approximately half or more of the torque applied to tighten a bolt is used to overcome the friction between nut face and the washer or steel. Roughly one-third of the torque applied is used to overcome the friction between nut threads and bolt threads. Often, it is necessary only to lubricate the nut, leaving the bolt and washer in the “dry” condition.

In some cases for uncoated bolts, lubrication, or relubrication mandates the retesting of fasteners in a bolt tension calibration device prior to installation in the structure. This verifies the effectiveness of the lubrication. Highly efficient lubricants can actually increase the risk of thread stripping, so this condition is also checked. If the calibrated wrench method is used for pretensioning, any relubrication mandates the recalibration of the installation wrenches.

Many twist-off-type tension control bolt assemblies use a lubricant that is not as oily as common structural bolts. Such fastener assemblies are particularly sensitive to inadequate lubrication and overlubrication, and loose bolts or broken bolts may result. Only the manufacturer of the twist-off bolt assembly is permitted to relubricate a component of the assembly.

Galvanized nuts, either hot-dipped galvanized or mechanically galvanized, are specifically lubricated, unlike uncoated nuts. They are not oily. The nut is the only lubricated component of the assembly. ASTM A563 requires that galvanized nuts receive a lubricant that is clean and dry to the touch. Usually a wax-based or proprietary product is used, but the lubricant’s presence may not always be determined by touch. Often, a dye is added to the lubricant to verify that the nuts have indeed been lubricated. Sometimes, a UV solution is used in the lubricant to make the nut “glow” under ultraviolet light.

Mixing of galvanizing types in a fastener assembly is not permitted, so only hot dip galvanized nuts can be used with hot dip galvanized bolts, and only mechanically galvanized nuts can be used with mechanically galvanized bolts. For a given manufacturer, the color in the dye used for most galvanized nuts can be used as an indicator of the type of galvanizing performed, and as a means to detect improper assemblies.

If relubrication of galvanized nuts is required, a wax-based or similar lubricant works well. Apply the lubricant to the threads of the nut and to the inside face of the nut. It is not necessary to lubricate the bolt or washer when this is done. After relubrication, test the assembly in a bolt tension calibration device for torque performance and resistance to stripping, using the rotational capacity testing procedures of ASTM F3125 or ASTM A325.

Relubrication rarely negatively affects the performance of bolts using the turn-of-nut or the DTI methods of installation. If the adequacy of a lubricant is uncertain, testing in a bolt tension calibration device will provide indication of the lubrication's adequacy. As a recommendation, if the torque required to tighten the assembly is less than the maximum torque permitted in the AASHTO or ASTM F3125 rotational-capacity test, then the assembly may be considered adequately lubricated. For bridge work, the rotational-capacity test is required following any relubrication.

Bolt Length Selection. *Stickout* is the amount of thread sticking out beyond the face of the nut after tightening. The RCSC Specification requirement is that the end of the bolt be at least flush with the face of the nut. The bolt end *cannot* be below the face of the nut after tightening is completed.

There is no maximum stickout by specification, but excessive stick-out indicates a risk that the nut has actually met the thread runout. If this has occurred, pretensioning is questionable for the calibrated wrench and twist-off bolting methods because the nut would cease rotation and the torque would become very high, although the bolt would remain loose. For the turn-of-nut method, the required turns could not be applied. For the DTI method, the DTI gap requirements would not be achieved.

For pretensioned bolts, a second danger of maximum stickout is that the risk of thread stripping is increased. The bolt threads will neck down in a very short region when the bolt is pretensioned, reducing the thread contact between bolt and nut.

Excessive stickout measurement is determined by the actual bolt and nut combination, and can be checked visually using an untightened bolt with the nut run up to the bolt thread runout. Generally, six threads of stickout can be permitted for ½-, ⅝-, ¾-, and 1-⅛-in bolts. For ⅞-, 1-, 1-¼-, and 1-⅜-in bolts, five threads of stickout can be permitted, and for 1-½-in bolts, four threads. Stickout beyond these values should be checked with the comparison set, and may be found acceptable.

Bolt ductility is highest when the nut is flush with the end of the bolt because of the maximum number of threads available for inelastic behavior from pretensioning. With maximum stickout, the bolt's ductility is reduced because the inelastic behavior is limited to the very short length of thread in the grip.

A traditional "rule of thumb" had been to require two threads of stickout for high-strength bolts. This was a guideline developed for applications when the threads-excluded condition was specified. It is neither a valid indicator that the threads-excluded condition has been achieved, nor is it required by specification; therefore, it should is not an installation or inspection requirement.

Use of Washers. The RCSC Specification, Section 6, provides the following situations where ASTM F436 hardened steel washers and other special washers are required. Washers are suggested, even for cases when not required, to ease installation and provide better consistency for installation and inspection.

1. For shear-bearing joints, if either snug-tight or pretensioned using the turn-of-nut method or the direct tension indicator method, and if only standard (STD) holes are present in the outer plies, washers are not required over the holes.
2. For shear-bearing joints with slotted (SSL and LSL) holes present in an outer steel ply, either snug-tight or pretensioned, an ASTM F436 washer or common plate washer is required over the slot.
3. If the slope of the face of the connected part exceeds 1:20, approximately 3°, relative to the bolt or nut face, a hardened beveled washer must be used between the fastener and the steel to compensate for the slope.

The following provisions apply only to pretensioned bolts:

1. If the calibrated wrench method is used, an ASTM F436 washer must be used under the turned element.
2. If twist-off bolts are used, the supplier's washer must be used under the nut.
3. If AISC Group B bolts are used in ASTM A36 steel [or other steels below 40 ksi (280 MPa) yield strength], an ASTM F436 washer must be provided over the hole.
4. If oversize or short-slotted (SSL) holes are used in an outer steel ply, and the bolts are AISC Group A of any diameter or AISC Group B of 1 in diameter or less, an ASTM F436 washer must be placed over the hole or slot.
5. If oversize (OVS) or short-slotted (SSL) holes are used in an outer steel ply, and the bolts are AISC Group B over 1 in diameter, an extra-thick ASTM F436 washer must be placed under both bolt head and nut. Multiple standard thickness washers cannot be substituted for the thicker single washer.
6. If a long-slotted (LSL) hole is used in an outer steel ply, and the bolts are AISC Group A of any diameter or AISC Group B of 1 in diameter or less, a plate washer or continuous bar of minimum 5/16 in (8 mm) thickness with standard holes must be used to cover the slot. The bar or plate material must be of structural grade, but need not be hardened.
7. If a long-slotted hole is used in an outer steel ply, and the bolts are AISC Group B of over 1 in diameter, a plate washer or continuous bar of minimum ⅜ in (10 mm) thickness with standard holes must be used to cover the slot, with an ASTM F436 washer over the hole. The bar or plate material must be of structural grade, but need not be hardened.
8. If a twist-off bolt has a round head with a diameter satisfying the requirements of ASTM F3125, ASTM F1852 or ASTM F2280, no washer is required under the bolt head.

Preinstallation Verification Testing. For snug-tight joints, preinstallation verification testing as specified in the AISC Specification and RCSC Specification is not applicable. It is performed only for pretensioned bolts, whether in pretensioned joints or in slip-critical joints.

Preinstallation verification testing checks the assembly of bolt, nut, and washer (if used) in a bolt calibration device for material quality, verifying that it is capable of achieving the required pretension without breaking, thread stripping, or excessive installation effort. Three assemblies of each lot combination are used, and is done at the start of the work, before the assemblies are used in the project. To perform calibration of the wrenches, for the calibrated wrench method only, the testing is done before the start of the work each day.

The testing confirms the effectiveness of the installation technique for that group of fasteners. Perform the installation technique in a bolt calibration device, or with a "calibrated" DTI if the bolt is too short to fit into the calibrator. Verify that at least the minimum required pretension, plus 5%, is achieved using the specified technique, as shown in Table 8.3. It should be noted that this table is based upon the 2016 AISC Specification, which incorporates the higher minimum tension requirements provided in ASTM F3125 for grade A325 and grade F1852 bolts above 1 in diameter, and adds the values for group C grade 2 assemblies. The 2014 RCSC Specification, including Table 7.1, and the prior AISC Specification uses a lower pretension value for ASTM A325 and ASTM F1852 bolts with 1⅛ in diameter and greater.

TABLE 8.3 Minimum Bolt Pretension for Preinstallation Verification

Inch	Kips		
	ASTM F3125		
Nominal bolt diameter, d_b	Grade A325 Grade F1852	Grade A490 Grade F2280	ASTM F3043, Grade 2 ASTM F3111, Grade 2
½	13	16	—
⅝	20	25	—
¾	29	37	—
⅞	41	51	—
1	54	67	94
1-⅛	67	84	119
1-¼	85	107	150
1-⅜	102	127	—
1-½	124	155	—
mm		kN	
	ASTM F3125		
Nominal bolt diameter, d_b	Grade A325M	Grade A490M	—
M16	96	120	—
M20	149	188	—
M22	185	232	—
M24	215	270	—
M27	280	351	—
M30	353	428	—
M36	499	625	—

The final reason of the preinstallation verification test is to confirm the knowledge of the installation crew, therefore the test is performed by the installation crew, not by supervisors, quality control or quality assurance. With QC and QA observing these tests, the crew demonstrates to these inspectors their knowledge of the proper technique.

8.5.3 Inspection during Bolting

Observation of bolting operations is the primary method to verify that the bolting and connected materials, bolting procedures, and workmanship conform to the RCSC Specification and the project specifications. During bolting, the inspectors are to observe the installation crew to verify use of proper techniques, including snugging the joint, use of a systematic tightening pattern, and use of the proper pretensioning techniques if the joint is pretensioned or slip-critical. Observation does not mean that the installation of each individual bolt or connection is observed, but that the crew is observed to confirm they understand and follow the proper techniques on a consistent, uniform basis.

Snug-Tight Condition. Snug-tightened joints are inspected to ensure that the proper fastener components are used and that the faying surfaces are brought into firm contact during installation of the bolts. The magnitude of the bolt tension, whether high or low, is not prescribed in a snug-tightened joint is therefore is not verified.

The majority of bolts in buildings must be tightened only to the snug-tight condition. Bolts in specific types of shear-bearing joints and direct tension joints, slip-critical joints and joints that are a part of the seismic force resisting system (SFRS) are pretensioned.

The definition of *snug tightened joint* is stated in the AISC Specification as a “joint with the connected plies in firm contact as specified in Chapter J.” The RCSC Specification (2014) defines a *snug tightened joint* as “a joint in which the bolts have been installed in accordance with Section 8.1.” The *snug tightened condition* is defined by RCSC (2014) 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 plies into firm contact.” *Firm contact* is defined by RCSC as “the condition that exists on a faying surface when the plies are solidly seated against each other, but not necessarily in continuous contact.”

It should be noted that the RCSC Specification (2009) defined the snug-tight condition as “the condition that exists when all of the plies in a connection have been pulled into firm contact by the bolts in the joint and all of the bolts in the joint have been tightened sufficiently to prevent the removal of the nuts without the use of a wrench.” This definition was later determined to be insufficient, especially when turn-of-nut methods were used, and was replaced in 2014 with the definition used in prior editions.

Ideally, firm contact is the condition of no gaps between the steel plies at the bolt holes. In the snug condition, and even in a pretensioned condition, there may be cases where gaps remain between the steel. Gaps along the edges of parts may be caused by member tolerances, misalignment, or welding and heat distortion. These types of gaps along the edges of joints should be permitted. A thin metal feeler gage such as a machinist’s rule may be used to verify that no gaps are present around the bolt holes, even though gaps exist at the edges. For large multi-row bolt patterns, it may be necessary to remove a snugged bolt to check if gaps exist between the steel at the bolt hole.

The idealized snug tight condition of steel in contact at all bolt holes may not be possible for thick steel parts with smaller diameter bolts. It may not be possible to close all gaps at bolt holes in stiff or heavy connections where member tolerances, misalignment, welding, and heat distortion and similar problems cause minor gaps at individual bolt holes. The rigidity of the connection may be such that the thick connection parts may not be completely drawn down into contact, and the connection material allows no further deformation or bending upon further tightening. If gaps exist at these bolt holes, the engineer should be consulted for evaluation.

If the joint is not in firm contact, the pretensioning method employed may fail to achieve the proper pretension for the bolts in the joint. Pretensioning the first bolt in the group will only serve to further draw down the gap between the steel elements. The installer erroneously assumes the first bolt is tight. The next bolt tightened further draws down any remaining gap, and the initial bolt becomes looser still. This can become a compounding series in some joints.

Systematic Tightening. Joints are to be snugged and tightened in a systematic manner. A pattern should be chosen for tightening the bolts so that the joint is drawn together without undue bending of the connected parts. The systematic pattern should also be used so that bolts are not inadvertently missed during snugging.

The joint should be snugged first, starting at the most rigid part of the joint. In a joint with a single or double row of bolts, this would be where the steel is already in contact, working toward the end where the steel may not be in contact. If there is solid contact between the steel at all locations, the direction of tightening does not matter. In a bolt pattern with several rows, such as a large web splice plate in a girder, the bolts in the center of the joint should be snugged first; then proceed to work toward the free edges of the plate.

After the joint has been completely snugged, pretensioning of the bolts should follow the same systematic pattern so that bolts are not inadvertently missed during pretensioning.

Pretensioning. Bolts in specific types of shear-bearing joints and direct tension joints, slip-critical joints and joints that are a part of the SFRS are pretensioned. The technique to be used is to provide at least the minimum pretension in Table 8.4, based upon the requirements of the 2016 AISC Specification. Note that lower values for ASTM A325 and ASTM F1852 bolts of 1-⅛ in diameter and greater are provided in the earlier AISC Specification and in the 2014 RCSC Specification, reflecting the minimum bolt pretensions prior to the adoption of ASTM F3125 in 2014.

TABLE 8.4 Minimum Bolt Pretension

Inch	Kips		
	ASTM F3125		
Nominal bolt diameter, d_b	Grade A325 Grade F1852	Grade A490 Grade F2280	ASTM F3043, Grade 2 ASTM F3111, Grade 2
½	12	15	—
⅝	19	24	—
¾	28	35	—
⅞	39	49	—
1	51	64	94
1-⅛	64	80	113
1-¼	81	102	143
1-⅜	97	121	—
1-½	118	148	—
mm		kN	
Nominal bolt diameter, d_b	ASTM F3125		—
	Grade A325M	Grade A490M	
M16	91	114	—
M20	142	179	—
M22	176	221	—
M24	205	257	—
M27	267	334	—
M30	325	408	—
M36	475	590	—

The presence of the inspector is dependent upon whether the installation method provides visual evidence of completed installation. Turn-of-nut installation with match-marking, installation using twist-off bolts, and installation using direct tension indicators provide visual evidence of a completed installation, and therefore routine observation is stated for these methods. Turn-of-nut installation without match-marking and calibrated wrench installation provides no such visual evidence, and the inspector is to be “engaged” in more rigorous observation, although not watching every bolt or joint as it is being pretensioned.

Turn-of-Nut Method. The turn-of-nut method has been used since the 1940s for bolts, and since the 1950s for structural bolting. The principle behind the turn-of-nut method is the controlled elongation of the bolt. Because of the pitch of the threads, turning the nut a prescribed rotation elongates the bolt a certain amount. The elongation has a direct correlation to the bolt tension. As bolts become larger in diameter, the number of threads per inch decreases accordingly; allowing rotation to be based on a bolt diameter to length ratio to provide at least the required amount of pretension.

The current table of prescribed rotation has been in use since 1978, except that the tolerances were modified in the 2014 RCSC Specification, increasing the plus tolerance to 60° and making the minus tolerance consistent at 30° regardless of rotation required. Table 8.5 provides the required turns for given bolt length-to-diameter ratios, as provided in the 2014 RCSC Specification Table 8.1. No such table has been prepared for metric-dimensioned bolts. As an example, with flat surfaces and bolts less than or equal to four diameters in length, say a *¾- by 3-in* bolt, one-third turn must be provided. A ⅞ × *5-in* bolt would receive one-half turn. A *1* × *6-in* bolt would receive two-thirds turn. For bolts over 12 diameters in length, too much variation exists to provide tabular values. It is required that the installer use a bolt tension calibrator, such as a Skidmore-Wilhelm, to determine the number of turns required to provide the required bolt pretension.

TABLE 8.5 Nut Rotation from Snug-Tight Condition for Turn-of-Nut Pretensioning

Bolt length, L relative to bolt diameter, d_b	Disposition of Outer Faces of Bolted Parts		
	Both faces normal to bolt axis	One face normal to bolt axis, other sloped not more than 1:20*	Both faces sloped not more than 1:20* from normal to bolt axis
$L \leq 4d_b$	⅓ turn	½ turn	⅔ turn
$4d_b < L \leq 8d_b$	½ turn	⅔ turn	⅚ turn
$8d_b < L \leq 12d_b$	⅔ turn	⅚ turn	1 turn

- Nut rotation is relative to bolt regardless of the element (nut or bolt) being turned.
- For all required nut rotations, the tolerance is plus 60° (⅙ turn) and minus 30° (1/12 turn).
- Applicable only to joints in which all material within the grip is steel.
- When the bolt length exceeds $12d_b$, the required nut rotation shall be determined by actual testing in a suitable tension calibrator that simulates the conditions of solidly fitting steel.

*Beveled washer not used.

The sloping surfaces provisions apply when there is a slope to the surface beneath the bolt head or nut. This slope must not exceed 1:20, or about 3°. Extra rotation is needed to overcome the loss caused by the bending at the head or nut; therefore, a one-sixth turn is added for each sloping surface. If the slope exceeds 1:20, a beveled washer must be used to reduce the slope to no more than 1:20.

If the sloping surface is caused by the 16-⅔% (10°) bevel used for C- and S-section flanges, then a standard 16-⅔% beveled washer, commonly called a “hillside washer,” is used. The required turns increase for the sloping surface is not required, because the beveled washer has returned the head or nut to the parallel condition.

There is a tolerance to the amount of applied rotation. The nut may be over-rotated by no more than 60°, and may be under-rotated by 30°. The potential risk from excessive over-rotation is that the bolt may be stretched to the point of breaking, or to the point where nut stripping may occur. The permitted over-rotation was chosen to provide a sufficient margin of safety for most bolting conditions, and should a nut exceed the stated rotation plus tolerance, it need not be rejected unless bolt fracture or nut stripping has occurred. If the nut does not receive sufficient rotation, the desired pretension may not be achieved, and additional rotation is to be applied.

The turn-of-nut installation sequence should start with snugging the joint. Following completion of the snug-tightening operation, the installation crew may match-mark the end of the bolt shank and a corner or “point” of the nut. The crew then applies the required turns from the RCSC Specification, and the joint is inspected to verify the applied turns by checking the match-mark rotation.

The installation crew may also use the “watch the wrench chuck” method for turn-of-nut, electing not to match-mark. The inspector should more closely monitor the crew’s efforts to verify that the proper technique is routinely applied during the pretensioning, but need not watch every bolt or every connection.

The inspector is to observe the preinstallation verification testing required in the RCSC Specification, Section 8.2.1. Subsequently, routine observation is used to ensure that the bolting crew properly brings the joint to the snug-tight condition, then systematically tightens each bolt by rotating the turned element relative to the unturned element by the amount specified. When fastener assemblies are match-marked after the initial fit-up of the joint, but prior to pretensioning, visual inspection of the match-marks after pretensioning is permitted in lieu of routine observation of the rotation being applied.

Twist-Off-Type Tension Control Bolt Method. The *twist-off bolt* is a specially designed bolt that has a spline at the end that is used by the dual-socket installation wrench to control the torque-controlled installation of the bolt. This torque is the result of the outer wrench socket turning the nut in the clockwise (tightening) direction, with the wrench inner socket and friction between the bolt head and steel keeping the bolt shank from rotation in the counter-clockwise direction. The spline is designed to shear off from the torque generated by the wrench.

The controlled lubrication of the assembly, combined with design of the spline and strength of the bolt, is such that the spline will not shear off, or “twist off,” until the bolt is above the required pretension, but will not be too strong to cause tensile failure of the bolt. When the spline shears off, the wrench no longer functions.

The twist-off bolt is completely dependent upon the torque-tension relationship, which can vary greatly depending upon the quality and type of lubrication of the assembly. Some manufacturers of twist-off bolt assemblies use a very consistent and durable lubricant that resists water, mild solvents, and rust for some time. Others may use water-soluble oil lubrication which is subjected to evaporation or washing off during rain or other weather events. Installing twist-off assemblies when wet has been shown to affect achieve pretensions, sometime increasing and sometimes decreasing pretension. Temperature is also known to affect achieved pretensions, with generally higher pretensions when hot and lower pretensions when cold.

Because of the interdependence of the bolt, nut, and washer upon the torque used for installation, the twist-off bolt assembly is supplied preassembled by the manufacturer. Substitution of other nuts or washers may adversely affect performance and cause bolt pretensions to be too high, breaking the bolt, or too low, and therefore substitution of assembly components for those supplied by the manufacturer is prohibited. Likewise, because the manufacturer controls the performance of the bolt through their lubrication and spline breakneck design, modification of the lubrication by anyone other than the manufacturer is prohibited.

The joint is first snugged using a systematic method, as with all installation procedures. Care must be used to make sure that the spline is not twisted off during the snugging operation. Any bolts that twist off during snugging must be replaced. In some cases, deep sockets are used on conventional impact or other wrenches to snug the joints, therefore protecting the splines. Once the snug-tight condition is achieved, the installation crew proceeds to systematically tighten each twist-off bolt with the installation wrench until the spline shears off.

The inspector is to observe the pre-installation verification testing required in the RCSC Specification, Section 8.2.3. Subsequently, routine observation is used to ensure the bolting crew properly snugs the joint with the splines remaining intact, and that the splined ends are properly severed during pretensioning by the bolting crew.

Direct Tension Indicator Method. The *direct tension indicator*, or DTI, is a load-cell device used as proof that the required pretension has been provided in the assembly. The manufacturing and testing of the DTI itself is governed by ASTM F959. The effectiveness and reliability of the DTI, however, is also dependent upon the techniques used in installing the fastener.

The DTI has protrusions formed into the device that will be compressed when the bolt is pretensioned. The average gap remaining between the DTI face and the fastener element against which it is placed should not close below a specified gap until after the fastener has reached the required fastener tension. A feeler gage or experienced visual observation may be used to verify that the gaps have been suitably closed, therefore verifying that the bolt has been tightened to at least the minimum required pretension.

During installation, the element (bolt head or nut) against the DTI face must be held from turning to prevent abrasion of the protrusions, or an ASTM F436 washer is used between the element and the DTI. When the element against the DTI is to be deliberately turned, then an ASTM F436 washer must be used between the DTI and element. Details are provided in the RCSC Specification in Figure C-8.1. Some DTI manufacturers provide alternate washer requirements in their installation instructions that indicate that such washers are not required for their product.

The DTI protrusions must face outward away from the steel to keep the DTI from cupping against the nut face or bolt washer face, opening the gaps larger and voiding the measurement technique. When the DTI is used over an outer ply containing an oversize (OVS) or short-slotted (SSL) hole, a standard ASTM F436 washer is needed behind the DTI to prevent the DTI from cupping into the hole and voiding the gap measurement technique. When long-slotted (LSL) holes are present beneath a DTI, then plate washers are required that cover the entire slot, as is required for any other bolt component.

For building applications in which the DTI is placed directly underneath the bolt head, without washer, and the nut is turned, the average gap between DTI face and bolt face is to be 0.015 in (0.38 mm) or less, which is verified when the feeler gage of this thickness is refused entry in half or more of the gaps of the DTI.

If a washer is placed between the DTI and bolt head, whether or not the bolt head is allowed to turn, or if the DTI is used at the nut end, with or without washer, whether or not the nut is allowed to turn, the average gap is to be 0.005 in (0.13 mm) or less, and a feeler gage of this thickness is used.

For bridges, a 0.005-in (0.13-mm) average gap is always used and verified. The smaller gap is so that a bridge coating can be applied over the assembly containing the DTI, and such coatings can seal the gap between DTI and other component, forestalling crevice corrosion at the DTI gap.

The joint is first snugged using a systematic technique. It should be verified that the snug-tightening operation does not compress the DTI such that at least half the gaps refuse the feeler gage, or in other words, at snug tight, more than half the gaps must permit entry of the feeler gage. This is done to ensure that the bolt did not reach its required pretension during snugging, then subsequently loosen when adjacent bolts were snugged. Since the DTI is inelastic, it will not rebound to return the gap when the preload is released. Therefore, a bolt “oversnugged” during snug-tightening, then subsequently loosened, will still appear to be properly tensioned by having the sufficient number of refusals for the check after pretensioning.

The inspector observes the pre-installation verification testing required in RCSC Specification Sections 8.2 and 8.2.4. After snug-tightening and prior to pretensioning, routine observation is used to ensure that the appropriate feeler gage is accepted in at least half of the spaces between the protrusions of the DTI and that the protrusions are properly oriented (facing outward). If the appropriate feeler gage is accepted in fewer than half of the spaces, the DTI is to be removed and replaced, and the bolt checked for possible reuse, if permitted. After pretensioning, routine observation is used to ensure that the appropriate feeler gage would be refused entry into at least the half of the spaces between the DTI protrusions.

Calibrated Wrench Method. The calibrated wrench method uses an adjustable impact, electric or hydraulic wrench to tighten the bolts. Rather than tighten until the wrench operator releases the trigger, the wrench is adjusted to stop tightening when a certain torque is developed by the wrench. The wrench is adjusted so that it stops tightening when the bolt has achieved at least the required bolt pretension, as determined using a bolt calibration device.

Pneumatic calibrated impact wrenches depend upon an internal cam unit for control. When the desired resistance, actually torque, is reached, the cam unit shifts and the wrench stalls out. If the air pressure or air volume is inadequate, however, the control mechanism will not function properly and will continue to impact the fastener, although at a slower, weaker level. For this reason, the calibrated impact wrench must be calibrated with a given air-supply condition. The wrench should be calibrated using the same compressor and pressure settings, air hose, and air hose length that will be used on the work. If an additional wrench is to be driven off the compressor, the wrench calibration should be checked with both wrenches in operation simultaneously as well as individually. If a significant length of hose from compressor to wrench is either added or removed, then the wrench should be recalibrated. Calibration of the wrench is required every day, before installation begins, with three fastener assemblies of each diameter, length, grade, and lot.

An assembly would be comprised of a bolt from a specific production lot, with a nut from a specific production lot, and the washers under the turned element. Washers representative of those being used in the work must be included in the test, but lot control for washers is not mandated by the RCSC Specification. Lot control for washers is required for bridge work.

Electric and hydraulic torque control wrenches require the same daily calibration, but may provide more consistent results because of the consistency of electric power or hydraulic pressure, compared to air supply.

If there is a significant difference in the quality of fastener lubrication, then the wrench must be calibrated for the varying lubrication conditions. A well-oiled bolt, washer, and nut assembly will require considerably less torque than one that is nearly dry or one that exhibits some indications of rust. Hence, if the wrench is calibrated using well-oiled bolts, then used on a poorly lubricated bolt, the resultant bolt pretension will be less. The same concerns apply if the bolt, nut or washer surfaces hold dirt, grit, or sand.

Snugging the joint can be done with either the calibrated wrench (actually in the uncalibrated condition, releasing the trigger when snug is achieved), with a separate wrench for snugging, or with a hand wrench for lighter framing. After the wrench is calibrated, pretensioning can begin. The wrench operator should tighten the bolts using a systematic pattern, observing the chuck rotation as tightening proceeds. If the rotation of the nut exceeds the rotation table for turn-of-nut, the wrench calibration should be rechecked. After tightening all the bolts in the pattern, the operator should return to "touch up" each bolt in the pattern. Only the calibrated wrench method calls for such "touching up."

The inspector is to observe the preinstallation verification testing required in RCSC Specification Section 8.2.2. Subsequently, routine observation is used to ensure that the bolting crew properly snugs the joint, and then properly applies the calibrated wrench to the assembly.

"Torque and Rotation" Method. The "torque and rotation" method is not addressed in the 2014 RCSC Specification, but is used for ASTM F3111 grade 2 and ASTM F3148 assemblies. In each case, the installation method is included in the ASTM standard, until such time as the RCSC Specification is updated to include these fastener assemblies. The method is similar to those used in Europe and Japan.

The first step is to snug the joint, and then apply an initial level of torque. This process provides a consistent starting point for pretensioning, without the variations possible with the uncontrolled tightening during snug-tightening with the other methods. After the torque is applied, then a specific rotation is applied, but using values determined and supplied by the fastener assembly manufacturer, or determined by testing in a bolt tension calibrator. The turns applied are typically different than those provided in the turn-of-nut method table in the RCSC Specification.

As an example, for the ASTM F3111 grade 2 heavy hex 200-ksi bolt assembly, the torque and rotation are as provided in Table 8.6.

TABLE 8.6 Installation of ASTM F3111 Grade 2, 200-ksi Heavy Hex Assemblies

Bolt diameter, in	Initial torque at snug		Rotation from snug
	ft-lbf	N-m	
1	400, +200, −100	540, +270, −135	½ turn (180°)* +60°, −30°
1-⅛	600, +300, −100	815, +405, −135	
1-¼	900, +300, −100	1220, +405, −135	

*Increase rotation by ⅙ turn (60°) when one sloping surface less than 1:20 is under either the bolt head or nut, and no compensating washer is used.

The inspector is to observe the preinstallation verification testing of the assembly, as prescribed for the specific product. Subsequently, routine observation is used to ensure that the bolting crew brings the joint to the snug-tight condition, applies the prescribed initial torque, and then applies the proper rotation to the assembly.

8.5.4 Inspection after Bolting

After the conclusion of bolt installation, whether a snug-tightened joint, a pretensioned joint, or a slip-critical joint, there are no inspection tasks other than documenting the completion of the work, noting whether or not the work was done in accordance with the RCSC Specification and the project specifications.

For many years, industry practice was to verify bolt tension using a torque wrench, checking a percentage of bolts to determine whether or not the nut would turn at a given torque. The practice relied upon torque, rather than tension, and gave inconsistent results. Because of the unreliability of the torque-tension relationship, it was possible that bolts below the prescribed pretension would be accepted, and those above the required pretension would be rejected. The high level of unreliability is because half or more of the torque applied is used to overcome the friction between nut face and washer or steel, and nearly one-third the torque applied is used to overcome the friction between bolt and nut threads. With only roughly 10% of the torque directly related to the bolt pretension, any variation in lubrication or other installed condition will lead to widely varying results. The RCSC Specification removed tabulated torque values for installation and inspection in 1954, and specified observation for inspection purposes beginning in 1962.

Reuse of Bolts Previously Tightened. Occasionally, it may be necessary to remove a previously tightened bolt and later reinstall it. The RCSC Specification permits reuse of black (uncoated) ASTM A325 bolts only with the engineer's permission. ASTM A490 bolts, galvanized bolts and other coated bolts cannot be reused in any case.

A bolt that has been installed to the snug condition, but subsequently loosens when adjacent bolts are snugged, is not considered a reused bolt. Similarly, bolts that are touched up, or further tightened, in the pretensioning process are not considered reused. To be considered as reused, the bolt must have been pretensioned, then loosened.

ASTM A325 bolts that have been installed only to the snug condition, then removed, can generally be reused. ASTM A490 bolts, twist-off bolts and coated ASTM A325 bolts should be considered for reuse only if snugged by hand wrench or if very lightly snugged with a power wrench, far less than the effort or torque needed to pretension the bolt and create inelastic stretch in the bolt threads.

To check previously snugged bolts and previously pretensioned black ASTM A325 bolts to determine whether they can be reused, run the nut up the entire length of the bolt threads by hand. If this is possible, the bolt may be reused. Bolts that have yielded from tightening will stretch in the first few threads nearest the bolt shank, preventing the nut from progressing further up the threads. These bolts are not to be reused.

Because of the overtapping of the nut threads for galvanized and other coated fasteners, this check is not valid for the bolts. ASTM A490 bolts do not have the same ductility as ASTM A325 bolts, therefore ASTM A490 bolts may not be reused.

Arbitration of Disputes. Arbitration of disputes is not a substitute for inspection (the visual observation of the preinstallation testing, checking for snug, and observation of the installation technique used by the installers). Disputes may arise when observation indicates that the proper techniques have not been followed by the installers. The dispute must be resolved shortly after installation and pretensioning, as delays in performing torque-based arbitration will result in widely varying test results that renders the results of the torque-based procedure of limited value, and it is more difficult to locate fasteners representative of the installed fastener assemblies.

RCSC Specification arbitration of disputes methods may be applied to slip-critical joints (whether statically loaded or loaded in fatigue), and to specific types of pretensioned joints, as follows:

- Those carrying load in combined shear and tension using AISC Group B or C bolts
- Those carrying tension-only loading using AISC Group B or C bolts
- Those carrying tension-only loading when the joint is designed for fatigue conditions

For bridge work, torque testing is still commonly required for 10% of the bolts in each connection, minimum two per connection, for bolts installed using turn-of-nut or calibrated wrench methods. The torque-testing procedures of AASHTO are similar to the RCSC Specification arbitration procedures, except that only three bolts are used to determine the inspection torque, not five bolts for the arbitration torque. For direct tension indicator, twist-off bolt, and lock pin and collar pretensioning methods, no torque testing is required.

8.6 INSPECTION OF WELDED CONNECTIONS

The responsibilities and levels of welding inspection must be established in the contract documents. Neither AWS nor AISC, nor the model building codes, provide a complete listing of all welding inspection duties.

In accordance with AWS D1.1, the fabricator and erector is to perform all inspection duties, termed “contractor’s inspection.” Third-party welding inspection by an individual or firm that reports to the owner or engineer, termed “verification inspection,” is not required unless invoked by the owner.

In accordance with the AISC Specification, the fabricator and erector is to perform all inspection duties, termed “quality control” inspection. Third-party welding inspection by an individual or firm that reports to the owner or engineer, termed “quality assurance” inspection, is to be performed when required by the authority having jurisdiction (AHJ), applicable building code (ABC), purchaser, owner, or engineer of record (EOR).

The level of welding inspection varies according to the type of structure, such as risk categories listed in the model building codes, the structural system or systems used, the level of fatigue or seismic demand anticipated, and the desired structural performance in such events or loadings. Other considerations may include the certification and experience of the fabricator and the erector, as well as the qualifications and experience of their welding personnel and inspection personnel.

Welding inspection is commonly broken into three timing categories: before welding, during welding and after welding. The “before welding” can be further categorized into “advance inspection,” checking preparation and documentation well before welding begins, and “prior to welding” just before the production welds are made.

The AISC Specification provides welding inspection tasks in Tables N5.4-1 through N5.4-3, and the AISC Seismic Provisions provides welding inspection tasks in Tables J6-1 through J6-3. Most tasks are addressed in some form within AWS D1.1, but there are differences. Tables 8.7 to 8.9 that describe welding inspection prior to welding, during welding and after welding are derived from the AISC Specification tables, and incorporate references from the

Commentary.

TABLE 8.7 Welding Inspection Tasks prior to Welding

	QC	QA
Welder qualification records and continuity records (AWS Clauses 6.4 and 6.5.5, AISC Section N3.2)	P	O
Welding procedure specifications (WPSs) available (AWS Clause 6.3, AISC Section N3.2)	P	P
Manufacturer certifications for welding consumables available (AWS Clause 6.2, AISC Section N3.2)	P	P
Material identification (type/grade) (AWS Clause 6.2)	O	O
Welder identification system* (AWS Clause 6.4)	O	O
Fit-up of groove welds (including joint geometry)	O	O
• Joint preparations (AWS Clause 6.5.2) • Dimensions (alignment, root opening, root face, bevel) (AWS Clause 5.21) • Cleanliness (condition of steel surfaces) (AWS Clause 5.14) • Tacking (tack weld quality and location) (AWS Clause 5.17) • Backing type and fit (if applicable) (AWS Clauses 5.9 and 5.21.1.1)		
Fit-up of CJP groove welds of HSS T, Y, and K joints without backing (including joint geometry) (AWS Clause 9.11.2)	P	O
• Joint preparations • Dimensions (alignment, root opening, root face, bevel) • Cleanliness (condition of steel surfaces) (AWS Clause 5.14) • Tacking (tack weld quality and location) (AWS Clause 5.17)		
Configuration and finish of access holes (AWS Clauses 6.5.2 and 5.16, AISC Section J1.6)	O	O
Fit-up of fillet welds	O	O
• Dimensions (alignment, gaps at root) (AWS Clause 5.21.1) • Cleanliness (condition of steel surfaces) (AWS Clause 5.14) • Tacking (tack weld quality and location) (AWS Clause 5.17)		
Check welding equipment (AWS Clauses 6.2 and 5.10)	O	—

*The fabricator or erector, as applicable, shall maintain a system by which a welder who has welded a joint or member can be identified. Stamps, if used, shall be the low-stress type.

TABLE 8.8 Welding Inspection Tasks during Welding

	QC	QA
Control and handling of welding consumables (AWS Clause 6.2)	O	O
• Packaging (AWS Clause 5.3.1) • Exposure control (AWS Clause 5.3.2 for SMAW and Clause 5.3.3 for SAW)		
No welding over cracked tack welds (AWS Clause 5.17)	O	O
Environmental conditions	O	O
• Wind speed within limits (AWS Clause 5.11.1) • Precipitation and temperature (AWS Clause 5.11.2)		
WPS followed (AWS Clauses 6.3.3, 6.5.2, 5.5, and 5.20)	O	O
• Settings on welding equipment • Travel speed • Selected welding materials • Shielding gas type/flow rate • Preheat applied (AWS Clauses 5.6 and 5.7) • Interpass temperature maintained (min./max.) • Proper position (F, V, H, OH)		
Welding techniques (AWS Clauses 6.5.2, 6.5.3, and 5.23)	O	O
• Interpass and final cleaning (AWS Clause 5.29.1) • Each pass within profile limitations • Each pass meets quality requirements		
Placement and installation of steel headed stud anchors (AWS Clause 7)	P	P

TABLE 8.9 Welding Inspection Tasks after Welding

	QC	QA
Welds cleaned (Clause 5.29.2)	O	O
Size, length, and location of welds (AWS Clause 6.5.1)	P	P
Welds meet visual acceptance criteria (AWS Clauses 6.5.3 and 9.25)	P	P
• Crack prohibition [AWS Table 6.1(1) and Table 9.16(1)] • Weld/base-metal fusion [AWS Table 6.1(2) and Table 9.16(2)] • Crater cross section [AWS Table 6.1(3) and Table 9.16(3)] • Weld profiles [AWS Table 6.1(4) and Table 9.16(4), and Clause 5.23)] • Weld size [AWS Table 6.1(6) and Table 9.16(6)] • Undercut [AWS Table 6.1(7) and Table 9.16(7)] • Porosity [AWS Table 6.1(8) and Table 9.16(8)]		
Arc strikes (AWS Clause 5.28)	P	P
k-area* (AISC Specification Commentary Section A3.1c and J10.8.)	P	P
Weld access holes in rolled heavy shapes and built-up heavy shapes† (AWS Clauses 6.5.2 and 5.16, AISC Section J1.6)	P	P
Backing removed and weld tabs removed (if required) (AWS Clauses 5.9 and 5.30)	P	P
Repair activities (AWS Clauses 6.5.3 and 5.25)	P	P
Document acceptance or rejection of welded joint or member (AWS Clauses 6.5.4 and 6.5.5)	P	P
No prohibited welds have been added without the approval of the engineer of record (AWS Clauses 5.17 and 6.5.1)	O	O

*When welding of doubler plates, continuity plates or stiffeners has been performed in the *k*-area, visually inspect the web *k*-area for cracks within 3 in (75 mm) of the weld.
†After rolled heavy shapes (AISC 360 Section A3.1c) and built-up heavy shapes (AISC Section A3.1d) are welded, visually inspect the weld access hole for cracks.

As is done for bolting inspection, welding inspection tasks are designated as “observe” or “perform.” Observe (O) means that the inspector observes these items on a random basis, and that operations need not be delayed pending these inspections. Perform (P) means that the inspector performs the task on each welded joint or member. In the *AISC Seismic Provisions*, for welds in the seismic force resisting system (SFRS), observation is to be performed on a random, *daily* basis.

Observe tasks are as described in AWS D1.1 Clauses 6.5.2 and 6.5.3. Clause 6.5.2 uses the term observe and also defines the frequency to be “at suitable intervals,” meaning not for each weld, rather as necessary to ensure that the applicable requirements of the code are met. “Suitable intervals” may depend upon the quality control program of the fabricator or erector, the skills and knowledge of the welders, the type of weld and importance of the weld. More inspection may be warranted for weld fit-up and monitoring for CJP and PJP groove welds loaded in transverse tension, and less inspection on groove welds loaded in compression or shear, or on fillet welds. More inspection may be warranted observing welding operations on multipass fillet welds, and less on single pass fillet welds. During the initial stages of a project, it is suggested to have higher levels of observation to establish the abilities and quality of the fabricator and erector and their personnel. These heightened levels of observation can then be reduced based upon experience.

8.6.1 Advance Inspection

Welding Personnel. Prior to the welding being performed, welding personnel (welders, tack welders, and welding operators) must be qualified by the fabricator or erector responsible for the welding, as required by AWS D1.1 Clause 4.2.2 and detailed in AWS D1.1 Clause 4 Part C.

Welders are individuals who manipulate the welding electrode or welding gun by hand to make the weld using manual or semi-automatic processes. A tack welder is a fitter who makes small welds as necessary to hold parts together until final welding of the joint is done by a welder or welding operator. A welding operator sets up and adjusts equipment to perform mechanized or automatic welding.

The fabricator or erector responsible for welding must have each welder, tack welder, and welding operator tested using the methods of AWS D1.1 Clause 4 Part C to prove their capability to make adequate quality welds. These individuals are tested and categorized by

- Welding process
- Welding position
- Electrode classification, if SMAW
- Base metal thickness range
- Weld type

The testing may be performed by the fabricator or erector or by an independent organization responsible to that fabricator or erector. Welding performance qualification records (WPQRs) must be completed and made available for the inspector's review prior to the start of welding. If a previous employer's testing results are to be used, then the engineer must approve the current employer's reliance upon these previous tests. Welders who perform and pass such testing at a testing facility accredited by the American Welding Society in accordance with AWS QC4 *Standard for Accreditation of Test Facilities for AWS Certified Welder Program* (AWS QC4-89) with AWS B5.4 *Specification for the Qualification of Welder Test Facilities* (AWS B5.4:2005) can have their test records placed on file with the AWS and receive the designation of AWS Certified Welder, under the provisions of AWS QC7 *Standard for AWS Certified Welders* (QC7-93).

A welder's or welding operator's qualification for a given employer remains in effect indefinitely, as long as that individual continues welding in that given process. Welders need not use the tested electrode classification, the tested position, or the tested weld type to retain their qualified status. If the welder fails to use that process for a period exceeding 6 months, the individual must complete and pass a welding test. However, if the welder's quality becomes subject to question, perhaps because of failing to maintain the skills for more difficult positions or more difficult welds, the welder's qualification may be revoked, forcing a retest for those welds. Tack welders' qualifications remain in effect perpetually, unless there is specific reason to question the tack welder's abilities. Although welder qualification is the responsibility of the contractor, under the provisions of AWS D1.1 Clause 6.4.2, the inspector may also force requalification testing if the welder's quality is poor.

Welding Equipment. In order to properly follow the essential variables of a welding procedure specification (WPS), the equipment used for welding must be in good repair and able to provide the output needed to weld. The inspector should check the maintenance and testing records of the equipment to be employed, and if necessary, use electrical testing equipment to verify that the equipment settings and the welding machine output are adequate and within the limits of the WPS.

AWS D1.1 Subclause 6.2 assigns this task specifically to the contractor's inspector, and not to the verification inspector, and the AISC Specification reflects this assignment.

Welding Procedures. The use of written, established WPSs is mandated by AWS D1.1 Clause 5.5. The WPS may be either prequalified or qualified by test. A prequalified WPS must fall within the limits prescribed in Clause 3 of AWS D1.1, in particular Table 3.6, must use a welding process listed that table, and must use a joint detail or fillet size given in Clause 3. All other WPSs must be qualified by test using the requirements set forth in AWS D1.1 Clause 4. Annex M of AWS D1.1 provides example procedure qualification record (PQR) and WPS forms.

WPSs are written by the fabricator or erector, often aided by filler metal suppliers or welding equipment suppliers, technical organizations, or consultants. WPSs are specific to the following parameters:

- Welding process
- Base metal (steel classification, strength, type)
- Base metal thickness (range)
- Electrode classifications
- Flux classifications
- Shielding gases
- Joint type (butt, tee, corner)
- Weld type (groove, fillet, plug)
- Joint design details (root opening, groove angle, root face, use of backing)
- Use of backgouging
- Position (flat, horizontal, vertical, overhead, tubular)

Using the above parameters, the following variables, with others as needed for a particular process or procedure, are established:

- Number and position of passes
- Electrode diameter
- Polarity
- Current or wire feed speed, or both
- Travel speed
- Voltage
- Technique
- Shielding gas (if used) flow rate

- Preheat, interpass, and postheat requirements
- Cleaning requirements

AWS D1.1 Clause 6.3 states that the contractor's inspector is responsible for verification of the WPS, whether prequalified or qualified by test, and that welding is done in conformance to the WPS. The verification inspector is not responsible for these duties. However, AISC Specification Chapter N includes inspection tasks for the QA inspector for these verifications.

8.6.2 Inspection prior to Welding

Base Metal Quality. The quality of the base metal is to be suitable for welding. The steel to be welded must be clean and smooth, and without surface discontinuities such as tears, cracks, fins, and seams. Such surface discontinuities could propagate into the weld in the form of cracks. The surface should also be free of excessive rust, mill scale, slag, moisture, grease, oil, and any other material that could cause welding problems to the extent that the weld quality requirements of the code could not be achieved. Some materials may be permitted, such as thin mill scale (mill scale that withstands a vigorous wire brushing), thin rust-inhibitive coatings, and anti-spatter compounds made specifically for weld-through applications. AWS D1.1 Clause 5.14 provides additional information and exceptions to these provisions.

Joint Preparation and Fit-Up. Fillet weld fit-up tolerances are given in AWS Clause 5.21.1. Gaps of 1/16 in (1.6 mm) or less between parts are permitted without correction. If the gap exceeds 1/16 in (1.6 mm) but does not exceed 3/16 in (5 mm), then the leg dimensions of the fillet weld should be increased to compensate for the gap between the parts. Gaps over 3/16 in (5 mm) are permitted only with thick materials over 3 in (76 mm). In these cases, the use of a backing material at the root is required as well as compensation in the weld leg dimensions. Such provisions cannot be used for gaps over 5/16 in (8 mm). Similar provisions are used for PJP groove welds when the welds are parallel to the length of the member.

When groove welds are used, tolerances to the root opening, groove angle, and root face apply. The specific tolerances depend upon the type of groove weld, the presence of backing, and the use of backgouging. AWS D1.1 Clause 5.21.4 and AWS D1.1 Figure 5.3 provide these values. Groove tolerances for nontubular connections are also provided in AWS D1.1 Figure 3.2 for prequalified PJP groove welds and Figure 3.3 for prequalified CJP groove welds. For tubular joints, refer to AWS D1.1 Clause 9, which includes Table 9.8 and Figs. 9.10 to 9.12.

Alignment of parts at butt joints can be critical, depending upon application. AWS D1.1 Clause 5.21.3 requires alignment within 10% of the part thickness, not to exceed ⅛ in (3 mm), when the parts are restrained from bending from such misalignment. No provisions are given for cases where such restraint does not exist. For girth welds in tubular joints, the alignment tolerances are provided in AWS D1.1 Clause 9.24.1.

The fit-up for groove welds and fillet welds prior to welding should be checked by the tack welder, welder or welding operator before beginning welding. Joint dimensions such as groove angle, root opening and root face should be shown on the WPS. The inspector should verify through observation that a prewelding fit-up check is done consistently and correctly by welding personnel, using proper tools and methods, and perform check measurements on a random basis.

Welding Consumables. Welding electrodes, fluxes, and shielding gases should be checked to be in conformance with AWS D1.1 Clause 5.3. Low hydrogen SMAW electrodes require inspection and monitoring, including requirements for baking and storage temperatures and exposure time limits. Fluxes for SAW require dry, contamination-free storage, with the removal of the top 1 in (25 mm) of material from previously opened bags prior to use. Drying of flux from damaged bags may be required.

Welding materials such as electrodes, fluxes, and shielding gases are to have manufacturers' certificates of compliance stating that they meet applicable American Welding Society A5-series filler metal standard. In accordance with AWS D1.1, these certificates of compliance may be requested by the owner or inspector. However, the AISC Specification Section N3.2 requires certificates of compliance to be available.

AWS D1.1 Subclause 6.2 assigns the responsibility to the contractor's inspector to ensure that materials conforming to the requirements of the code are used. However, the AISC Specification assigns inspection tasks for materials to both the QC and QA inspectors.

Welding Conditions. The welder and the welding equipment must have conditions suitable for welding. The environmental conditions for welding must be adequate, and limits are given in AWS D1.1 Clause 5.11. The temperature of the area immediately surrounding the welding must be above 0°F (–18°C). The temperature in the general vicinity may be lower, but heating must be provided to raise the temperature immediately around the weld to at least this temperature. The surfaces to be welded must not be wet or exposed to moisture. High winds must be avoided. For GMAW, GTAW, EGW, and gas-shielded FCAW, the wind speed must not exceed 5 mi/h (8 km/h), requiring protective enclosures in most field applications. For seismic welding, this wind speed value is reduced to 3 mi/h (5 km/h) to minimize small porosity, otherwise deemed acceptable, that reduces the notch toughness of the weld. No maximum wind speed is specified for welding processes requiring no shielding gases, but a practical limit is generally around 20 mi/h (35 km/h).

Preheat. Preheating of the steel is necessary for thick steels, certain high-strength steels, and steels when their temperature is below 32°F (0°C). The preheating requirements should appear in the welding procedure specification (WPS). AWS minimum prequalified preheat requirements are provided in AWS D1.1 Table 3.3. In this table, the minimum preheat is given for a specified steel specification, welding process and/or filler metal classification, and thickness of base metal. When the temperature of the steel is below 32°F (0°C), the steel must be heated to at least 70°F (21°C). The thicker the steel, the higher the required preheat temperature. Higher-strength steels also require higher preheats. Certain high-strength steels listed in AWS D1.1 Table 4.9 have preheats limited to a maximum of 400°F (200°C) or 450°F (230°C), depending upon thickness. Preheat requirements may also be modified using the provisions of AWS D1.1 Annex H, which evaluates the welding filler metal diffusible hydrogen, joint restraint, and the weldability (carbon equivalency) of the steel, but the use of lower temperatures than those stated in AWS D1.1 Table 3.3 requires qualification testing.

8.6.3 Inspection during Welding

After checking the welder qualifications, WPS, welding consumables, steel materials, welding conditions, equipment, joint fit-up, and preheat, the welding inspection performed during welding is to verify that the WPS is properly followed.

This includes the maintenance of interpass temperature during welding, usually the same temperature required for preheat. Each pass should be thoroughly cleaned and visually inspected by the welder, with the inspector verifying through random observation that the welder is performing this task. Small tack welds may crack during the shrinkage and distortion that takes place during root pass welding, and should be monitored to ensure this does not take place. Tack welds should be of adequate size, length and quality to ensure this does not take place. Control of electrodes, especially low hydrogen SMAW electrodes, must be maintained. In some cases, nondestructive testing may be performed at various stages during welding.

In the case of automatic stud welding, the inspector should verify the suitability of stud welding materials, base metal welding conditions, and that the welding operator is qualified, preproduction testing is performed, placement is correct, the WPS is followed, and any WPS adjustments, including stud gun adjustment beyond those permitted by AWS D1.1, are verified using preproduction test methods. After stud welding, visual inspection is performed of the stud weld, relying upon the presence of flash around the entire stud base, and bend testing is performed as required. Studs welded manually rather than by stud gun should be inspected in a manner similar to fillet welds, with verification prior to welding that the stud base has been properly prepared for fillet welding.

8.6.4 Inspection after Welding

After weld completion, visual inspection of all welds is to be performed to verify they satisfy the applicable visual weld quality criteria. All welds are to have their size, length, and location measured to verify they meet the project requirements.

The "*k*-area" is a portion of the web of a rolled shape that has been rotary straightened, and may have low notch-toughness in that area as a result of the straightening. Welds placed in the *k*-area under high restraint, such as doubler plates, continuity plates and stiffeners, may cause base metal cracks to form in the web from a combination of shrinkage stress, restraint, weld termination conditions and low base metal notch-toughness. Previous versions of the AISC Specification and AISC Seismic Provisions specified MT for this condition, but effective with the 2016 AISC standards, this has been changed to a visual inspection.

Although thermally cut weld access holes are examined prior to welding for size, profile, and freedom from cracks, it is possible that high welding restraint and shrinkage may cause cracks in the weld access holes after welding has been completed. Therefore, a final visual check of the weld access hole is to be performed to ensure it is crack free.

Arc strikes are to be removed, and the base metal surface repaired and inspected. At certain locations such as members subject to fatigue and protected zones of seismic connections and members, inadvertent welds or temporary welds may adversely affect performance of the member or connection. The member or connection should be examined when such conditions exist to ensure that no welds have been added to such locations. Similarly, such details and others noted in the contract documents may require the removal of backing and/or weld tabs, and it should be verified that this removal, and perhaps surface improvement of the removal area, has been completed in accordance with the applicable standard or project specification.

After visual welding inspection is completed, nondestructive testing of the completed weld, if required by the contract documents, is then performed. For both final visual inspection and final NDT, a delay period may be needed because of the risk of delayed hydrogen-assisted cracking with susceptible steels, high restraint and high levels of diffusible hydrogen produced while welding. If repairs are required, the repair work should be subjected to reinspection and the same NDT as was used for the original weld.

The inspector responsible for inspection of the completed weld should place an identifying mark near the weld, or use another acceptable method, for identifying the welds inspected and their acceptance or rejection.

8.6.5 Nondestructive Testing

NDT Methods. Several methods of nondestructive testing (NDT), also called nondestructive examination (NDE), may be used on a structural steel project.

The first common form of NDT is *visual testing* (VT), although this is termed visual inspection in the AWS D1.1 codes. Most visual inspection is performed without the use of magnifiers. Magnifying glasses may be used to more closely examine areas that are suspected of cracks and other small, but potentially significant discontinuities. Adequate light and good visual acuity is necessary. Various weld gages are used to determine weld size, convexity, undercut, reinforcement, and other measurements as needed.

An enhanced form of visual inspection is *penetrant testing* (PT). The weld surface and surrounding steel is thoroughly cleaned. A penetrating liquid dye is applied to the weld surface and allowed time to penetrate cracks, pores, and other surface discontinuities. After an allotted time (dwell time), the penetrant is removed and a developer is applied to the surface. The developer draws the penetrant back to the surface of the weld and base metal. The developer is of a color (often white) that contrasts with the color of the dye in the penetrant. The inspector observes the dye in the developer, then removes the developer and dye to more closely inspect the weld surface visually. Some penetrant testing uses an ultraviolet solution, rather than a dye, to aid in visibility when a UV lamp is available. Penetrant testing can detect surface discontinuities only. Permanent records of discovered defects are typically done using photography.

Magnetic particle testing (MT) can be used to detect surface and slightly subsurface discontinuities. The general limit to the depth of examination is approximately ⅛ in (3 mm) when using a typical yoke. A magnetic field on and near the surface of the steel is induced into the region of the weld through the use of the yoke and power supply. Fine magnetic particles, typically iron with color added, are then applied to the surface of the steel. These particles may be in the form of a dry powder or may be in a liquid emulsion.

When cracks or other discontinuities are on or near the surface, the flux lines generated by the current are interrupted, creating two new magnetic poles in the steel or weld that attract and hold the particles in place while the particles away from high magnetic attraction are blown from the surface. The inspector then observes and interprets the position and nature of the remaining particles, judging them to indicate a crack or other discontinuity on the surface or subsurface. False indications may appear at weld toes and at transitions, and require further evaluation to verify that cracks are nor present.

For best performance, the flux lines must flow approximately perpendicular to the discontinuity. Therefore, the MT technician must rotate the yoke at approximately 90° angles along the length of the weld to inspect it for both longitudinal and transverse discontinuities. Permanent records of discovered defects is typically done using photography.

Ultrasonic testing (UT) is the preferred method of NDT for detection of subsurface discontinuities. It is capable of testing weldments from approximately 5/16 to 8 in (8 to 200 mm) in thickness using the standard technique, calibration, and acceptance provisions of AWS D1.1. The most common method of testing uses a pulse-echo mode similar to radar or sonar. The control unit sends electronic signals into a transducer made of piezoelectric material. The electrical energy is transformed by the transducer into vibration energy. The vibration is transmitted into the weldment through a coupling liquid. The vibration carries through the weldment until a discontinuity or other interruption, such as an edge or end of the material, disrupts the vibration. The disruption causes the vibration to reflect the ultrasound wave back toward the transducer. The reflected vibration is then converted back into electrical energy by the piezoelectric material, sending a signal to the display unit. The return signal's configuration, strength, and time delay are then interpreted by the testing technician.

The interpretation by the technician uses the characteristics of the response shown on the display unit to determine the type of discontinuity, and manipulates the transducer in various patterns to determine a better understanding of the location, length, depth, orientation, and nature of the discontinuity. The strength of the returning vibration, indicated by the height on the display, in combination with the length of the discontinuity as determined by manipulation of the transducer, determines the acceptance of the weld with the discontinuities found by the UT. The location of the returning vibration on the display unit is used to determine the distance from the transducer to the discontinuity.

AWS D1.1 Table 6.7 prescribes the testing procedures for butt, tee, and corner joints of various thicknesses. The search angle and joint faces to be used are given. Annex Q of AWS D1.1 gives alternative techniques for ultrasonic testing and the evaluation of weld discontinuities.

More advanced methods of UT are gaining acceptance in codes and standards, including the use of automated UT (AUT), in which the location of transducer is controlled and recorded, along with the response in real time, providing a consistent and recordable evaluation of the weld. Phased array UT (PAUT) uses advanced transducers that scan at multiple angles, with real-time recording of results, providing examination of the weld that is much more thorough than conventional UT and AUT, but the acceptance criteria for PAUT results has not been generally resolved for code use at the time of this publication, and is generally limited to engineering assessment through fracture mechanics.

UT is best suited for planar flaws, such as cracks and incomplete fusion, and is less sensitive to volumetric flaws such as porosity and slag inclusions. Difficulties may also arise in interpreting results when backing remains in place, or when PJP groove welds are examined, so special procedures and techniques must be used in these instances.

Radiographic testing (RT) is another method of NDT for welds in structural steel. RT is performed, using either x-rays or gamma rays, sending energy into the steel weldment. Film is placed on the side of the weldment opposite the energy source. The steel and weld metal absorb energy, reducing the exposure of the film, but volumetric weld discontinuities allow more energy to reach the film, producing a darkened area on the film to be interpreted by the radiographer.

Radiographic testing is effective in steels up to about 9 in (230 mm) in thickness. Exposure time and film selection are varied according to conditions and thicknesses. Image quality indicators (IQIs), either wire-type or a hole penetrameter, are used to verify the sharpness and sensitivity of the film image, as well as to provide a measurement scale on the exposed film.

RT is best suited for locating volumetric discontinuities such as slag inclusions and porosity, and may miss planar flaws such as cracks and incomplete fusion unless they are oriented perpendicular to the energy and film, and are of significant enough depth to be seen on the film, Because of this limitation and the radiation exposure hazards, radiographic testing is typically the most expensive of the methods previously mentioned, and is uncommon in steel structures today. Hence, the preference is for UT for examining weldments for subsurface flaws.

Specification of NDT. For building structures, the AISC Specification, Chapter N, cites specific welded connections that are required to receive NDT, generally ultrasonic testing. These include

1. For structures in risk category III or IV, the highest risk levels defined by the applicable building code, all CJP groove welds that are subject to transversely applied tension loading in butt, T- and corner joints, in materials 5/16 in (8 mm) thick or greater. CJP groove welds loaded in shear or in compression are not included.
2. For structures in risk category II, as defined by the applicable building code, 10% of CJP groove welds in butt, T- and corner joints subject to transversely applied tension loading, in materials 5/16 in (8 mm) thick or greater. The 10% applies to the number of welds, to be selected at random, and is not to be interpreted as 10% of the length of all welds. CJP groove welds loaded in shear or in compression are not included.
3. For fatigue applications, when required by AISC Specification Appendix 3, Table A-3.1, CJP groove welds in butt joints requiring weld soundness to be established by radiographic or ultrasonic inspection are subjected to either RT or UT, as prescribed. Reduction in the rate of UT is prohibited.

No UT is required for welds below 5/16 in (8 mm) in thickness, PJP groove welds, fillet welds, or for welds in structures in risk category I, the lowest level of risk defined in the applicable building code.

In addition to the above three requirements for buildings as given in the AISC Specification Chapter N, addition NDT is required for welded joints if they are a part of a seismic force resisting system (SFRS). These NDT requirements are defined in the AISC Seismic Provisions, Chapter J, and include magnetic particle testing (MT), penetrant testing

(PT), and/or ultrasonic testing (UT) for certain joints, as follows:

1. UT is to be performed of 100% of all CJP groove welds in materials 5/16 in (8 mm) thick or greater, regardless of direction of loading. In addition, MT is to be performed on 25% of all beam-to-column CJP groove welds. For ordinary moment frames in structures in risk categories I and II, UT and MT of CJP groove welds is required only at demand critical welds.
2. To detect lamellar tearing in T and corner joints where the base metal loaded in tension in the through-thickness direction is thicker than 1-½ in (38 mm), and the connecting material is greater than ¾ in (19 mm) and contains CJP groove welds, UT after joint completion is performed to check for discontinuities and lamellar tears behind and adjacent to the fusion line of the welds. Weld acceptance criteria found in AWS D1.1 Table 6.2 is used for any base metal discontinuities found within the top quarter-thickness of the steel surface subjected to the through-thickness strain.
3. At welded splices and at members with beam copes and weld access holes that are thermally cut, when the member flange thickness exceeds 1-½ in (38 mm) if a rolled shape or when the member web thickness exceeds 1-½ in (38 mm) if a built-up shape, MT or PT is to be performed to verify that the cut surface is free of cracks.
4. If the thermal cut edge of a reduced beam section has had a sharp notch removed by grinding or a gouge repaired using welding, MT of the repair area is to be performed.
5. At beam-to-column weld tab removal sites, MT is to be performed on the same joints receiving UT as described in (1) above.
6. At column splices and column to base plate PJP groove welds, UT is to be performed. A new column splice detail using PJP groove welds, with minimal root face and a lower column shaft thicker than the upper shaft, was added in 2016 to the AISC Seismic Provisions for special moment frames, and may be used for other systems as well. Because the weld is demand critical and subject to inelastic strains, UT requirements for PJP welds in such column splices were added in 2016 to address this particular detail. Because such PJP groove welds can be subjected to false rejection from indications created by the root face, special UT procedures must be written and qualified by sample testing, and the UT technicians must be similarly qualified for their ability to test this particular detail. It is not intended that other PJP groove welds used in column splices that are not a part of the SFRS would be subjected to UT.

UT and MT rates may be reduced, where permitted, when it has been determined than an individual welder has a rejection rate of 5% or lower. However, welders with rejection rates higher than 5% have the rate of UT increased to 100% until such time that the welder's rejection rate is deemed acceptable.

Unlike the requirements of the AISC Specification and the AISC Seismic Provisions, for statically loaded structures, AWS D1.1 does not require any specific nondestructive testing (NDT), leaving it to the engineer to determine the appropriate NDT method(s), locations or categories of welds to be tested, and the frequency and type of testing (full, partial or spot), in accordance with AWS D1.1 Clause 6.15.

8.6.6 Weld Acceptance Criteria

The acceptance criteria to be used for weld quality is to be established by the engineer. Commonly, the quality and inspection criteria found in AWS D1.1 Clauses 5 and 6 are adopted. However, the use of alternate criteria is both accepted and encouraged. AWS D1.1 Clause 6.8 states that "The fundamental premise of the Code is to provide general stipulations applicable to most situations. Acceptance criteria for production welds different from those specified in the Code may be used for a particular application, provided they are suitably documented by the proposer and approved by the Engineer." The commentary to Clause 6.8 provides additional insights into the development and use of alternate acceptance criteria.

The visual weld acceptance criteria for welds is provided in AWS D1.1 Table 6.1 for nontubular joints and in Table 9.16 for tubular joints. Generally, cyclically loaded and tubular joints require higher standards of quality than statically loaded nontubular joints. These values also apply when penetrant testing (PT) and magnetic particle testing (MT) are used.

When ultrasonic testing is used, AWS D1.1 Table 6.2 is used for statically loaded nontubular connections and cyclically loaded nontubular connections in compression, and Table 6.3 is used for cyclically loaded nontubular connections in tension. For tubular connections, use AWS D1.1 Clause 9.27. The alternative techniques and criteria of AWS D1.1 Annex Q may be used when approved by the engineer.

When radiographic testing is used, AWS D1.1 Figure 6.1 is used for statically loaded nontubular connections and statically or cyclically loaded tubular connections, Figure 6.2 for cyclically loaded nontubular connections in tension, and Figure 6.3 for cyclically loaded nontubular connections in compression.

Because many of the acceptance criteria found in AWS D1.1 are based upon workmanship, meaning what a qualified welder can provide for weld quality, rather than the quality necessary for structural integrity, alternate acceptance criteria can be used to save both time and cost. In addition, repairs to some welds with innocuous discontinuities may result in more damage to the material in the form of additional discontinuities, lower toughness, larger heat-affected zones, more distortion, and higher residual stresses.

Alternative acceptance criteria have been published by several organizations in various forms. In the United States, the Electric Power Research Institute published *Visual Weld Acceptance Criteria, NP-5380*, for use in reinspections of statically loaded structural welds in existing nuclear power plant facilities, and has been accepted for use by the Nuclear Regulatory Commission. The document provides engineering-based weld acceptance criteria, based upon an assumed overstrength and conservative design, that allows fillet weld convexity and groove weld reinforcement without height limitations, more undercut and more porosity than AWS D1.1 criteria, and new criteria for acceptance of limited amounts of underlength, mislocation of welds, and surface slag.

When more advanced nondestructive test methods are available for investigation of flaws beneath the surface, *API 579-1/ASME FFS-1-2007 Fitness-For-Service*, 2nd edition, an update to *API RP 579-2000 Recommended Practice for Fitness-for-Service*, provides detailed assessment methods for numerous conditions, including crack-like flaws and joint misalignment. Similarly, British standard *BS 7910:2013+A1:2015 Guide to methods for assessing the acceptability of flaws in metallic structures* provides several engineering-based assessment methods. The Welding Research Council has published several WRC Bulletins providing suggested criteria, and the International Institute of Welding has published several documents providing suggested acceptance criteria, with considerable research documentation justifying the criteria.

CHAPTER 9
STEEL DECK CONNECTIONS

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(*Courtesy of The Steel Institute of New York.*)

Fastening deck is an important design function which requires the attention of the design professional. Unlike structural steel, the fastening of steel deck has little or nothing to do with its fabrication so the deck supplier has no responsibility to choose the type of fastener or the spacing. However, the deck supplier, or the Steel Deck Institute (SDI), can aid the designer by providing information that can be helpful in the selection process.

For construction purposes the deck is almost always used as a working platform. It is, therefore, quite important that the deck be quickly and adequately attached as it is placed.

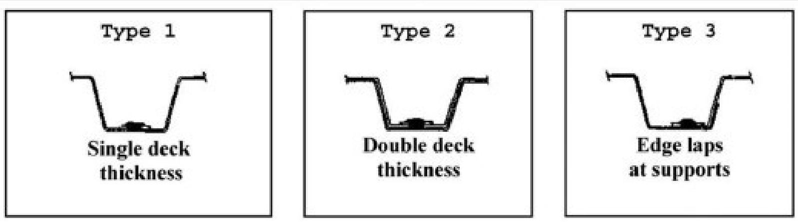
Additionally, the fastened deck acts to stabilize joists and brace beams. Although the construction process is usually not part of the design, safety of the working platform is obviously important.

The factors that most affect the fastening are the anticipated wind and earthquake loads. These cause both horizontal (diaphragm) and vertical (uplift) forces to be applied to the fasteners and, as a result, are of most interest to designers. In the case of wind, shear and tension interaction exists and interaction equations are provided in the SDI *Diaphragm Design Manual*, Fourth Edition. Some Underwriters Laboratories' fire-rated constructions also specify fastener types and spacing, and must be consulted.

Shear and uplift strengths are the parameters most needed. Table 9.1 shows the ultimate tensile strength of arc spot (puddle) welds through steel deck. Three types of welds are illustrated in Table 9.1: Type 1 is through a single deck thickness, Type 2 is at a deck end lap or through cellular deck and is through two thicknesses of steel, and Type 3 is at a deck edge (side) lap and its lower values are the result of the eccentric loading at the edge—a 0.7 multiplier is applied to the Type 1 value. An end member prying factor of 2 is also required in design for welds but this is only critical at single spans since an interior support tributary area is normally twice that at the end of the member. The minimum recommended diameter is ½ in (13 mm) but $^5/_8$ in (16 mm) is common.

TABLE 9.1 Tensile Strength of Arc Spot (Puddle) Welds

Arc Spot Weld Uplift Capacity



Case	Gage Number	Design Thickness (in)	ASD (lb) Ω = 2.50 Visible Weld Diameter (in) ½	⅝	¾	1	LRFD (lb) Φ = 0.60 Visible Weld Diameter (in) ½	⅝	¾	1
1	22	0.0295	347	439	531	716	520	659	797	1074
	20	0.0358	415	527	639	863	623	791	959	1294
	18	0.0474	536	684	833	1129	804	1027	1249	1693
	16	0.0598	658	845	1032	1406	987	1267	1548	2108
2	22	0.0295	650	835	1019	1388	976	1252	1529	2082
	20	0.0358	767	991	1214	1500	1150	1486	1822	2250
	18	0.0474	814	1257	1500	1500	1221	1885	2250	2250
	16	0.0598	549	1256	1500	1500	823	1884	2250	2250
3	22	0.0295	243	307	372	501	364	461	558	752
	20	0.0358	291	369	447	604	436	554	671	906
	18	0.0474	375	479	583	790	563	719	874	1185
	16	0.0598	461	591	722	984	691	887	1083	1476

Source: SDI Roof Deck Design Manual, 1st Edition

Table 9.2 shows the nominal (ultimate) shear strength of puddle welds. These are to be used for diaphragm loads. The resistance and safety factors for diaphragms are based on the

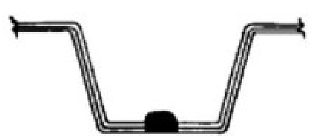
AISI *North American Specification for the Design of Cold-Formed Steel Structural Members, 2012 Edition*. Stress increase for temporary loading is not allowed. The diaphragm factors are system and repeating member factors. When isolated fasteners are used to resist shear, use the individual factors presented in the North American Specification. A quality control procedure is shown in Fig. 9.1.

TABLE 9.2 Weld Shear Strengths, lb (for Diaphragm Calculations)

		Arc Spot Weld Shear Capacity							
		ASD (lb) $\Omega = 2.55$ (2.20)				LRFD (lb) $\Phi = 0.60$ (0.70)			
		Visible Diameter (in)							
	Gage	0.5	0.625	0.75	1	0.5	0.625	0.75	1
1	22	718	909	961	1009	1099	1391	1470	1544
	20	860	1092	1324	1436	1316	1671	2025	2197
	18	(1215)	1417	1724	2337	(1871)	2168	2638	3576
	16	(1088)	(1898)	2137	2910	(1676)	(2923)	3269	4453
⅔	22	673	864	955	1003	1030	1322	1461	1535
	20	794	1026	1257	1427	1215	1569	1924	2184
	18	(694)	(1401)	1608	2221	(1068)	(2157)	2460	3398
	16	(468)	(1070)	(1919)	2725	(720)	(1648)	(2955)	4170

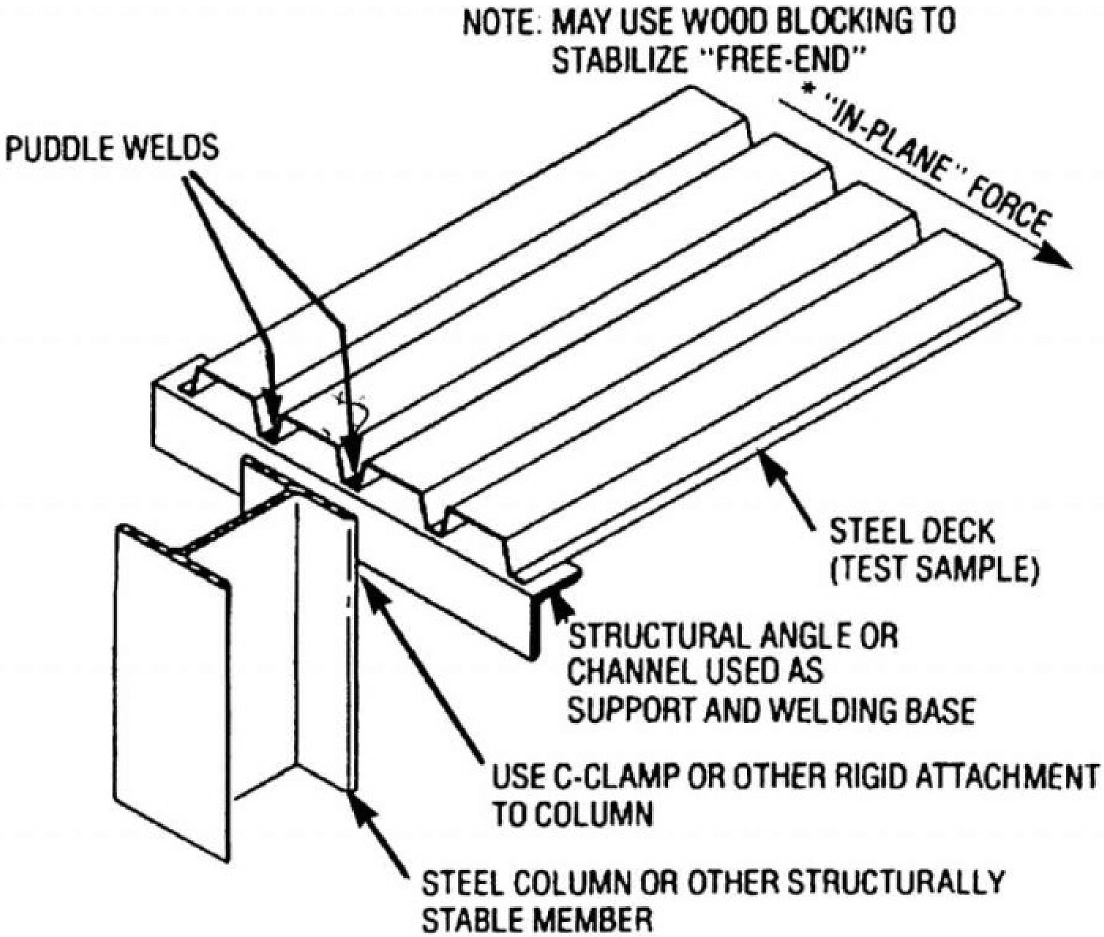
Notes:
1. Assumes $F_y = 40$ ksi and $F_u = 50$ ksi, which provides resistances that are less than that for $F_y = 33$ ksi steel.
2. Assumes $F_{xx} = 60$ ksi.
3. Information in Tables 9.1 and 9.2 is developed in accordance with Section E of AISI S100.

Case 1—Weld through single sheet thickness



Case 2—Weld through double sheet thickness

Case 3—Weld at edge of sheet sidelap

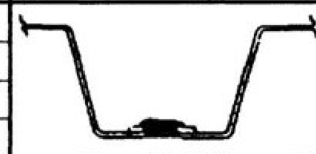
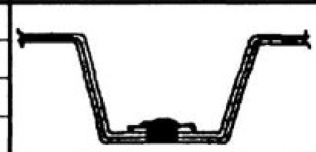
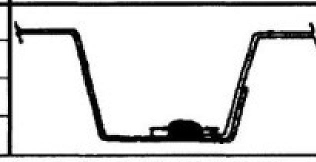


A preliminary check for welding machine settings and operator qualifications can be made through a simple field test by placing a pair of welds in adjacent valleys at one end of a panel. The opposite end of the panel can then be rotated, which places the welds in shear. Separation leaving no apparent external weld perimeter distresses, but occurring at the sheet-to-structure plane, may indicate insufficient welding time and poor fusion with the substrate. Failure around the external weld perimeter, showing distress within the panel but with the weld still attached to the substrate, would indicate a higher quality weld.

FIGURE 9.1 Weld quality control check. (*Courtesy of the Steel Deck Institute.*)

Weld washers are only recommended for attaching deck to the structural frame or bar joists when the deck steel is less than 0.028 in (0.71 mm) thick. The purpose of the weld washers is to provide a heat sink and keep the weld burn from consuming too much of the thin steel. The weld washer then forms a "head" on the weld button and provides the uplift and shear strengths as shown in Table 9.3. The 0.7 prying multiplier is applied at case 3. Common weld washers furnished by deck manufacturers are made of 16 gage material [0.057 in (1.44 mm)] and have a ⅜-in (10-mm)-diameter hole. The weld should slightly overfill the hole to produce a visible weld diameter of approximately ½ in (13 mm). Figure 9.2 shows the patterns and pattern nomenclature of deck-to-frame connections.

TABLE 9.3 Weld Washer Strengths

Case	Gage	Pn, Uplift (Tensile) Values, lbs[1]		Shear, lbs[2]	Profile
1	28	0.0149	1390	1200	
	26	0.0179	1430	1550	
	24	0.0239	1520	2350	
	23	0.0269	1560	2830	
2	28	0.0149	1590	1200	
	26	0.0179	1670	1550	
	24	0.0239	1840	2350	
	23	0.0269	1780	2830	
3	28	0.0149	960	1200	
	26	0.0179	990	1550	
	24	0.0239	1050	2350	
	23	0.0269	1090	2830	

(1) A suggested safety factor (ASD) is 2.5; the recommended ϕ factor (LRFD) is 0.60.

(2) A recommended safety factor (ASD) is 2.75; the recommended ϕ factor (LRFD) is 0.55. The table is based on typical form deck material (F_y = 80 ksi); a 70 ksi electrode strength was used. Washers are 16 gage.

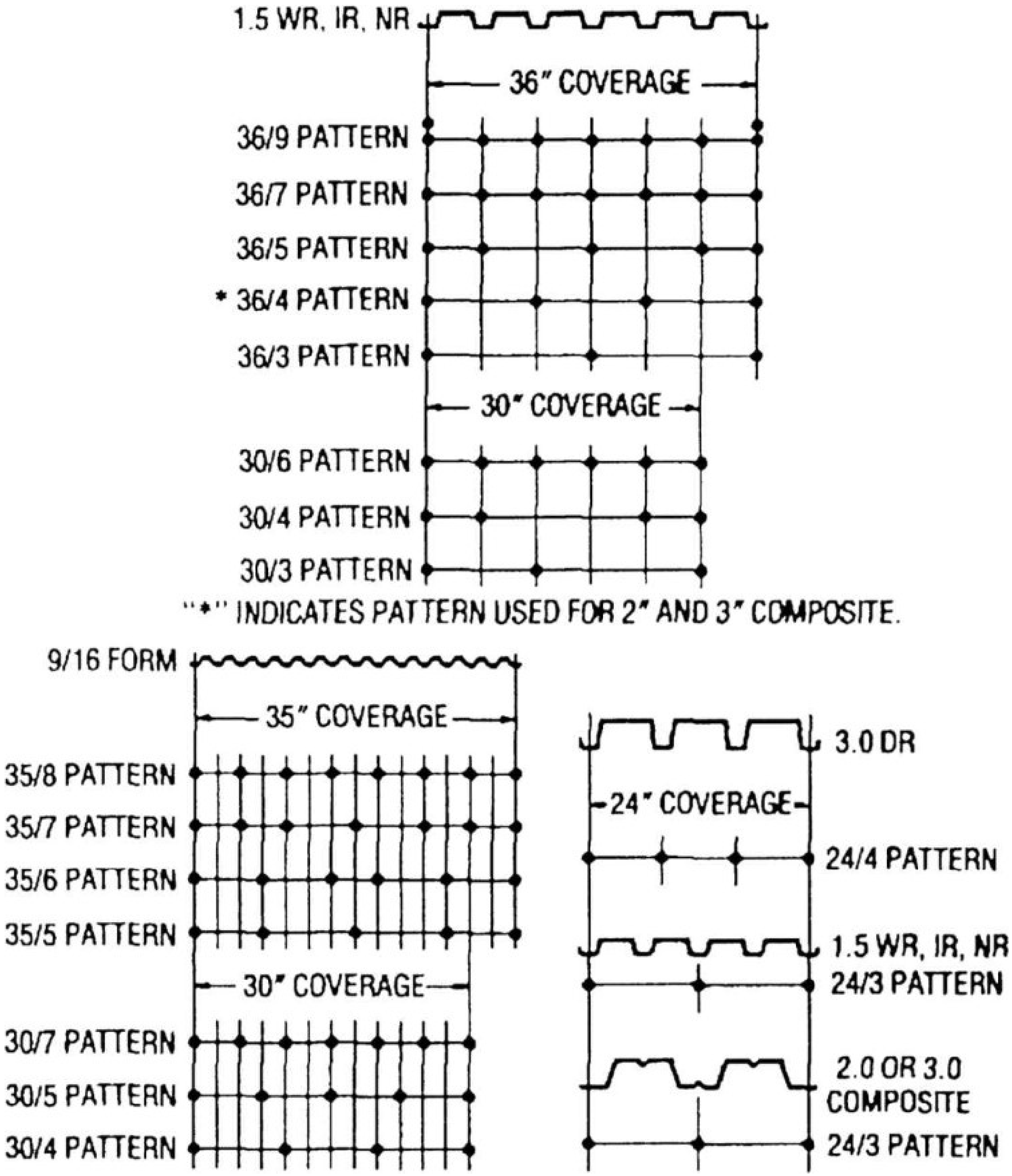


FIGURE 9.2 Frame connection layouts. (*Courtesy of the Steel Deck Institute*.)

Self-drilling screws are frequently used as deck-to-frame attachments. These are installed with an electric screw gun that has a clutch and a depth-limiting nose piece to prevent over-torque. Screws are #12s or #14s (¼ in) with the drill point selected to drill through the total metal thickness of deck and beam (or joist) flange. Uplift (pullover and pullout) values are shown in Table 9.4. The lesser value of P_{nov} and P_{not} is used in design. Screws are preferred when fastening to light gage framing that is thinner than 10 gage. Self-drilling screws are available for the special application of steel deck to wood framing. Consult the screw manufacturer for proper selection and specification of fasteners. Corrosion of screws in timber due to moisture and salt preservatives must be considered.

TABLE 9.4 Uplift Values for Screwed Deck

SCREW DATA			
Screw Size	d dia.	d_w nom. head dia.	Avg. tested tensile strength, kips
10	0.190	0.415 or 0.400	2.56
12	0.210	0.430 or 0.400	3.62
¼	0.250	0.480 or 0.520	4.81

Pull Out Values, kips

$P_{not} = 0.85t_2dF_{u2}$; Steel thickness = t_2, F_{u2} = tensile strength of support

Screw	Gage									
	1/4"	3/16"	10	1/8"	12	14	16	18	20	22
#10	2.34	1.76	0.98	1.17	0.76	0.54	0.43	0.34	0.26	0.21
#12	2.66	2.00	1.11	1.33	0.86	0.62	0.49	0.39	0.30	0.24
1/4"	3.08	2.31	1.29	1.54	1.00	0.71	0.57	0.45	0.34	0.28

The table pullout strengths kips, are based on F_u = 45 for 10, 12 thru 22 gage, and 58 ksi for 1/4", 3/16", and 1/8". The thread must penetrate the full thickness.

Pull Over Values, kips

$P_{nov} = 1.5t_1d_wF_{u1}$; $d_w < 0.50"$; Sheet thickness = t_1, F_{u1} = tensile strength of sheet

d_w	Gage						
	16	18	20	22	24	26	28
0.400	1.61	1.28	0.97	0.80	0.89	0.67	0.55
0.415	1.68	1.33	1.00	0.83	0.92	0.69	0.58
0.430	1.74	1.38	1.04	0.86	0.96	0.72	0.60
0.480	1.94	1.54	1.16	0.96	1.07	0.80	0.67
0.500	2.02	1.60	1.21	1.00	1.11	0.83	0.69

The table pull over strengths kips, are based on F_u = 45 ksi for 16 thru 22 gage, and 62 ksi for 24 thru 28 gage.
The safety factor, Ω, for pull over (ASD) is 3. The ϕ factor (LRFD) is 0.5.

Deck can be screwed to structural steel, SJI joists or light gage steel framing. The lowest steel strength was used to produce the tabulated values. For joists and structural steel, a tensile strength (F_u) of 58 ksi was used which is the lowest value for A36 steel. For gage supports, F_u =45 ksi was used which is the lowest provided in ASTM A653 for Structural Steel grade 33. Deck material furnished in gages 24, 26 28 are usually grade 80 steel which uses a tensile strength (F_u) of 62 ksi as limited by the AISI specifications. Either pull out of the screw or pullover of the deck will normally control. The values are based on the equations provided by the AISI Specifications. These specifications do not require prying at side-laps or ends. A safety factor of 3 is applied for ASD & a ϕ factor of 0.5 is applied for LRFD. If it is known that the tensile strength of the support steel or the sheet steel is greater than those used for the tables, the tabulated screw ultimate strengths may be increased by a straight line ratio.

Other excellent deck-fastening methods have been developed which compete with traditional welds and screws. These fastening methods use powder or air pressure to drive pins through the deck into structural steel. These fasteners are proprietary and not generically covered in the AISI specification. Strength values and diaphragm tables are published by the manufacturers of these products and they also provide technical assistance for designers. The Steel Deck Institute advises that, “No substitution of fastener type or pattern should be made without the approval of the designer.” Fastener manufacturers can provide the data needed to make substitutions using their products.

Shear studs can be welded through the deck into the steel framing with an automatic stud “gun.” The primary function of the studs is to make the beam act compositely with concrete but they also act to fasten the deck to the frame. Studs also increase the composite deck slab capacity. Shear studs can be welded through two well-mated thicknesses of steel such as cellular deck. But, for deck heavier than 16 gage, consultation with the stud manufacturer is advised. Welding time and settings are dependent on: deck coating, steel thickness, and ambient conditions. Although research is ongoing, paint is not recommended on the beam surface receiving studs. The American Welding Society (AWS D1.1) provides a quality control check for welded studs.

Deck-to-deck connections at side laps are sometimes called “stitch connections.” Screws, welds, and button punches are the usual ways to accomplish the connection. The primary purpose of side-lap attachments is to let adjacent sheets help in sharing vertical and horizontal loads.

Stitch screws are usually of the self-drilling type; #8s through #14 (¼ in) diameter can be used but screws smaller than #10s diameter are not recommended. The installer must be sure that the underlying sheet is drawn tightly against the top sheet and that adequate edge dimension is maintained—the normal rules are 1.5 times the screw shank diameter measured to the center of the screw and as required to develop shear when edge is measured parallel to the line of force. Again, as when screws are used as the frame attachment, special screw-driving guns are used to prevent over-torque.

Manual button punching of side laps requires a special crimping tool. Button punching requires the worker to adjust his or her weight so the top of the deck stays level across the joint. Since the quality of the button punch attachment depends on the strength and care of the tool operator, it is important that a consistent method be developed. Automatic power-driven crimping devices are rarely seen on deck jobs but should not be ruled out as a fastening method. Some manufacturers do provide proprietary crimping tools and can provide test based diaphragm load tables using these connections.

Good metal-to-metal contact is necessary for sidelap welds. Burn holes are the rule rather than the exception and an inspector should not be surprised to see them in the deck. The weld develops its strength by holding around the perimeter. On occasion, side-lap welds will be specified for deck that has the button punchable side-lap arrangement (see Fig. 9.3 for comments on this subject; see Fig. 9.4 for welding these deck units to the frame). Welding side laps is not recommended for a 22-gage deck (0.028 in) or lighter. Weld washers should never be used at side laps between supports. The SDI recommends that side laps be connected at a maximum spacing of 36 in (1 m) for deck spans greater than 5 ft (1.5 m). This minimum spacing could be increased to enhance diaphragm values. Edge fasteners parallel with the deck span and over supports are recommended. Supports that are parallel to the deck span and between support beams are recommended at roof perimeters or shear walls. This allows edge fasteners. The edge fastener spacing at these parallel supports should match the deck side-lap fastener spacing.

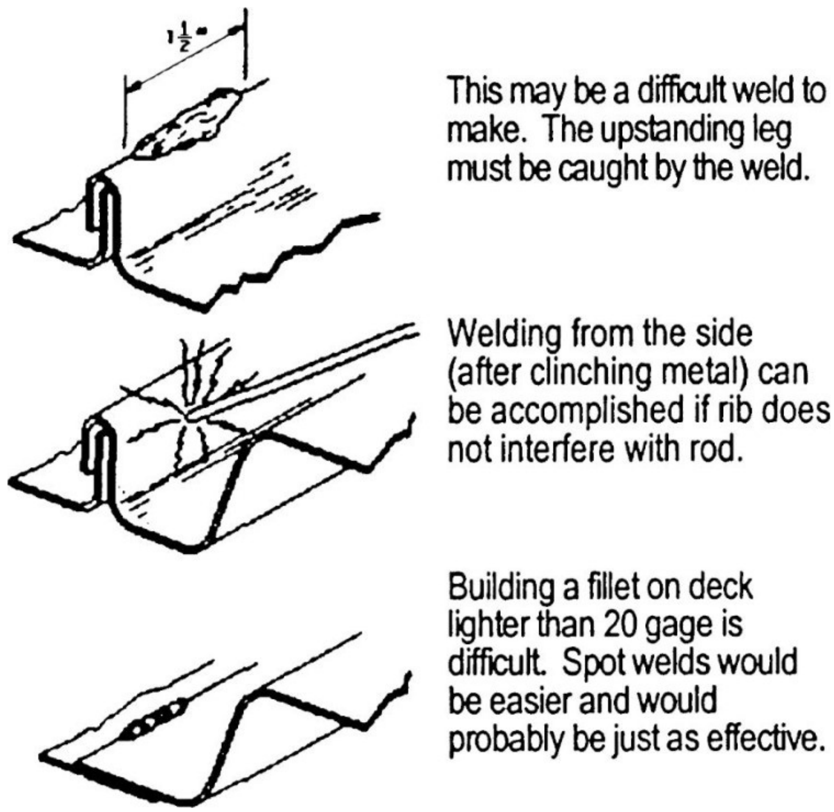


FIGURE 9.3 Sheet-to-sheet welds between supports. (*Courtesy of the Steel Deck Institute.*)

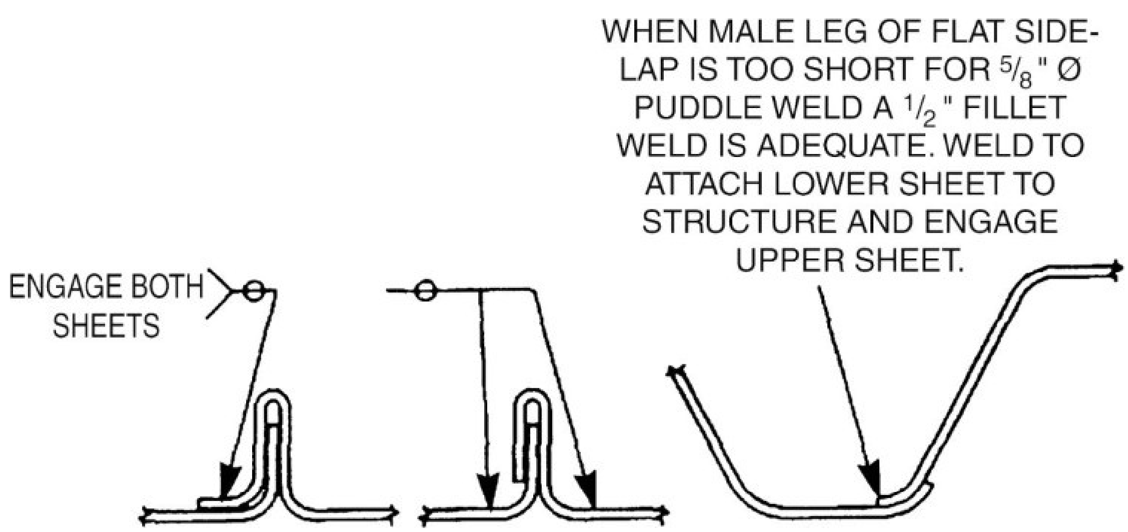


FIGURE 9.4 Side-lap welds at supports. (*Courtesy of the Steel Deck Institute.*)

Accessories attached to the deck are welded, screwed, pop riveted, or (rarely) glued. Usually the choice is left to the erector and many times is simply the result of the tools available at the time. The importance of fastening accessories can be either structural or architectural, and the designer may need to become involved. For instance, the attachment of reinforcement around penetrations, and the fastening of pour stops, may have a great deal to do with the expected performance of the accessory and care must be taken to see that sufficient attachment is done. If the deck is to be exposed to view, then architectural considerations might be of concern and the fasteners may be selected accordingly. Button punched side laps are often specified at exposed interlocking or cellular deck side laps.

Frequently the expression "tack welding" is used to describe attachment of accessories to deck or to structural steel. A *tack weld* is defined by the AWS as "a weld made to hold parts of a weldment in proper alignment until the final welds are made." The term, when applied to accessories, means a weld of unspecified strength or size simply used to hold the accessory securely in its proper position. When floor deck accessories are tack-welded, the concrete is usually the medium that will hold the parts in their final place. The accessories shown in Fig. 9.5 can be tack-welded or screwed as is appropriate. The one exception is the case of pour stops; the SDI calls for 1-in fillet welds at 12-in oc to the structural steel.

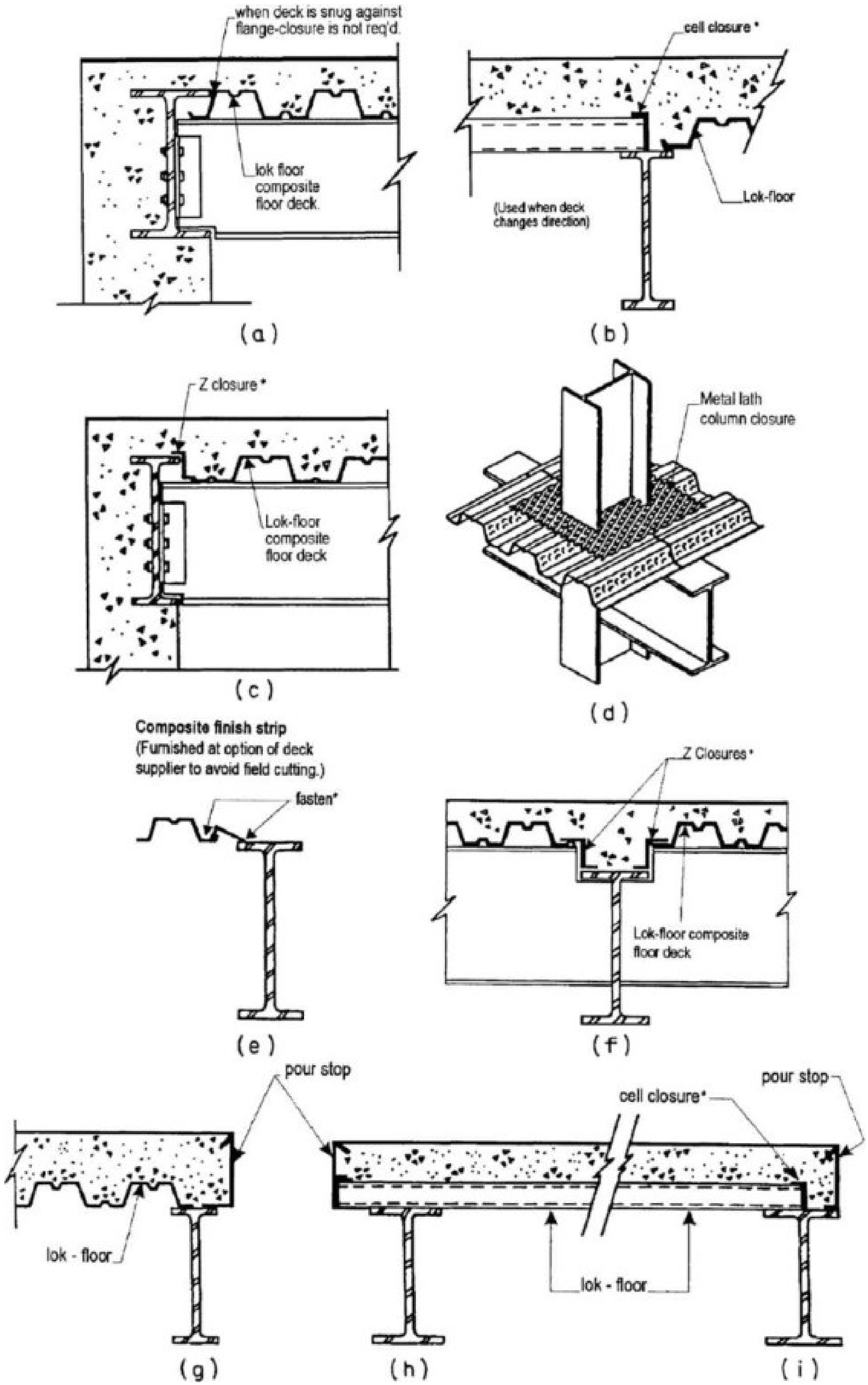


FIGURE 9.5 Fastening floor deck accessories. (*Courtesy of Canam Manufacturers of United Steel Deck Products*.)

*Attach closures to deck or supports using #10 screws (minimum) or 1-in fillet welds at a maximum spacing of 24 in on centers. Welds are commonly used at supports.

Some additional details on steel joist bearing and connections are shown in Figs. 9.6 and 9.7.

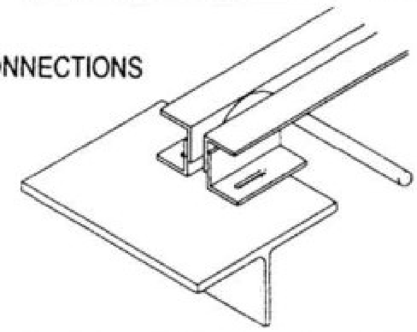
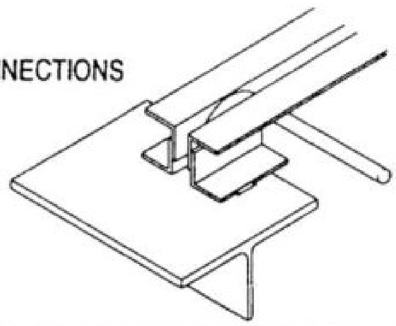
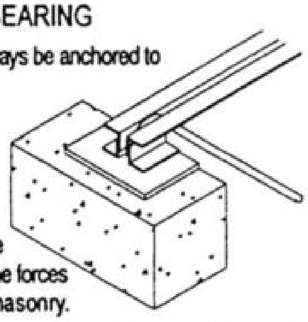
TYPE	K SERIES	LH/DLH SERIES
BOLTED CONNECTIONS 	Slotted holes in bearing plates are furnished whenever bolted connections are required. Bolts ($\frac{1}{2}$-in diameter) are not furnished by the joist manufacturer. Minimum bearing on structural steel supports is $2\frac{1}{2}$-in.	Slotted holes in bearing plates are furnished whenever bolted connections are required. Bolts ($\frac{3}{4}$-in diameter) are not furnished by the joist manufacturer. Minimum Bearing on structural steel supports is 4-in.
WELDED CONNECTIONS 	Ends of K Series joists are normally anchored by two 1/8-in fillet welds 1-in long. Minimum bearing on structural steel supports is $2\frac{1}{2}$-in.	Ends of LH/DLH Series joists are normally anchored by two 1/4-in fillet welds 2-in long. Minimum bearing on structurtal steel supports is 4-in.
TYPICAL MASONRY BEARING The setting plates should always be anchored to the masonry wall. The setting plate must be located not more than ½" from the face of the wall. The design professional must design the bearing plate and must take into account the forces acting on the concrete or masonry. 	Minimum bearing is 4-in.	Minimum bearing is 6-in.

FIGURE 9.6 Joist bearing details.

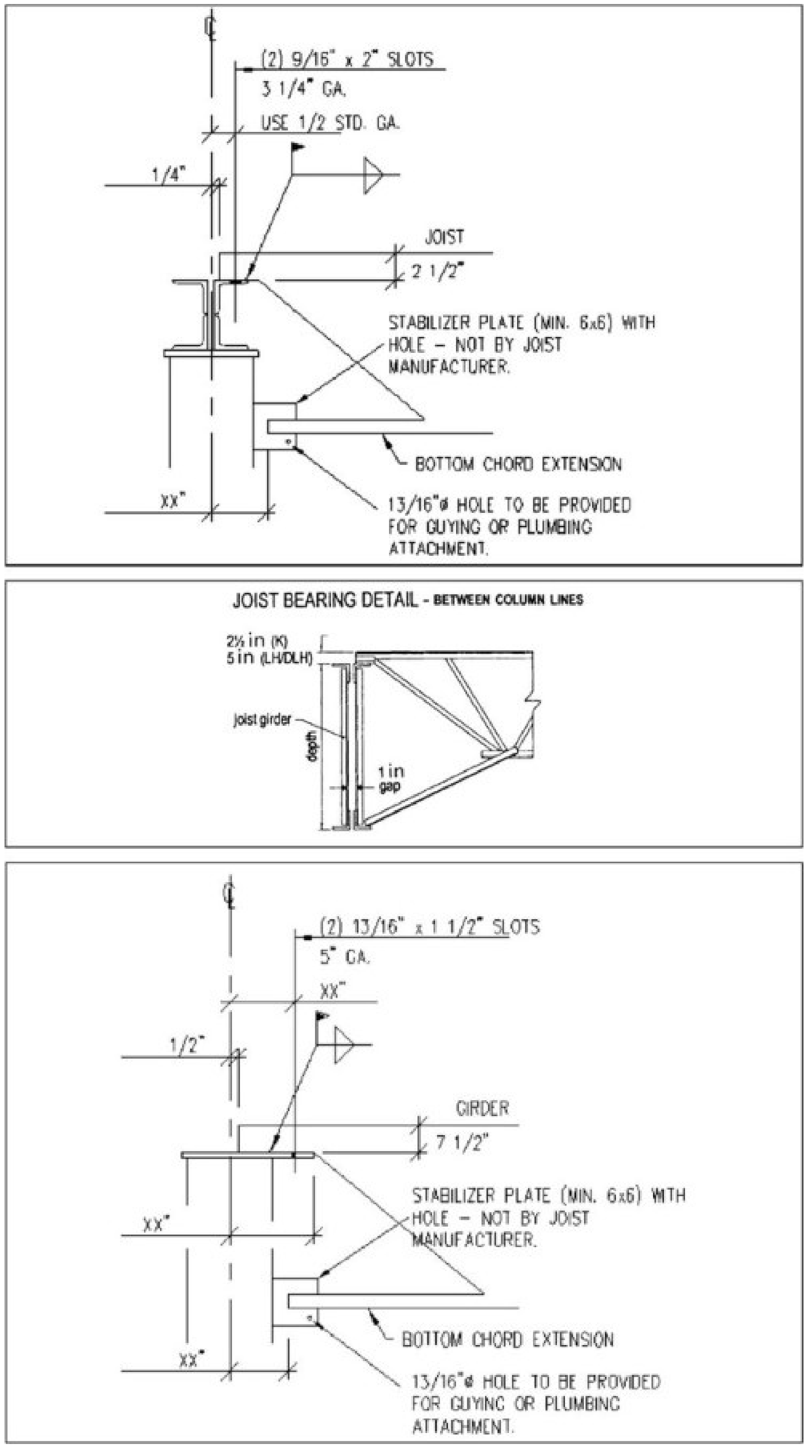
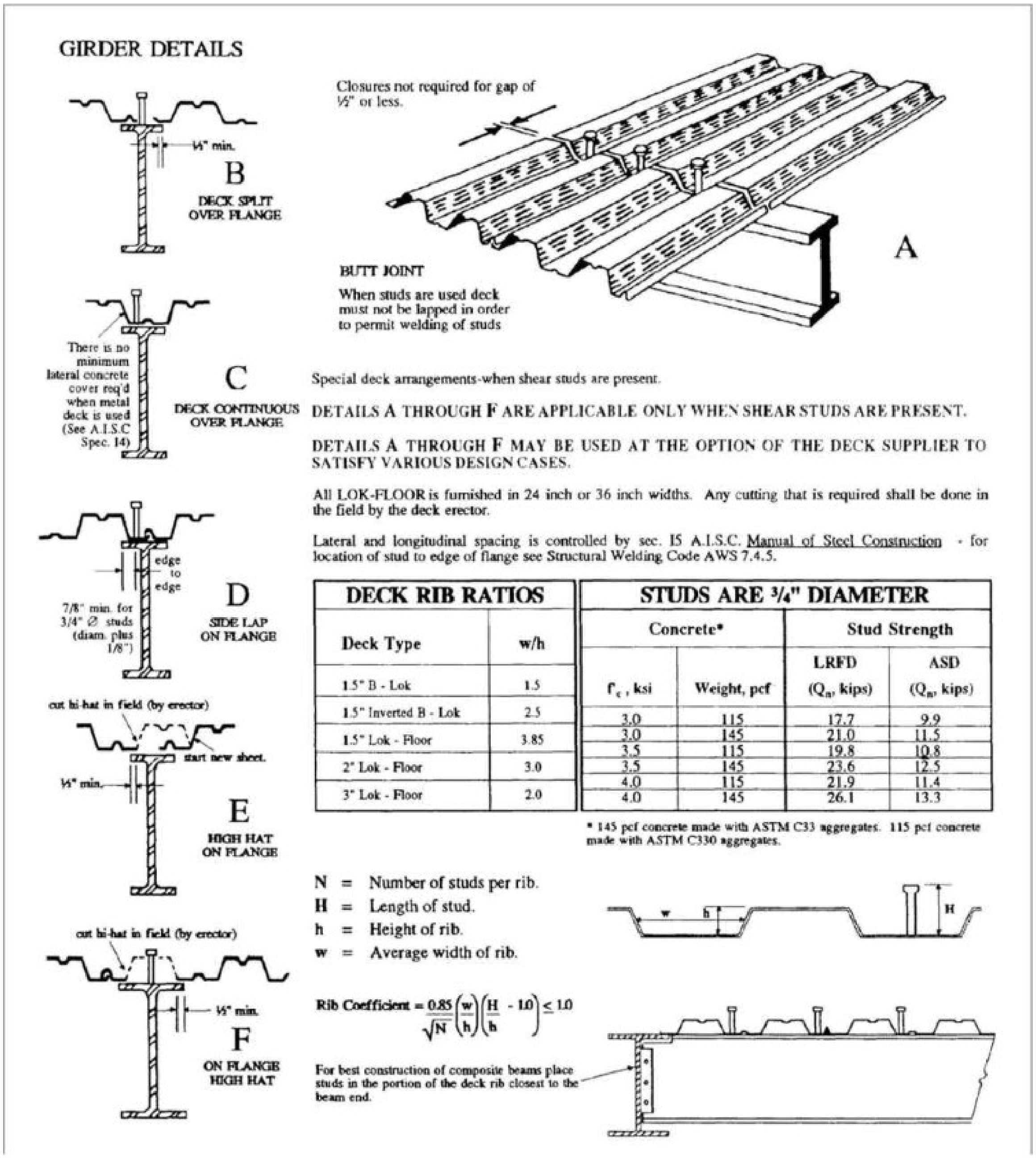


FIGURE 9.7 Joist bearing on joist girders.

Composite beam details showing metal deck connected to steel beams are shown in Fig. 9.8. Table 9.5 presents the ¾-in diameter shear stud values in deck. Additional details of commonly used metal deck constructions are shown in Figs. 9.9 through 9.12. Industry generic details are available in the SDI publication, “Standard Practice Details.”



DECK RIB RATIOS	
Deck Type	**w/h**
1.5" B - Lok	1.5
1.5" Inverted B - Lok	2.5
1.5" Lok - Floor	3.85
2" Lok - Floor	3.0
3" Lok - Floor	2.0

STUDS ARE ¾" DIAMETER			
Concrete*		**Stud Strength**	
		LRFD	**ASD**
f'_c, ksi	**Weight, pcf**	**(Q_n, kips)**	**(Q_n, kips)**
3.0	115	17.7	9.9
3.0	145	21.0	11.5
3.5	115	19.8	10.8
3.5	145	23.6	12.5
4.0	115	21.9	11.4
4.0	145	26.1	13.3

FIGURE 9.8 Composite beam details.

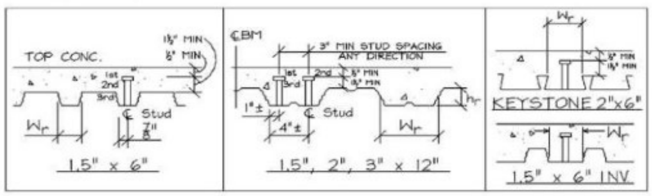
TABLE 9.5 Shear Stud Strength

Special Note: Section I3.2c of the *2005 ANSI/AISC Specification for Structural Steel Buildings* in the *AISC ASD/LRFD Steel Construction Manual, 13th Edition* specifies new strength criteria for studs in composite beams with formed deck. This table supersedes the values in the SDI Standard Practice Details, May 2001 and the revisions of April 2003.

Q_n in Steel Deck—LRFD

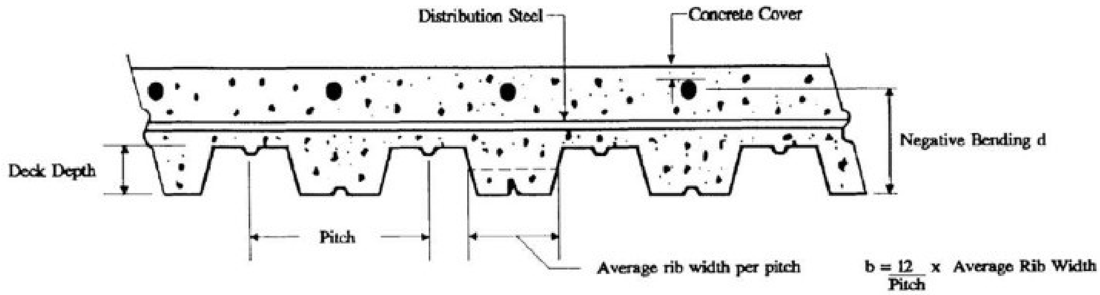
¾ in ϕ Shear Stud's Average Nominal Shear Strength/Stud in a Deck Corrugation, kips

Profile	w_r in	Concrete density, pcf	Studs per corr.	Perpendicular to beam			Parallel to beam		
				f'c Concrete compressive strength, ksi					
				3.0	3.5	4.0	3.0	3.5	4.0
Solid concrete	NA	145	NA	21.0	23.6	26.1	21.0	23.6	26.1
		115		17.7	19.8	21.9	17.7	19.8	21.9
1.5 × 6 deck	2.125D	145	1	17.2			18.3		
			2	14.6					
			3	12.1					
		115	1	17.2			17.7	18.3	18.3
			2	14.6					
			3	12.1					
1.5, 2, 3 × 12	6	145	1	17.2			21.0	21.5	21.5
			2	16.5					
			3	13.1					
Keystone	4.6	115	1	17.2			17.7	19.8	21.5
			2	16.5					
1.5 × 6 inverted	3.75		3	13.1					



1. Preferred (strong) stud location is closest to beam ends in each corrugation with studs.
2. Table assumes that the first stud will be located in the weak location, the second in the strong location, and the third in the weak location. All studs through 1.5 × 6-in deck are in the weak location.
3. w_r = average width of rib = 2 in. When deck is parallel to beam, w_r = 5 in for two studs across the corrugation. For multiple studs in Keystone Deck or 1.5 × 6-in inverted, split the deck when parallel to beam and see values when perpendicular to beam.
4. h_r = height of rib = 3 in.
5. Density = 145 PCF conforms to ASTM C33; 115 PCF conforms to ASTM C330.
6. Studs conform to ANSI/AWS D1.1 with F_u = 65 ksi.
7. When deck is parallel to the beam: (a) The minimum center-to-center spacing of studs installed along the beam is 4½ in. (b) When w_r is wide enough, we suggest that studs be staggered either side of the corrugation. (c) Deck may be split over beams. (d) When studs are side-by-side, the minimum transverse spacing is 3 in.
8. The maximum center-to-center spacing of studs shall neither exceed 8 times total slab thickness nor 36 in.
9. Studs of lesser diameter are allowed and shear values and minimum spacing are reduced.

**Courtesy of the Steel Deck Institute.*



DECK SECTION	PITCH	AVERAGE RIB WIDTH	b WIDTH for NEGATIVE BENDING
B-LOK	6"	2.25"	4.5"
INV B-LOK	6"	3.75"	7.5"
LOK FLOOR	12"	6"	6"

Use Standard concrete design procedures as per ACI.

1.) Allowable bending stress of 20 ksi with loading of concrete + deck + 20 psf or concrete + deck + 150 lb. concentrated load, whichever is worse.

2.) Allowable deflection of free edge (based on fixed end cantilever) of 1/120 of cantilever span under loading of concrete + deck.

3.) Bearing width of 3½" assumed for web crippling check - concrete + deck + 20 psf over cantilever and adjacent span: if width is less than 3½" check with Summit, New Jersey office.

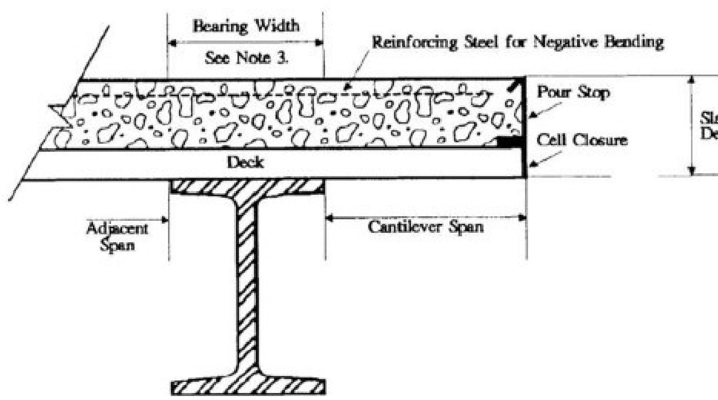


FIGURE 9.9 Negative bending information. LRFD methods are allowed and a deflection limit of 1/90 has been adopted by industry.

FLOOR DECK CANTILEVERS

NORMAL WEIGHT CONCRETE (150 PCF)

	UNITED STEEL DECK, INC. DECK PROFILE															
	B-LOK				1.5 LOK-FLOOR				2.0 LOK-FLOOR				3.0 LOK-FLOOR			
Slab Depth	22	20	18	16	22	20	18	16	22	20	18	16	22	20	18	16
4.00"	1'11"	2'3"	2'10"	3'4"	1'11"	2'4"	3'0"	3'6"								
4.50"	1'10"	2'2"	2'9'	3'3"	1'10"	2'3"	2'10"	3'4"	2'6"	2'11"	3'8"	4'3"				
5.00"	1'10"	2'2"	2'8"	3'2"	1'10"	2'3"	2'9"	3'3"	2'5"	2'10"	3'6"	4'1"	3'8"	4'3"	5'3"	6'0"
5.50"	1'9"	2'1"	2'7"	3'0"	1'9"	2'2"	2'9"	3'2"	2'4"	2'9"	3'5"	4'0"	3'7"	4'1"	5'0"	5'9"
6.00"	1'9"	2'0"	2'6"	2'11"	1'9"	2'1"	2'8"	3'1"	2'3"	2'8"	3'4"	3'10"	3'5"	3'11"	4'10"	5'7"
6.50"	1'8'	2'0"	2'6"	2'11"	1'9"	2'1"	2'7"	3'0"	2'3"	2'8"	3'3"	3'9"	3'4"	3'10"	4'8"	5'5"
7.00"	1'8"	1'11"	2'5"	2'10"	1'8"	2'0"	2'6"	2'11"	2'2"	2'7"	3'2"	3'8"	3'3"	3'9"	4'6"	5'3"
7.50"	1'8"	1'11"	2'4"	2'9"	1'8"	2'0"	2'6"	2'10"	2'2"	2'6"	3'1"	3'7"	3'2"	3'8"	4'5"	5'1"
8.00"	1'7"	1'11"	2'4"	2'8"	1'7"	1'11"	2'5"	2'10"	2'1"	2'5"	3'0"	3'6"	3'1"	3'6"	4'3"	4'11"

FIGURE 9.10 Floor deck cantilevers. These values are dependent on the back span. Do not walk on the deck until properly fastened to supports. Side laps are to be fastened at 12 in oc at the cantilever.

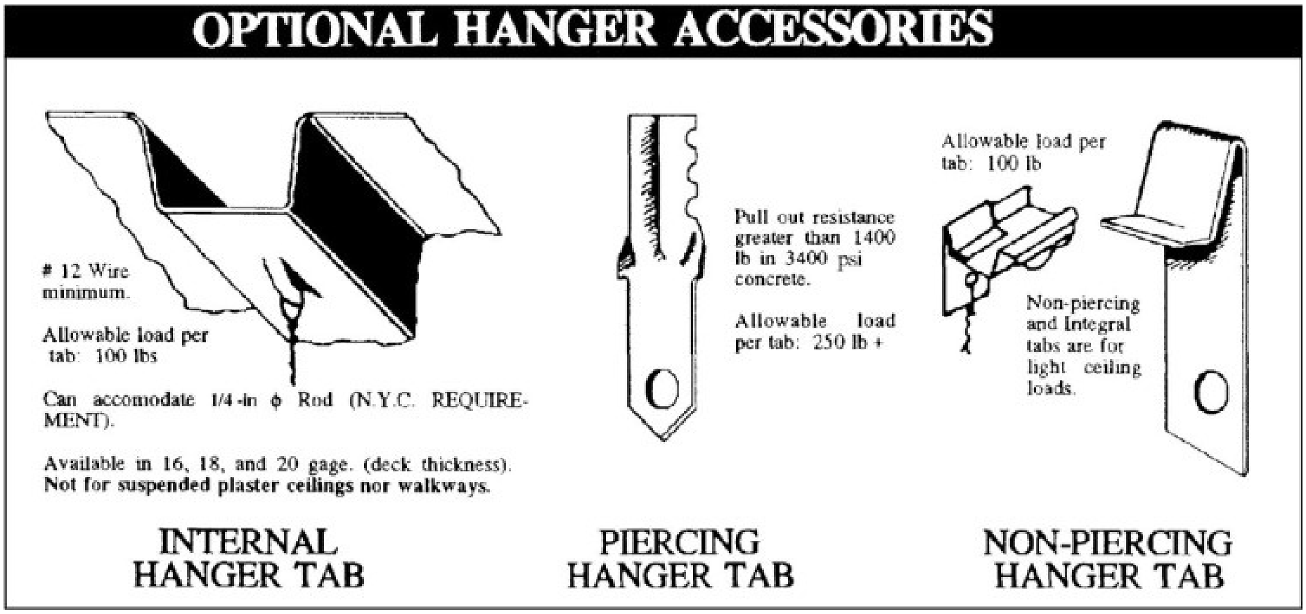


FIGURE 9.11 Optional hanger accessories.

Slab Depth, in	Overhang, in												
	0	1	2	3	4	5	6	7	8	9	10	11	12
4.00	20	20	20	20	18	18	16	14	14	12	12	10	10
4.25	20	20	20	18	18	16	16	14	12	12	12	10	10
4.50	20	20	20	18	18	16	16	14	12	12	12	10	10
4.75	20	20	18	18	16	16	14	14	12	12	10	10	10
5.00	20	20	18	18	16	16	14	14	12	12	10	10	
5.25	20	20	18	16	16	14	14	12	12	12	10	10	
5.50	20	18	18	16	16	14	14	12	12	12	10	10	
5.75	20	18	18	16	14	14	12	12	12	12	10	10	
6.00	20	18	16	16	14	14	12	12	12	12	10	10	
6.25	18	18	16	16	14	14	12	12	12	10	10	10	
6.50	18	16	16	14	14	12	12	12	12	10	10		
6.75	18	16	16	14	14	12	12	12	10	10	10		
7.00	18	16	14	14	12	12	12	12	10	10	10		
7.25	16	16	14	14	12	12	12	10	10	10	10		
7.50	16	14	14	12	12	12	12	10	10	10			
7.75	16	14	14	12	12	12	12	10	10	10			
8.00	16	14	12	12	12	12	10	10	10				
8.25	14	14	12	12	12	12	10	10	10				
8.50	14	14	12	12	12	10	10	10					
8.75	14	12	12	12	12	10	10	10					
9.00	14	12	12	12	10	10	10	10					
9.25	14	12	12	12	10	10	10						
9.50	12	12	12	10	10	10	10						
9.75	12	12	12	10	10	10							
10.00	12	12	10	10	10	10							
10.25	12	12	10	10	10								
10.50	12	12	10	10	10								
10.75	12	10	10	10									
11.00	12	10	10	10									
11.25	12	10	10										
11.50	12	10	10										
11.75	10	10	10										
12.00	10	10											

Type	Thickness Inches
20	0.0358
18	0.0474
16	0.0598
14	0.0747
12	0.1046
11	0.1196
10	0.1345

This Selection Chart is based on following criteria:

1. Normal weight concrete (150 pcf).
2. Horizontal and vertical deflection is limited to ¼ -in maximum for concrete dead load.
3. Design stress is limited to 20 ksi for concrete dead load temporarily increased by one-third for the construction live load of 20 pcf.
4. Pour Stop Selection Chart does not consider the effect of the performance, deflection, or rotation of the pour stop support which may include both the supporting composite deck and/or the frame.
5. Vertical leg return lip is recommended.
6. This selection is not meant to replace the judgement of experienced Structural Engineers and shall be considered as a reference only.

Use return lip for composite construction.

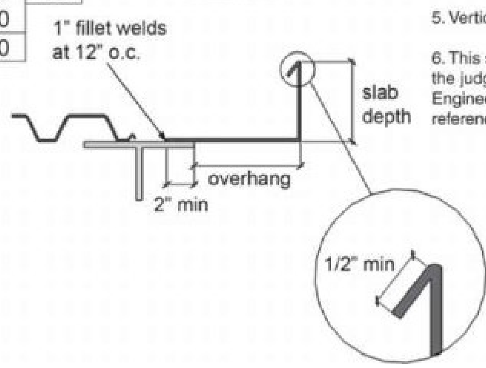


FIGURE 9.12 Pour stop selection chart. Industry recommends a minimum weld of 1 in at 12 in oc. This detail is slightly more restrictive.

Safety provisions mandated by OSHA in the latest edition of *29 CFR Part 1926 Subpart R* generally require that openings be "decked over" until the trade requiring the opening is ready to fill the opening. Unless directed otherwise by a site-specific erection plan, details should be consistent with this provision.

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CHAPTER 10
CONNECTIONS TO COMPOSITE MEMBERS

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(*Courtesy of The Steel Institute of New York.*)

10.1 INTRODUCTION

The combined use of steel and concrete to form composite structures has been used widely. The introduction of new composite building systems has allowed the design and construction of more efficient mid- and high-rise composite buildings. In most composite building systems, the main problem facing designers has been the selection of an appropriate and economical connection.

This chapter provides suggestions for connection details for three types of composite structure elements: (1) connection details for connecting coupling steel beams to reinforced concrete shear walls (Sec. 10.3), (2) joint details for connecting steel beams to reinforced concrete columns (Sec. 10.4), and (3) connection details for attaching steel beams to rectangular and circular concrete-filled steel-tube columns (Sec. 10.5).

10.2 GENERAL DESIGN CONSIDERATIONS

The design of connections, in general, requires consideration of stiffness, strength, stability, serviceability, and cyclic behavior. Following is a brief discussion of each of these items.

10.2.1 Strength and Stiffness

When connections are subjected to applied moment, they will cause rotation at the member end. For instance, if a beam is attached to a column using top and seat angles, the applied moment to the beam end generated by vertical or lateral loads will cause the beam end to rotate with respect to the column face. The amount of this rotation is dependent on the stiffness of the connected elements. Experimental results indicate that all connections exhibit some level of rotation and, therefore, one could argue that all connections are semirigid. For design purposes, however, design manuals divide connections into three categories: (1) connections that exhibit relatively large end rotations (simple connections), (2) connections that result in very small rotation (rigid connections), and (3) those which exhibit end rotations between simple and rigid connections, referred to as semirigid connections.

To date, the majority of efforts in the development of connection details for composite members has been focused on rigid-type connections.

Design of connections for strength requires knowledge about the capacity of each connection element at the ultimate strength limit state. To ensure satisfactory performance of connections at ultimate strength limit strength, failure of connection elements must be prevented or controlled. The objective in design is to prevent damage to the connection at its ultimate strength limit state and shift the failure locations to other parts of the structure. Connections could finally fail if the applied load level exceeds a certain limit. As a result, it is desirable to proportion the connection so that it will fail in a “controlled” and “desirable” manner. For instance, design of connection elements could be “controlled” through proportioning such that at the strength limit state the connection elements fail by yielding and not weld fracture. Yielding and, finally, fracture of steel elements of connections are usually “desirable” modes of failure in comparison to weld fracture, which could take place without warning.

For most composite connections, another major consideration is the ability to inspect the connection after a major event. For instance, after an earthquake, one should be able to inspect the connection and make judgments as to the safety of the connection. Unfortunately, most elements of composite connections are not easily accessible and their full inspection is not feasible. Therefore, the designer needs to proportion the connection elements such that the failure of “hidden” elements is prevented.

10.2.2 Stability

Connection elements could fail as a result of buckling (elastic or inelastic) of connection elements. This mode of failure, however, is not usually a major concern in connection design.

10.2.3 Serviceability

Connections, as with any other member of the structure, should perform satisfactorily at different limit states. At service load levels, performance of connections should not adversely affect the behavior of the structure. For instance, at service load levels connections could be subjected to a large number of load cycles. These loads could be generated by wind loads or machinery in the case of industrial buildings. Although these loads could be substantially lower than the ultimate load-carrying capacity of each connection element, the connection could develop fatigue cracking, which could result in failure.

Large flexibility at the connection level could result in large interstory drift and member deflections. Therefore, the selection of the connection types at various floor levels could be dictated by the service limit state.

10.2.4 Cyclic Behavior

Connections could fail under a large (high-cycle fatigue) or small (low-cycle fatigue) number of cyclic loadings. In the case of high-cycle fatigue, the magnitude of the applied stress is relatively low. Cracking in bridge elements is caused by high-cycle fatigue. On the other hand, the level of applied stress in the case of low-cycle fatigue is relatively high and could approach the yield strength of the connected elements. During major earthquakes, connections in buildings could experience a few cycles of loading with relatively high stress levels at each cycle. Failure of connections by low-cycle fatigue is confined to earthquake-type loading. The amount of available information on low-cycle fatigue characteristics of connections is limited. This is especially true for composite connections. Principles of fracture mechanics and fatigue could be used to establish life of connections under variable cyclic loading. Two approaches could be undertaken. Full-scale testing of connections under constant and variable amplitude loading provides the most reliable information. In the absence of such information, designers could identify the high stress points within the connection and possible load histories that that particular point within the connection could experience during an earthquake. Information on cyclic behavior of different materials, obtained from simple tension-type specimens, is available. Knowledge of the cyclic load history for the portion of the connection with the highest stress and available damage models for particular materials could then be used to estimate the life of connections under cyclic loading.

However, it should be noted that predicting the life of connections under cyclic loading is a very complex process and its accuracy, in many cases, depends on the experience and judgment of the designer. One of the major questions is estimating the load history that the connection could experience during an earthquake. In addition, it is necessary to conduct nonlinear dynamic time-history analyses, incorporating connection behavior (through inclusion of moment-rotation characteristics of the connection). Fortunately, in general, connections in major earthquake events are subjected to a very few cycles of loading with high stress levels. In general, bolted connections demonstrate better cyclic behavior than welded connections. Behavior of welded connections depends, to a large extent, on quality control and workmanship.

10.3 BEAM-TO-WALL CONNECTIONS

10.3.1 Introductory Remarks

Structural walls/cores are commonly used for lateral load strength and stiffness. For low- to moderate-rise buildings, up to 25 to 30 stories, the walls/cores can be used to provide a majority of the lateral force resistance. For taller buildings, the use of dual systems is more common, where the perimeter frames are engaged with the walls/cores. Outrigger beams are framed between the core walls and columns (which may be all steel or composite) in the perimeter frame. Core walls can effectively be formed by coupling individual wall piers, which may be slip-formed to accelerate construction, with the use of reinforced concrete or steel/steel-concrete composite coupling beams. The floor plan of a representative hybrid building is shown in Fig. 10.1. The walls may be reinforced conventionally, that is, consisting of longitudinal and transverse reinforcement, or may include embedded structural steel members in addition to conventional reinforcing bars. The successful performance of such hybrid structural systems depends on the adequacy of the primary individual components which are the walls/cores, steel frames, and frame-core connections. The focus of this section is on the connections between outrigger beams and walls and the connections between steel/steel-concrete composite coupling beams and walls. Issues related to design of steel/steel-concrete composite coupling beams and connections between floor diaphragms and walls are also discussed.

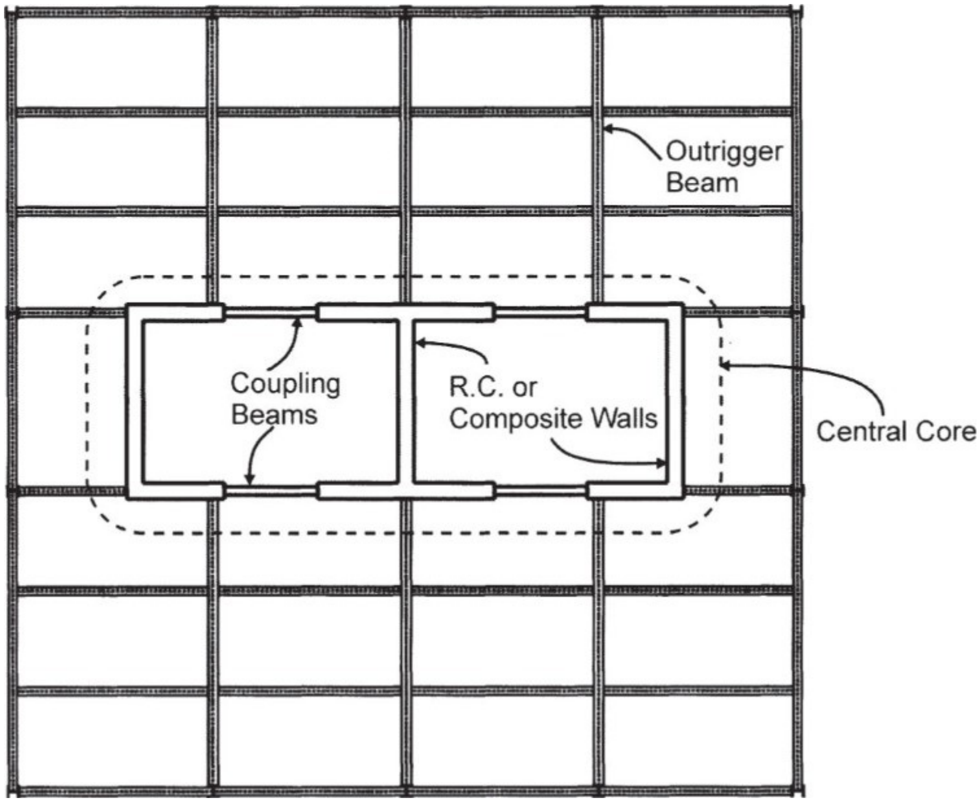


FIGURE 10.1 Structural components of core wall frame systems.

10.3.2 Qualitative Discussion About Outrigger Beam-Wall Connection and Coupling Beam-Wall Connection

Connections between walls and steel/composite coupling beams or outriggers depend on whether the wall boundary element is reinforced conventionally or contains embedded structural steel columns, the level of forces to be developed, and whether the walls are slip-formed or cast conventionally. A summary of possible connections is provided in the following.

10.3.2.1 Coupling Beam-Wall Connection. Well-proportioned coupling beams above the second floor are expected to dissipate a majority of the input energy during severe earthquakes. Coupling beams will, therefore, undergo large inelastic end rotations and reversals, and adequate connection between coupling beams and wall piers becomes a critical component of the overall system behavior. The connection varies depending on whether reinforced concrete or steel/steel-concrete composite coupling beams are used. A comprehensive discussion for reinforced concrete coupling beams and their connections to walls is provided elsewhere (for example, Barney et al., 1978; Paulay, 1980, 1986; Aktan and Bertero, 1981; Paulay and Binney, 1975; Paulay and Santhakumar, 1976).

10.3.2.2 Steel/Steel-Concrete Coupling Beams. Structural steel coupling beams provide a viable alternative, particularly in cases where height restrictions do not permit the use of deep reinforced concrete beams, or where the required capacities and stiffness cannot be developed economically by a concrete beam. The member may be encased with a varying level of longitudinal and transverse reinforcement.

If the wall boundary elements include embedded structural steel columns, the wall-coupling beam connection is essentially identical to steel beams and columns but with some modifications. For steel boundary columns located farther away than approximately 1.5 to 2 times the beam depth from the edge, the beam forces can be transferred to the core by the bearing mechanism mobilized by the beam flanges, as illustrated in Fig. 10.2. In such cases, the beam-column connection becomes less critical, and the necessary embedment length can be computed based on a number of available methods, as discussed in Sec. 10.3.3.3. If the embedded steel boundary column is located within approximately 1.5 times the beam depth from the wall edge, the forces can be transferred by mobilizing the internal couple involving the column axial load and bearing stresses near the face, as shown in Fig. 10.3. Clearly, the beam-column connection becomes critical in mobilizing this mechanism. The connection between the coupling beam and steel boundary column is expected to be enhanced by the presence of concrete encasement as indicated by a recent study (Leon et al., 1994) which shows improved performance of encased riveted beam column. Due to insufficient data, however, it is recommended to ignore the beneficial effects of the surrounding concrete, and to follow standard design methods for steel beam-column connections. Outrigger beams may also be directly attached to columns which are closer to the core face and protruded beyond the column. This detail is illustrated in Fig. 10.4. Considering the magnitude of typical coupling beam forces, the steel boundary column may deform excessively, particularly if the column is intended to serve as an erection column, leading to splitting of the surrounding concrete and loss of stiffness. Adequate confinement around the column and headed studs improves the behavior by preventing separation between the steel column and surrounding concrete.

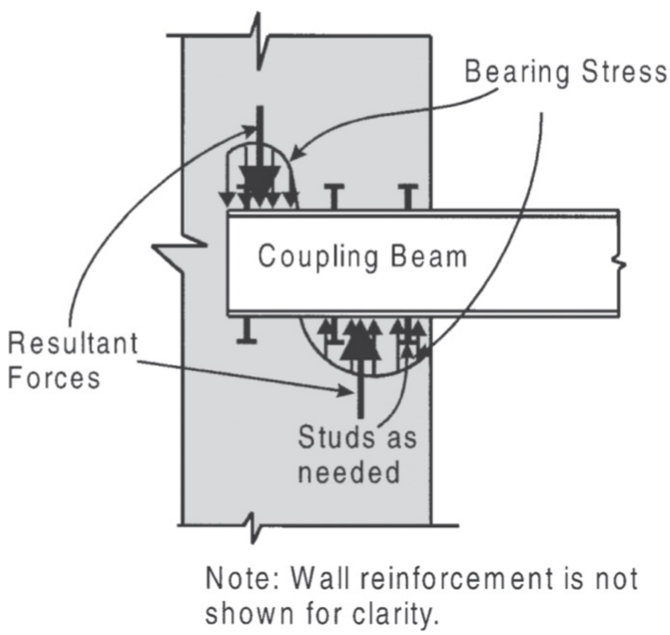


FIGURE 10.2 Transfer of coupling beam forces through bearing.

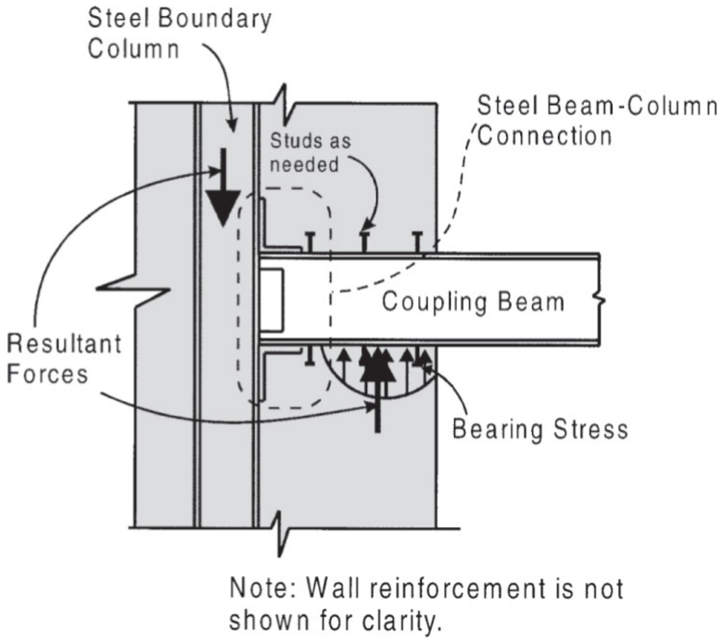


FIGURE 10.3 Transfer of coupling beam forces through bearing and beam-column connections.

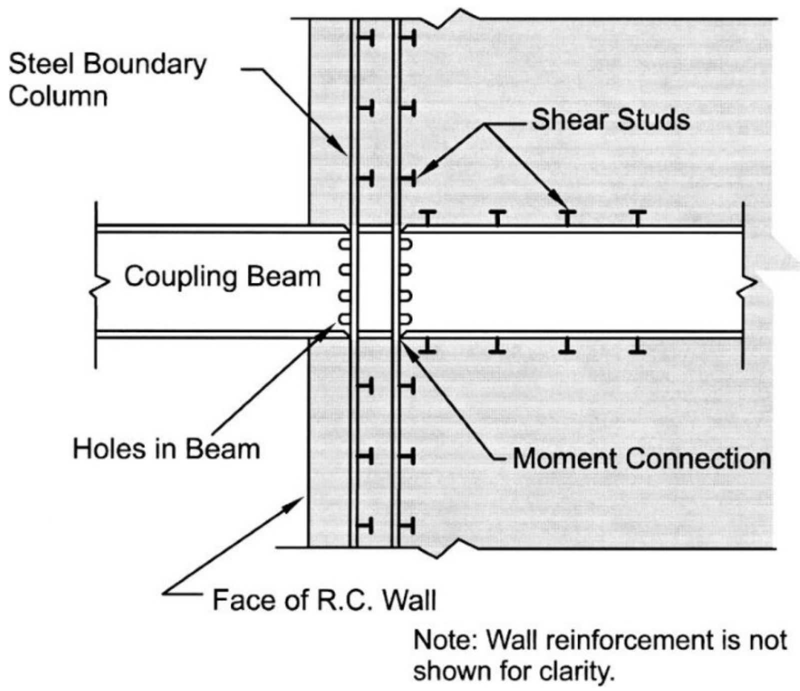


FIGURE 10.4 Transfer of coupling beam forces through a direct beam-column connection.

If the wall boundary element is reinforced with longitudinal and transverse reinforcing bars, a typical connection involves embedding the coupling beam into the wall and interfacing it with the boundary element, as illustrated in Fig. 10.5. The coupling beam has to be embedded adequately inside the wall such that its capacity can be developed. A number of methods may be used to calculate the necessary embedment length (Marcakis and Mitchell, 1980; Mattock and Gaafar, 1982). These methods are variations of Precast Concrete Institute (PCI) guidelines for design of structural steel brackets embedded in precast reinforced concrete columns. Additional details regarding the design methodology are provided in Sec. 10.3.3.3. A second alternative is possible, particularly when core walls are slip-formed. Pockets are left open in the core to later receive coupling beams. After the forms move beyond the pockets at a floor, steel beams are placed inside the pockets and grouted. This detail is illustrated schematically in Fig. 10.6. Calculation of the embedment length is similar to that used for the detail shown in Fig. 10.5.

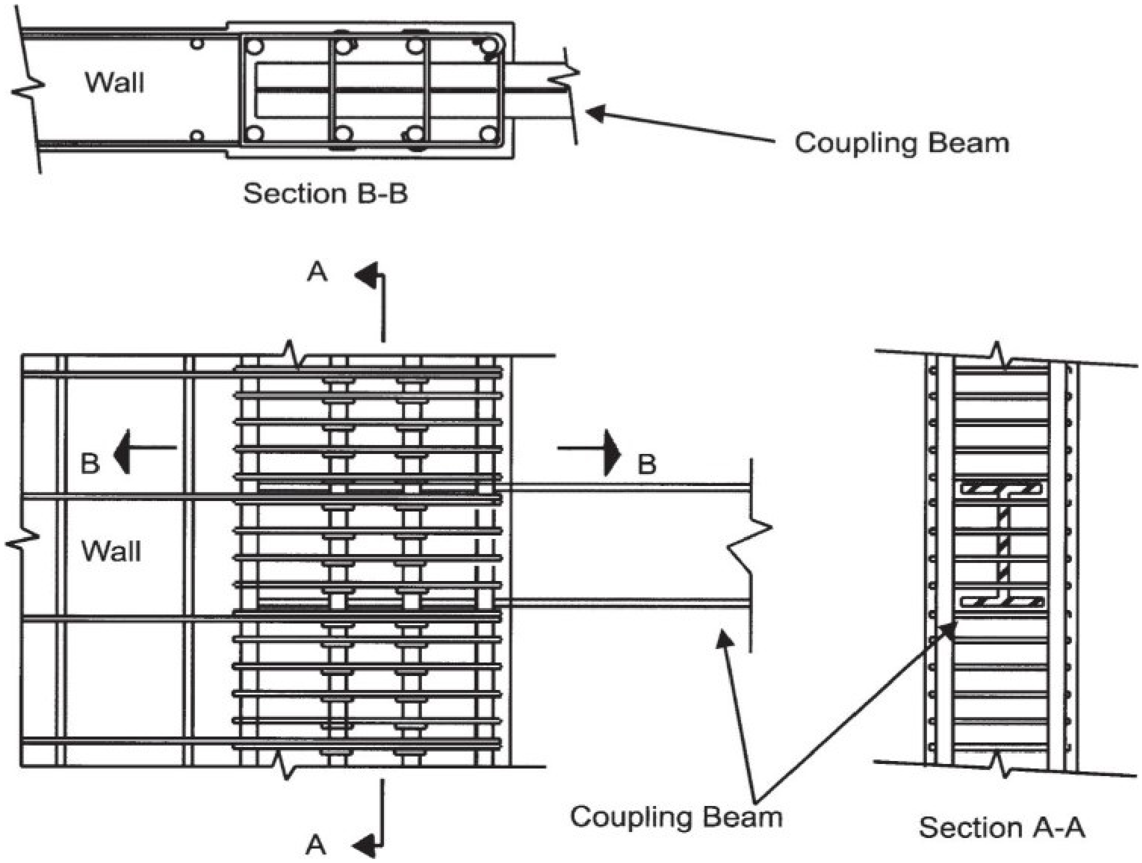


FIGURE 10.5 Coupling beam-wall connection for conventionally reinforced walls.

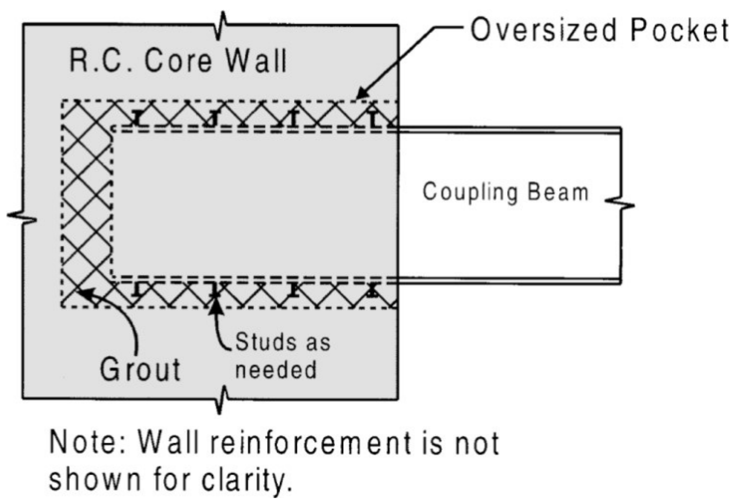


FIGURE 10.6 A possible coupling beam-wall connection for slip-formed walls.

10.3.2.3 Outrigger Beam-Wall Connection. In low-rise buildings, up to 30 stories, the core is the primary lateral load–resisting system, the perimeter frame is designed for gravity loads, and the connection between outrigger beams and cores is generally a *shear connection.* A typical shear connection is shown in Fig. 10.7. Here, a steel plate with shear studs is embedded in the wall/core during casting, which may involve slip-forming. After casting beyond the plate, the web of the steel beam is welded to the stem of a steel plate (*shear tab*) which is already welded to the plate. Variations of this detail are common.

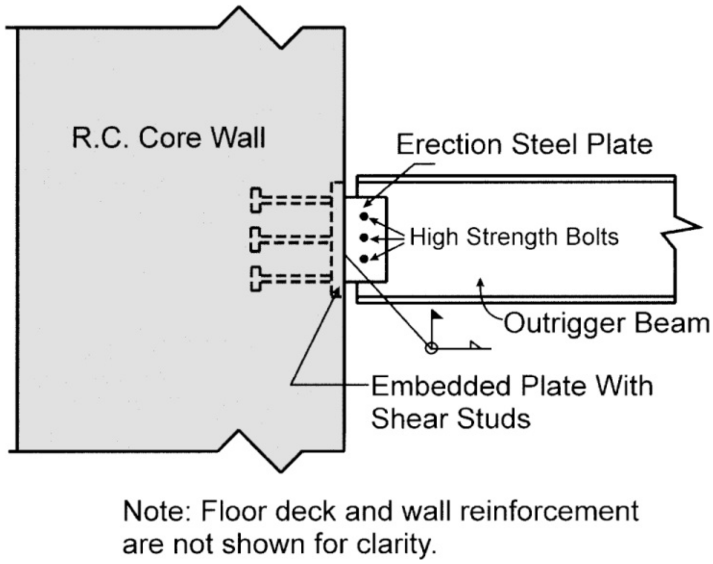


FIGURE 10.7 Shear connection between outrigger beams and walls.

In taller buildings, moment connections are needed to engage the perimeter columns as a means of reducing lateral deformation of the structural system. For short-span outrigger beams, a sufficient level of stiffness can be achieved by a single structural member (either a built-up or a rolled section). In such cases, a number of different moment-resisting connection details are possible. The detail shown in Fig. 10.8 is suitable for developing small moments (clearly not the full moment capacity of the beam) as found by Roeder and Hawkins (1981) and Hawkins et al. (1980). A larger moment can be resisted by embedding the outrigger beam in the wall during construction, similar to that shown in Fig. 10.5 or 10.6, or by using the detail shown in Fig. 10.9. In the latter option, the outrigger beam is welded to a plate which is anchored in the wall by an embedded structural element similar to the outrigger beam. The latter detail is suitable for slip-formed core walls, as well as for conventional construction methods. These details rely on developing an internal couple due to bearing of the beam flanges against the surrounding concrete or grout. If the wall boundary is reinforced with a structural column, the outrigger beam can be directly attached to the wall, as shown in Fig. 10.3 or 10.4.

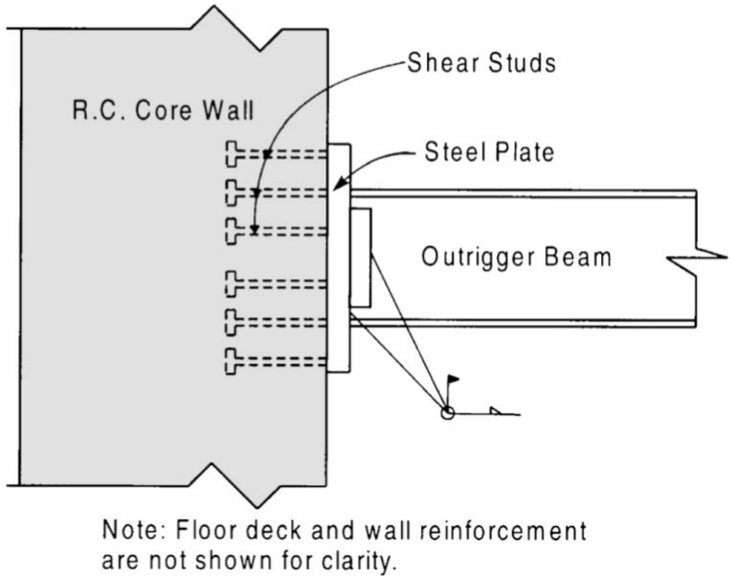


FIGURE 10.8 Moment connection between outrigger beams and walls (small moments).

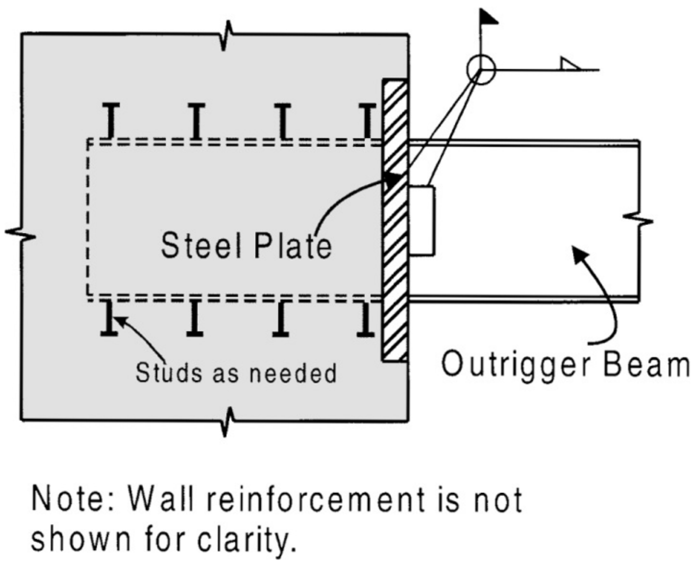


FIGURE 10.9 Moment connection between outrigger beams and walls (large moments).

The span of most outrigger beams is such that a single girder does not provide adequate stiffness, and other systems are needed. Story-deep trusses are a viable choice. As shown schematically in Fig. 10.10, the connection between the top and bottom chords is essentially similar to that used for shear connections between outrigger beams and wall piers.

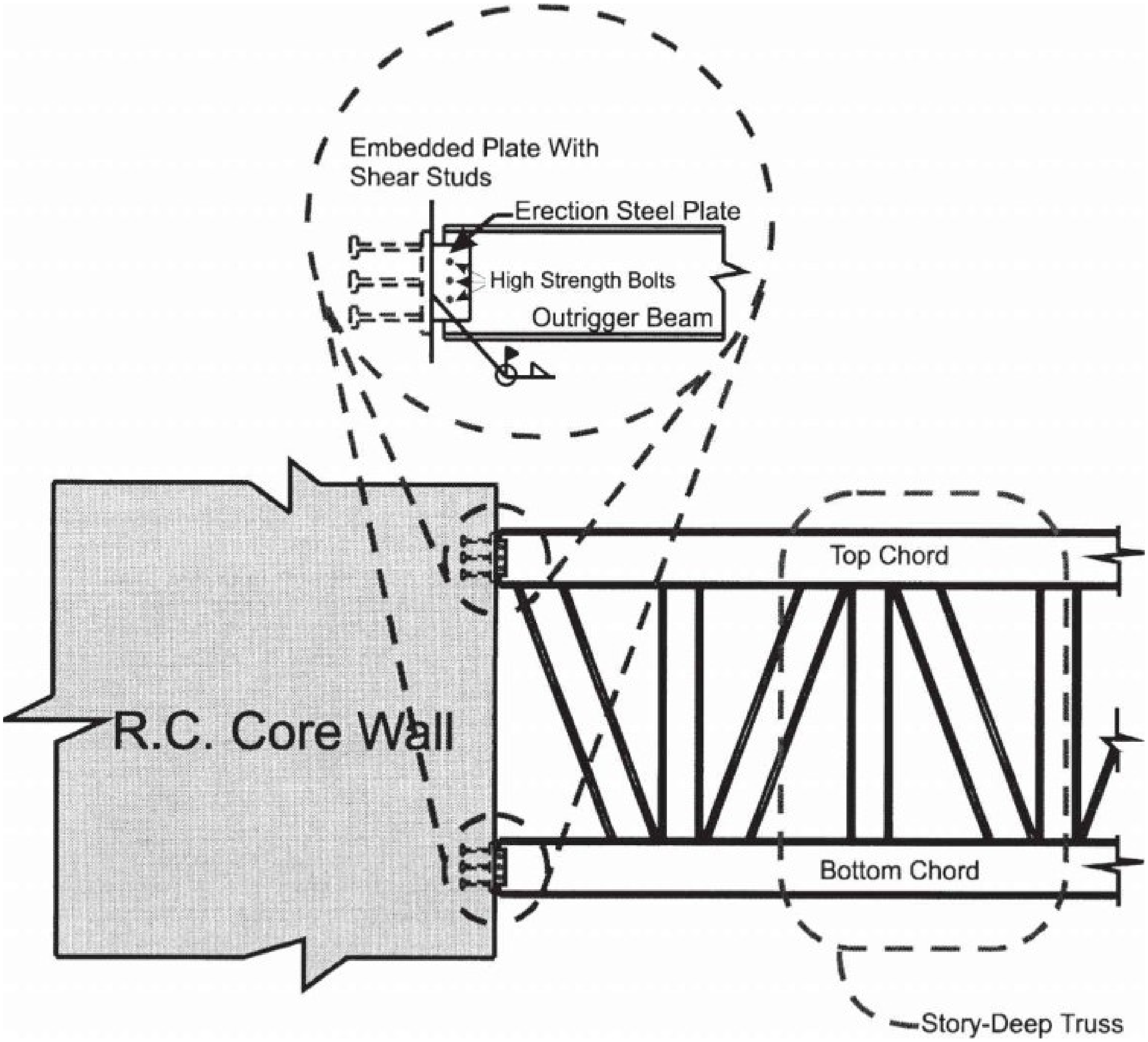


FIGURE 10.10 Connection between story-deep trusses and walls.

10.3.2.4 Floor-Wall Connection. A common component for either of the connections discussed previously is the connection between the floor and walls. In hybrid structures, the floor system consists of a composite metal deck. When the metal deck corrugations are parallel to the core, continuous bent closure plates are placed to prevent slippage of concrete during pouring. These plates may consist of continuous angles, as shown in Fig. 10.11, which are either attached to weld plates already cast in the wall, or anchored directly to the wall. When the metal deck corrugations are perpendicular to the core, the deck is supported by steel angles which are attached to the core typically at 12 to 24 in on center (Fig. 10.11*b*). In addition, dowels at regular intervals (18 in on center is common) are used to transfer lateral loads into the core. Note that for encased coupling beams (that is, steel-concrete composite members), the floor system is a reinforced concrete slab or posttensioned system. For these cases, the floor-wall connection is similar to reinforced concrete slab or posttensioned floor-wall connections.

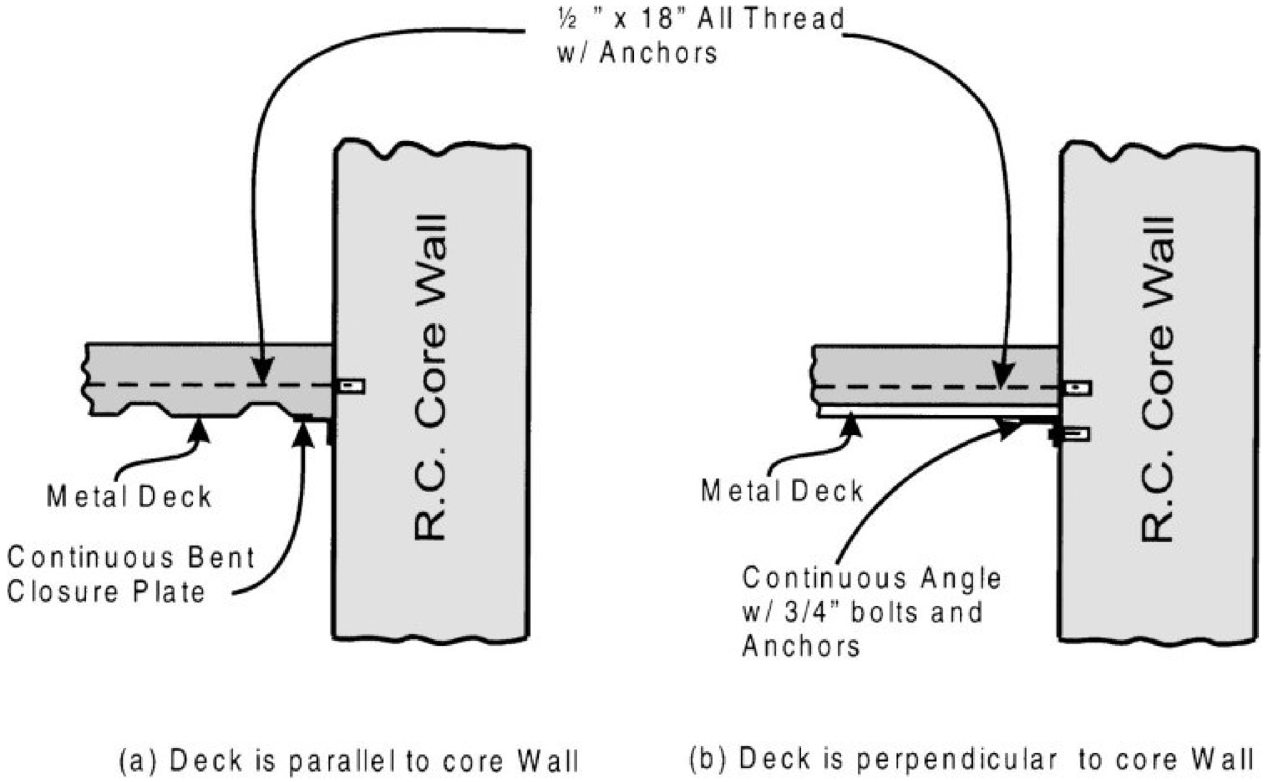


FIGURE 10.11 Representative connection between composite floors and core walls.

10.3.3 Design of Steel or Steel-Concrete Composite Coupling Beam-Wall Connections

10.3.3.1 Analysis. Accurate modeling of coupled wall systems is a critical step, particularly when steel or steel-concrete composite coupling beams are used. Previous studies (Shahrooz et al., 1992, 1993; Gong and Shahrooz, 1998) suggest that steel or steel-concrete composite coupling beams are not fixed at the face of the wall. As part of design calculations, the additional flexibility needs to be taken into account to ensure that wall forces and lateral deflections are computed reasonably well. Based on experimental data (Shahrooz et al., 1992, 1993; Gong and Shahrooz, 1998), the *effective fixed point* of steel or steel-concrete composite coupling beams may be taken as one-third of the embedment length from the face of the wall. The corresponding design model is illustrated in Fig. 10.12.

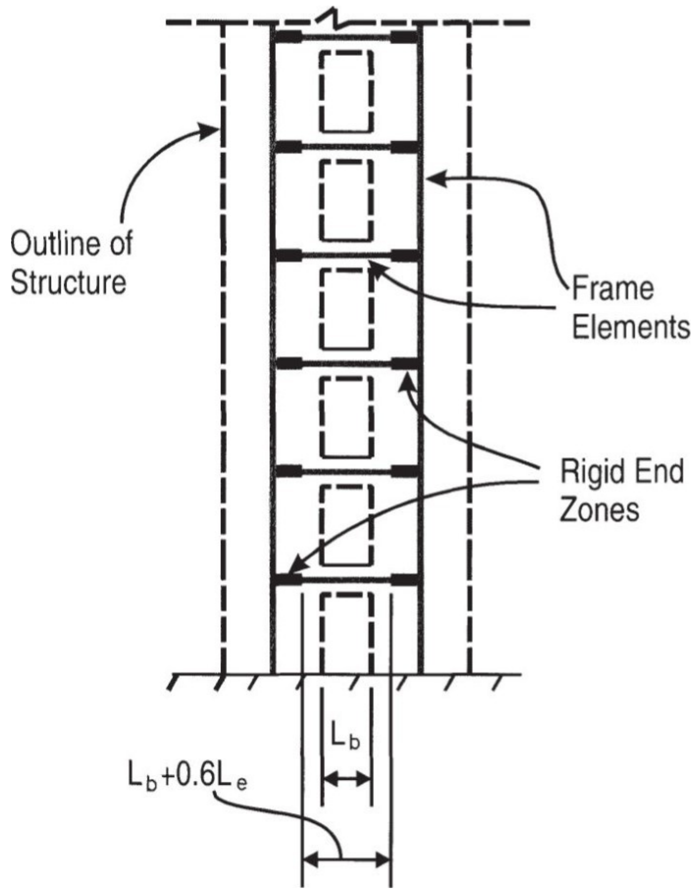


FIGURE 10.12 Design model for coupled wall systems using steel or composite coupling beams.

Stiffness of coupling beams needs to be estimated properly as the design forces and hence detailing of coupling beam-wall connection are impacted. For steel coupling beams, standard methods are used to calculate the stiffness. The stiffness of steel-concrete composite coupling beams needs to account for the increased stiffness due to encasement. Stiffness based on gross transformed section should be used to calculate the upper-bound values of demands in the walls, most notably wall axial force. Cracked transformed section moment of inertia may be used when deflection limits are checked and to compute the maximum wall overturning moment. Note that a previous study suggests that the additional stiffness due to floor slab is lost shortly after composite coupling beams undergo small deformations (Gong and Shahrooz, 1998). Until additional experimental data become available, it is recommended to include the participation of the floor slab for calculating wall axial force. Effective flange width for T beams, as specified in American Concrete Institute (ACI 318), may be used for this purpose. The participation of floor slab toward the stiffness of steel-concrete composite coupling beams may be ignored when drift limits are checked or when the maximum wall overturning moments are computed.

10.3.3.2 Design of Coupling Beam

Steel Coupling Beams. Well-established guidelines for shear links in eccentrically braced frames (AISC, 2005) may be used to design and detail steel coupling beams. The level of coupling beam rotation angle plays an important role in the number and spacing of stiffener plates which may have to be used. This angle is computed with reference to the collapse mechanism shown in Fig. 10.13 which corresponds to the expected behavior of coupled wall systems, that is, plastic hinges form at the base of walls and at the ends of coupling beams. The value of θ_p is taken as $0.4Rq_e$ in which elastic interstory drift angle θ_e is computed under code level lateral loads (for example, NEHRP, 1994; UBC, 1994). The minimum value of the term $0.4R$ is 1.0. Knowing the value of θ_p, shear angle γ_p is calculated from Eq. (10.1):

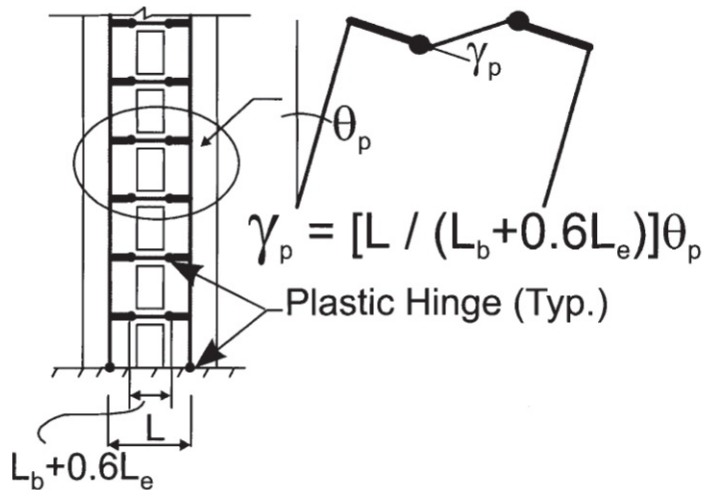


FIGURE 10.13 Model for calculating shear angle of steel or composite coupling beams.

$$\gamma_p = \frac{\gamma_p L}{L_b + 0.6 L_e} \tag{10.1}$$

Note that in this equation, the additional flexibility of steel/composite coupling beams is taken into account by increasing the length of the coupling beam to $L_b + 0.6L_e$. This method is identical to that used for calculating the expected shear angle in shear links of eccentrically braced frames with the exception of the selected collapse mechanism.

Steel-Concrete Composite Coupling Beams. Previous research on steel-concrete composite coupling beams (Shahrooz et al., 1992, 1993; Gong and Shahrooz, 1998) indicates that nominal encasement around steel beams provides adequate resistance against flange and web buckling. Therefore, composite coupling beams may be detailed without web stiffener plates. Due to inadequate data regarding the influence of encasement on local buckling, minimum flange and web thicknesses similar to steel coupling beams need to be used.

10.3.3.3 Connection Design. The connection becomes more critical when steel or steel-concrete composite coupling beams are used. For the details shown in Fig. 10.3 or 10.4, standard design methods for steel beam-column connections can be followed. If the connection involves embedding the coupling beam inside the wall (see Fig. 10.5 or 10.6), the required embedment length is calculated based on mobilizing the moment arm between bearing forces C_f and C_b, as shown in Fig. 10.14. This model was originally proposed by Mattock and Gaafar (1982) for steel brackets embedded in precast concrete columns. Previous studies (Shahrooz et al., 1992, 1993; Gong and Shahrooz, 1998) have shown the adequacy of this model for steel or steel-concrete composite coupling beams. This model calculates the required embedment length, L_e, from Eq. (10.2):

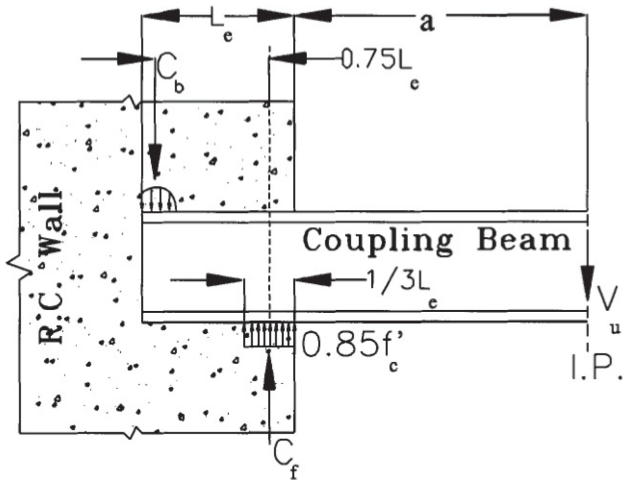


FIGURE 10.14 Model for computing embedment length.

$$V_u = 48.6\sqrt{f_c'}\left(\frac{t_{\text{wall}}}{b_f}\right)^{0.66} \beta_1 b_f L_e \left(\frac{0.58 - 0.22\beta_1}{0.88 + (\alpha / L_e)}\right) \tag{10.2}$$

For the detail shown in Fig. 10.6, the value of f_c' in Eq. (10.2) is to be taken as the minimum of the compressive strength of the wall and grout.

The value of V_u in Eq. (10.2) should be selected to ensure that the connection does not fail prior to fully developing the capacity of the coupling beam. For steel coupling beams, V_u is taken as the plastic shear capacity of the steel member as computed from Eq. (10.3):

$$V_p = 0.6F_y\,(h - 2t_f)\,t_w \tag{10.3}$$

To account for strain hardening, it is recommended that F_y be taken as 1.25 times the nominal yield strength.

The contribution of encasement toward shear capacity of composite coupling beams needs to be taken into account when the embedment length is calculated. Embedment length should be adequate such that most of the input energy is dissipated through formation of plastic hinges in the beam and not in the connection region (Shahrooz et al., 1992, 1993; Gong and Shahrooz, 1998). In lieu of fiber analysis, shear capacity of composite coupling beams V_u can be computed from Eq. (10.4), which has been calibrated based on a relatively large number of case studies (Gong and Shahrooz, 1998):

$$\begin{aligned} V_u &= 1.56(V_{\text{steel}} + V_{\text{RC}}) \\ V_{\text{Steel}} &= 0.6F_y t_w (h - 2t_f) \\ V_{\text{RC}} &= 2\sqrt{f_c'}\,b_w d + \frac{A_v f_y d}{s} \end{aligned} \tag{10.4}$$

In this equation, the nominal values of F_y and f_c' (in psi) are to be used because the equation has been calibrated to account for strain hardening and material overstrength.

Additional bars attached to the beam flanges (*transfer bars*) can contribute toward load resistance. These bars can be attached through mechanical half couplers which have been welded onto the flanges. The embedment length as computed by Eq. (10.2) can be modified to account for the additional strength (Gong and Shahrooz, 1998). However, to ensure that the calculated embedment length is sufficiently large to avoid excessive inelastic damage in the connection region, it is recommended that the contribution of transfer bars be neglected.

A pair of stiffener plates (on both sides of the web) placed along the embedment length will mobilize compression struts in the connection region as depicted schematically in Fig. 10.15. These stiffener plates are commonly referred to as *face-bearing plates*. The first face-bearing plate should be inside the confined core of the wall boundary element. The distance between the face-bearing plates should be such that the angle of compression struts is approximately 45° (hence, the distance between the plates should be about equal to the clear distance between the flanges). To ensure adequate contribution of the face-bearing plates, the width of each face-bearing plate should be equal to the flange width on either side of the web. The thickness of the face-bearing plates can be established based on available guidelines for the detailing of shear links in eccentrically braced frames (AISC, 2005).

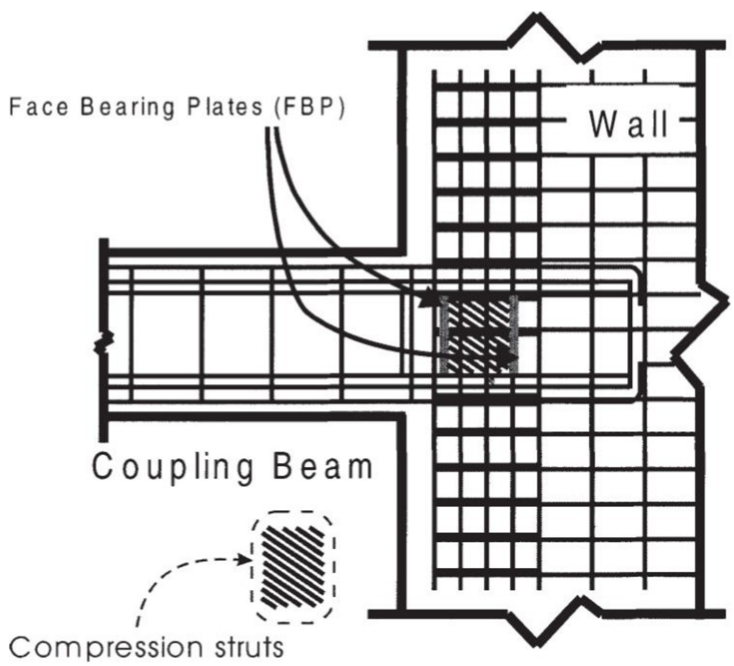


FIGURE 10.15 Face-bearing plates.

10.3.3.4 Design Example. An example is used to illustrate the procedure for computing the required embedment length of steel or steel-concrete composite coupling beams. A representative connection at floor 7 of the structure shown in Fig. 10.16 is designed in this example. The building in this example has 20 floors. The coupling beams are encased, that is, composite, and the walls are assumed to be reinforced only with longitudinal and transverse reinforcement. The clear span of the coupling beam is 8 ft. The thickness of the wall boundary element, t_{wall}, is 22 in. The material properties are f'_c (for the encasement as well as the wall) = 4 ksi, F_y (yield strength of the web of the steel coupling beam) = 40 ksi, and f_y (yield strength of reinforcing bars in the encasement) = 60 ksi. The cross section of the coupling beam is shown in Fig. 10.17. The effective depth for the concrete element is taken as 21.5 in. The goal is to compute the required embedment length of the steel coupling beam inside the reinforced concrete wall.

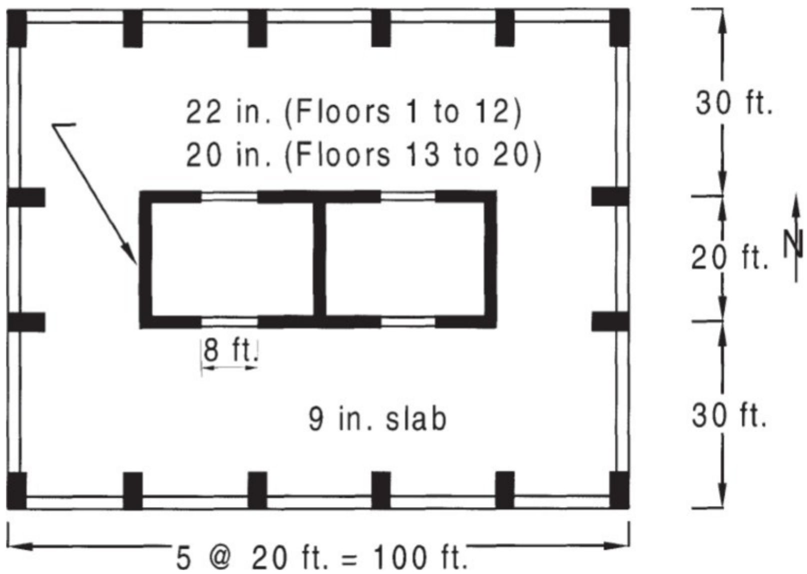


FIGURE 10.16 Floor plan of example structure.

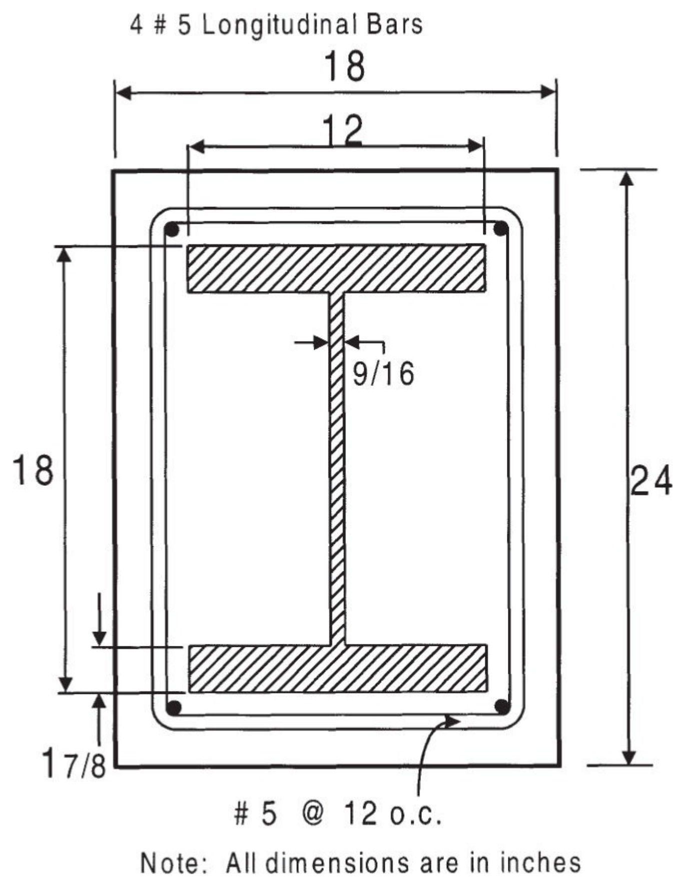


FIGURE 10.17 Cross section of composite coupling beam.

The embedment length needs to develop $V_u = 1.56\,(V_{\mathrm{steel}} + V_{\mathrm{RC}})$. The values of V_{RC} and V_{steel} are computed in Eq. (10.5):

$$V_{RC} = 2\sqrt{f_c'}\,b_w d + \frac{A_s f_y d}{s}$$

$$V_{RC} = 2\frac{\sqrt{4000}}{1000}(18)(21.5) + \frac{2(0.31)(60)(21.5)}{12}$$

$$V_{RC} = 116 \text{ kips} \tag{10.5}$$

$$V_{steel} = 0.6F_y t_w (d - 2t_f)$$

$$V_{steel} = 0.6(40)(9/16)[18 - (2)(1.875)]$$

$$V_{steel} = 193 \text{ kips}$$

The embedment length is designed to develop $V_u = 1.56(116 + 193) = 480$ kips.

$$V_u = 48.6\sqrt{f_c'}\left(\frac{t_{wall}}{b_f}\right)^{0.66} ß_1 b_f L_e \left(\frac{0.58 - 0.22ß_1}{0.88 + (\alpha / L_e)}\right) \tag{10.6}$$

Therefore,

$$480 = 48.6\frac{\sqrt{4000}}{1000}\left(\frac{22}{12}\right)^{0.66}(0.85)(12)L_e\left(\frac{0.58 - 0.22(0.85)}{0.88 + (48/L_e)}\right) \tag{10.7}$$

By solving Eq. (10.7), the required embedment length is 48.6 in, say 49 in. The final detail is shown in Fig. 10.18. Note that 1.5-in transfer bars have been added to the top and bottom flanges as shown.

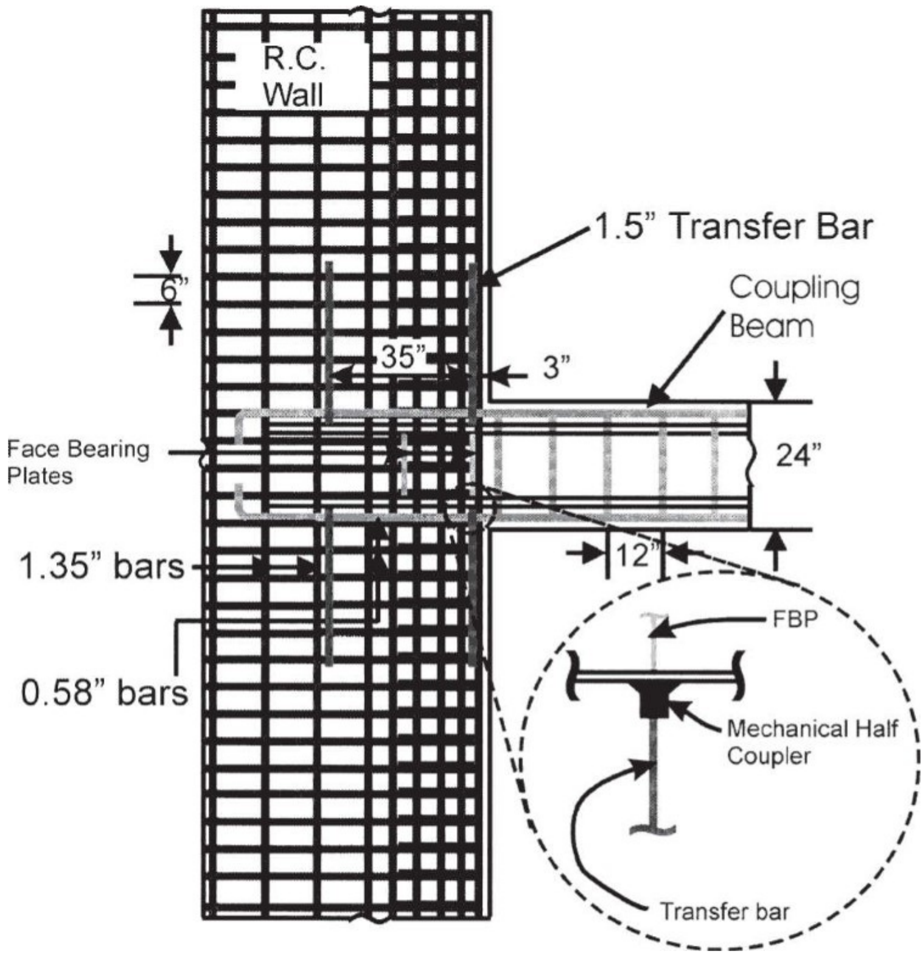


FIGURE 10.18 Connection detail.

10.3.4 Design of Outrigger Beam-Wall Connections

10.3.4.1 Shear Connections. As explained in Sec. 10.3.2.3, outrigger beams are typically connected to core walls through *shear connections* similar to that shown in Fig. 10.7. Although this connection provides some moment resistance, it is generally accepted that the connection is flexible and does not develop large moments. The main design issues are (1) the connection between the steel outrigger beam and shear tab which is welded onto the embedded plate and (2) the transfer of forces, which are gravity shear force and diaphragm force, as shown in Fig. 10.19, to the wall. Note that the diaphragm force may be tensile or compressive, and the line of action of gravity shear is assumed to be along the bolts according to standard practice. The outrigger beam-shear tab connection is a typical shear connection, and common design methods for steel structures (AISC, 2005) are followed for this purpose. The most critical part is the transfer of forces, particularly tensile diaphragm forces, from the shear tab to the core wall, which is achieved by headed studs. To ensure adequate safety against stud failure, the following design methodology is recommended. This method is based on a research conducted by Wang (1979):

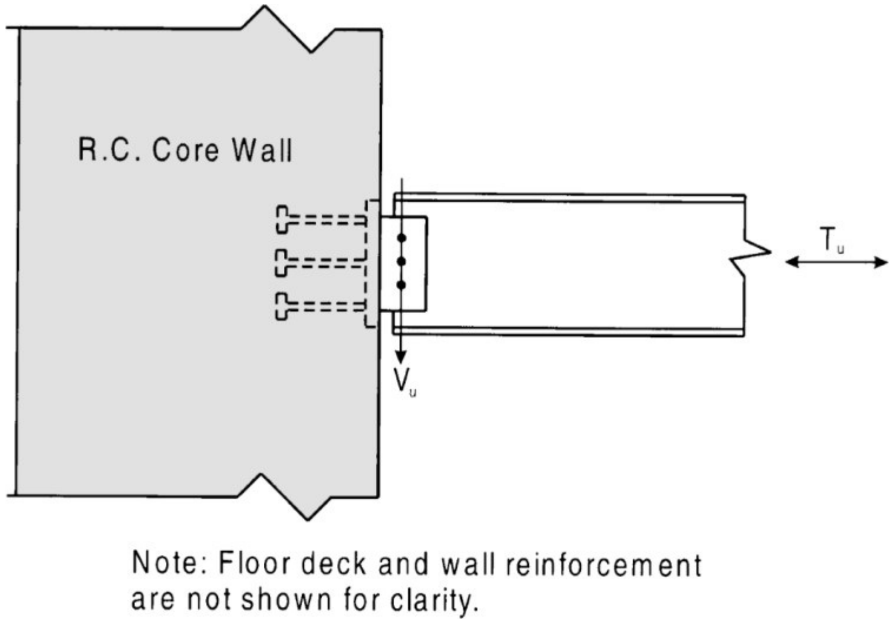


FIGURE 10.19 Forces on shear connection between outrigger/collector beams and core walls.

1. Based on an assumed layout of studs, establish the tensile capacity as the lesser of the strength of the stud or concrete cone. Available guidelines (from PCI) can be used for this purpose.
2. Assuming that all the applied shear is resisted by the studs in the compression region, calculate the required number of studs. The shear capacity is taken as the smaller of (*a*) shear capacity of a single stud, which can be calculated based on available guidelines (PCI) and (*b*) tensile capacity calculated in step (1). Use the same number of studs in the tension zone. Once the required number of studs is known, compute shear strength governed by concrete failure.
3. Using the stud arrangement obtained in step (2), compute tensile capacity of the stud group.
4. Increase the value of T_u by 50% to ensure adequate ductility.
5. Based on the model shown in Fig. 10.20 and the formulation shown in Eq. (10.8), calculate the depth of compression region, k_d:

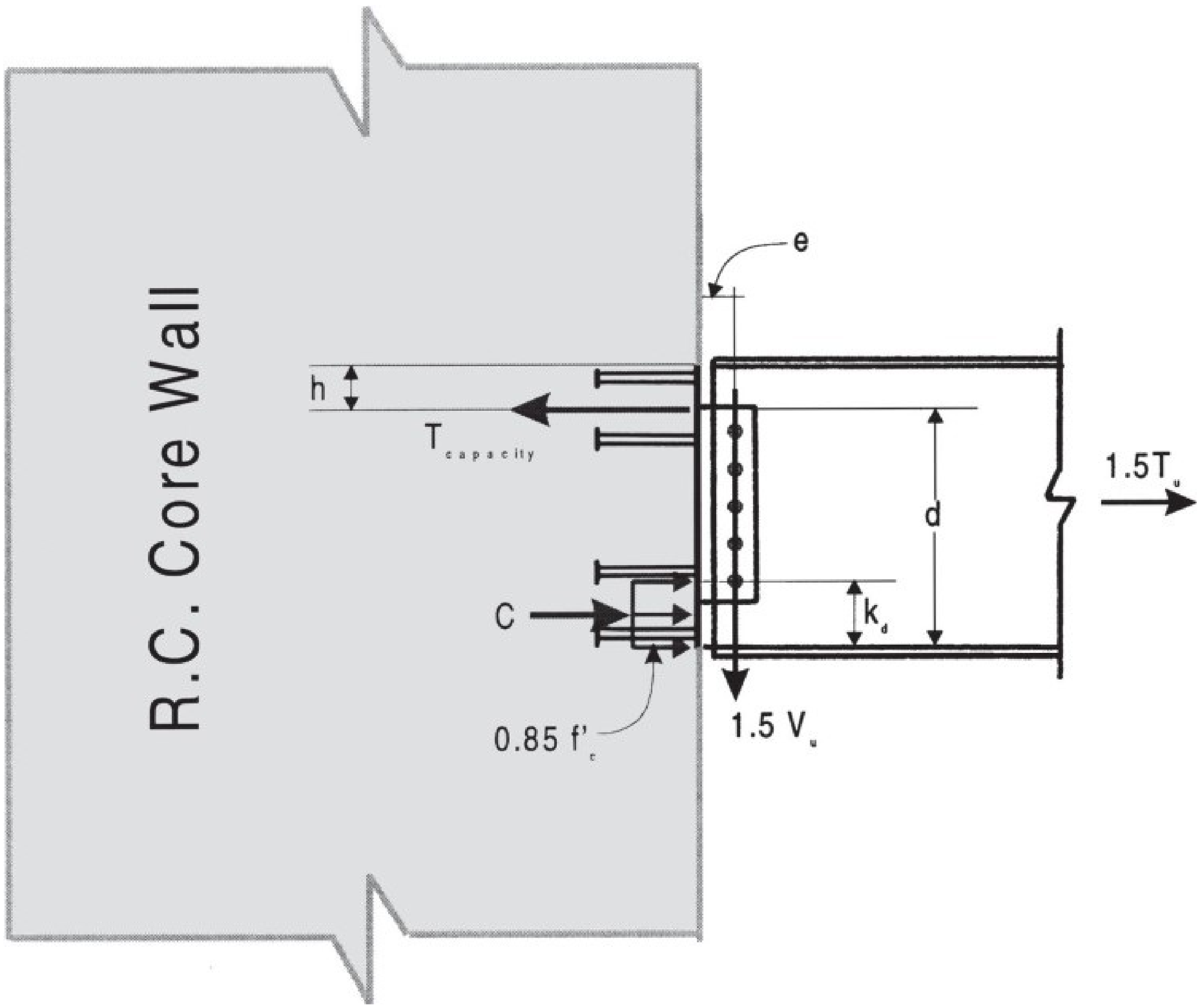


FIGURE 10.20 Design model for design of outrigger beam-wall shear connections.

$$k_d = \frac{T_{\text{capacity}} - 1.5T_u}{(0.85 f_c')b} \tag{10.8}$$

6. Calculate the required depth of the embedded plate from Eq. (10.9):

$$d = \frac{1.5eV_u + 0.425bf_c'k_d^2 + 0.75T_u h}{0.85bf_c'k_d + 0.75T_u} \tag{10.9}$$

$$\text{depth} = d + h$$

Note that in this equation, the value of gravity shear V_u is amplified by 1.5 to ensure a ductile mode of failure.

7. Check the capacity of studs under combined actions of tension and shear. For this purpose, the shear may be assumed to be resisted equally by the tension and compression studs, but the tensile force is resisted by tension studs. Available interaction equations in PCI guidelines can be used for this purpose.

10.3.4.2 Moment Connections. As mentioned previously in Sec. 10.3.2.3, outrigger beams may be attached to core walls through moment connections to enhance the overall structural stiffness. The basic force-transfer mechanism for the connections shown in Fig. 10.2, 10.5, 10.6, or 10.9 is similar to that discussed for coupling beams embedded inside core walls. For the connection shown in Fig. 10.8, the aforementioned design procedure for shear connections can generally be followed, but the term, $1.5eV_u$, in Eq. (10.9) is replaced by $1.5M_u$. Once again, the calculated design moment, M_u, has been increased by 50% to ensure a ductile behavior. The connections for top and bottom chords of story-deep outrigger trusses (Fig. 10.10) are similar to shear connections, and are designed according to the formulation described in Sec. 10.3.4.1.

10.3.4.3 Floor-Wall Connections. In a structure with the floor plan shown in Fig. 10.1, it is possible to transfer diaphragm forces directly to core walls through the outrigger beams, which also serve as collector elements. In such cases, the connection between composite floor systems and core walls, which were discussed in Sec. 10.3.2.4, has to simply resist the gravity shear. The connection between the necessary supporting elements and core walls is designed according to established guidelines (from PCI). To reduce the demands on outrigger beam-wall connections, the floor system may be designed to participate in the transfer of diaphragm forces to the core walls. Dowels at regular spacing can be used for this purpose. The dowels have to be embedded adequately in the floor slab, and be anchored to the wall so that their capacity can be developed. These dowels have to resist the portion of tensile diaphragm force not resisted by the collector element.

10.3.4.4 Design Example for Shear Connections. An example of shear connections between outrigger beams and core walls is illustrated in this section. The example is with reference to a 15-story building with the floor plan shown in Fig. 10.21. The calculated forces for the outrigger beam in floor 5 are T_u = 40 kips and V_u = 93 kips. The outrigger beam is W 24 × 55, and the core walls are 18 in thick. The concrete compressive strength of the wall is 6000 psi.

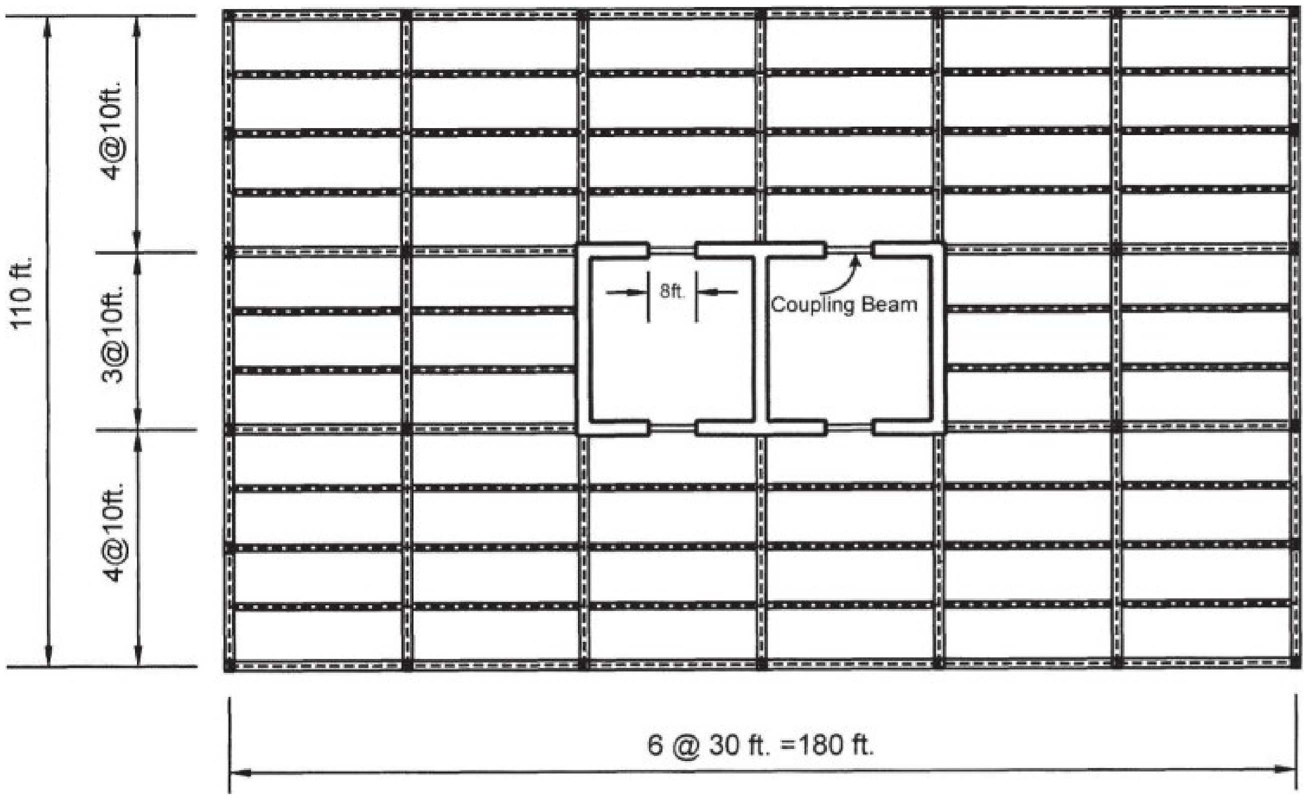


FIGURE 10.21 Plan view of design example.

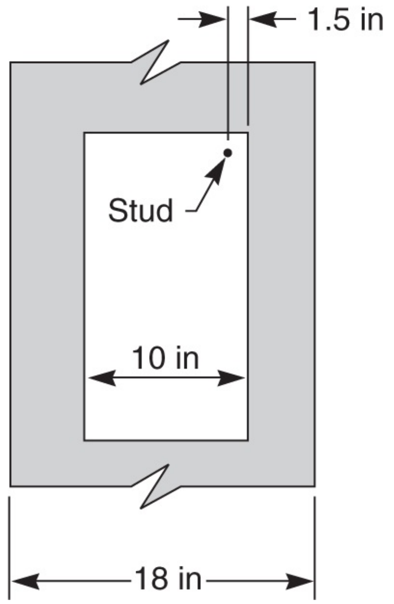
- *Design of shear tab:* The shear tab is designed and detailed by following standard design practice for steel structures (AISC, 2005). The shear tab dimensions are 15.5 in deep × 4.5 in wide × ½ in thick. The shear tab is welded to the embedded steel plate through ¼-in fillet weld. Five 1-in A490 bolts are used to connect the outrigger beam to the shear tab.
- *Design of embedded steel plate:* Try ¾-in-diam studs with 7 in of embedment:

1. *Tensile capacity of studs:*

$$\phi P_s = \phi 0.9 A_b f_y = (1)(0.9)(0.4418)(60) = 23.85 \text{ kips}$$

Assuming that the stud is located as shown below, the tensile strength governed by concrete failure is

$$\phi P_c = 10.7 l_e (l_e + d_h)\sqrt{f_c'}\,\frac{d_e}{l_e}$$



in which $l_e = 7\frac{3}{8} = 6.625$ in

$d_e = 5.5$ in

$\phi P_c = 10.7(6.625)(6.625 + 1.25)\sqrt{6000}\,(5.5/6.625)$

$\phi P_c = 35{,}900 \text{ lb} = 35.9 \text{ kips}$

Therefore, use ϕP_s.

2. *Shear strength:*

$$\phi V_s = \phi 0.75 A_b f_y = (1)(0.75)(0.4418)(60) = 19.9 \text{ kips}$$

The shear strength is the smaller of ϕV_s and tensile strength. Hence, shear strength = 19.9 kips. The number of required studs = 93/19.9 = 4.7, say 5 studs. Compute shear strength governed by concrete failure. Since the edge distance > $15d_b$ (= 11.25 in),

$$\phi V_c = \phi 800\, A_b \sqrt{f_c'}\, n$$

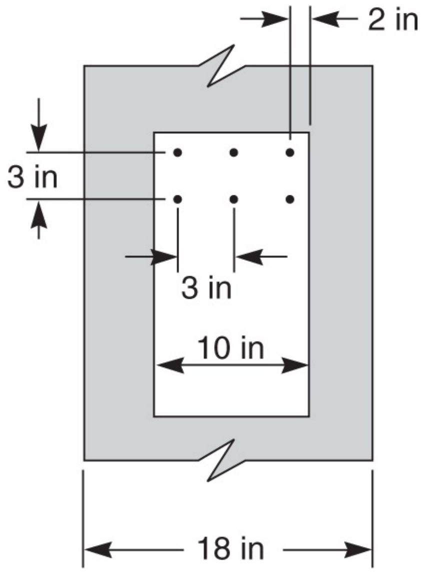
Therefore, $\phi V_c = 0.85(800)(0.4418)(\sqrt{6000})5 = 116354 \text{ lb} = 116.4$ kips, which is larger than V_u, ok. To have an even number, use six studs in both tension and compression zones.

3. *Tensile strength of stud groups:* Assuming the stud pattern shown below, the capacity is computed from the following equation:

$$\phi P_c = \phi(4)\sqrt{f_c'}\,(x + d_{e1} + d_{e2})(y + 2l_e)$$

$$\phi P_c = 0.85(4)(\sqrt{6000})(6 + 6 + 6)[3 + 2(6.625)] = 77{,}033 \text{ lb}$$

$$\phi P_c = 77.0 \text{ kips} > 1.5T_u, \text{ ok}$$



4. *Size the embedded plate:* From Eq. (10.8),

$$k_d = \frac{77.0 - 1.5(40)}{0.85(6)(10)} = 0.334 \text{ in}$$

Assuming that the plate extends 1 in above the top stud, the value of h in Eq. (10.9) is 2.5 in. As seen from Fig. 10.22, the value of e = 2.75 in. Use Eq. (10.9) to solve for d:

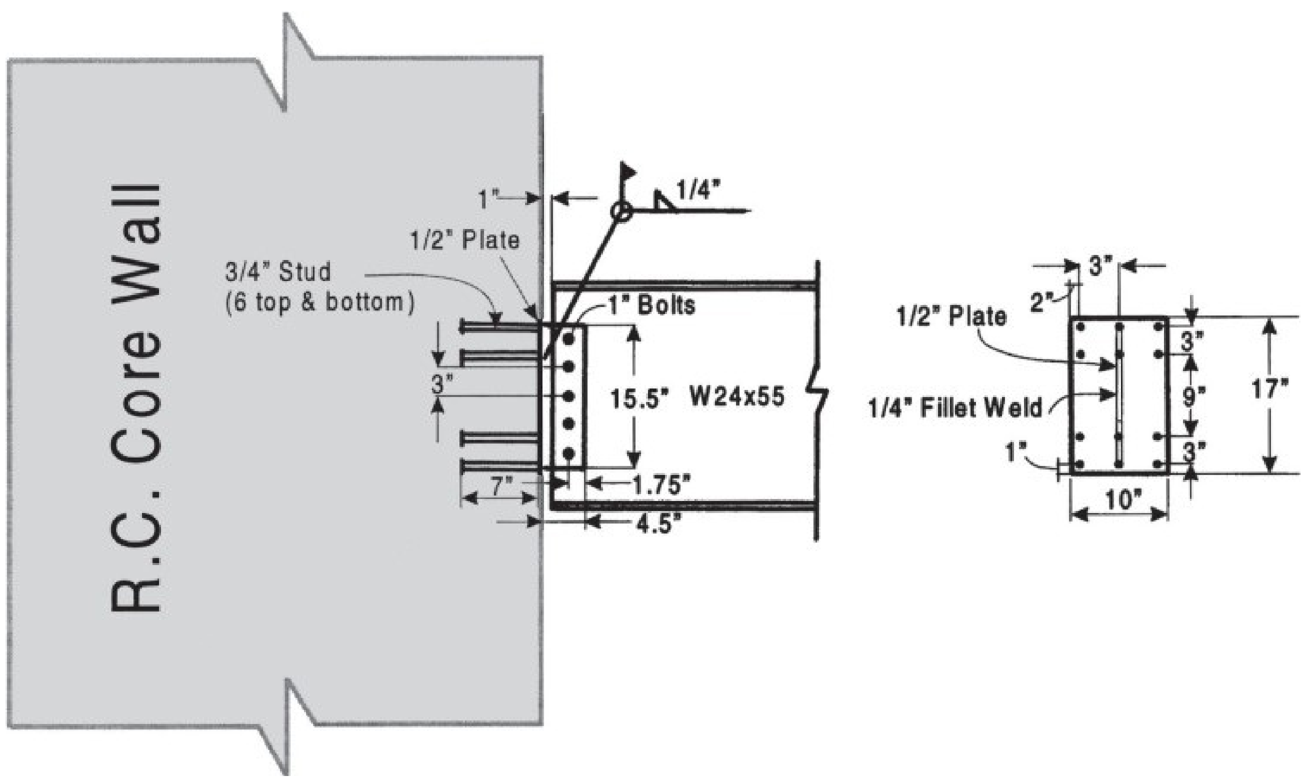


FIGURE 10.22 Detail of shear connection between outrigger/collector beam and core wall.

$$d = \frac{1.5(93)(2.75) + 0.425(6)(0.334)^2(10) + 0.75(40)(2.5)}{0.75(40) + 0.85(6)(0.334)(10)} = 9.76 \text{ in}$$

Therefore, the depth of the embedded plate is $d + h = 2.5 + 9.76 = 12.3$ in, say 12.5 in.

Note that this depth is less than that required for the shear tab. Assuming that the embedded plate extends ¾ in beyond the shear tab, the required depth of the embedded plate is 0.75 + 15.5 + 0.75 = 17 in.

According to PCI guidelines, the plate thickness is taken as two-thirds of the diameter of the stud. Hence, the plate thickness is 0.5 in.

The final design is sketched in Fig. 10.22.

5. *Check the studs for combined effects of shear and tension:* Use the following interaction equations recommended by PCI:

$$\frac{1}{\phi}\left[\left(\frac{T}{P_c}\right)^2+\left(\frac{V_u}{V_c}\right)^2\right]\leq 1.0$$

$$\left[\left(\frac{T}{P_s}\right)^2+\left(\frac{V_u}{V_s}\right)^2\right]\geq 1.0$$

Using the free-body diagram shown in Fig. 10.23, the value of T can be computed as follows:

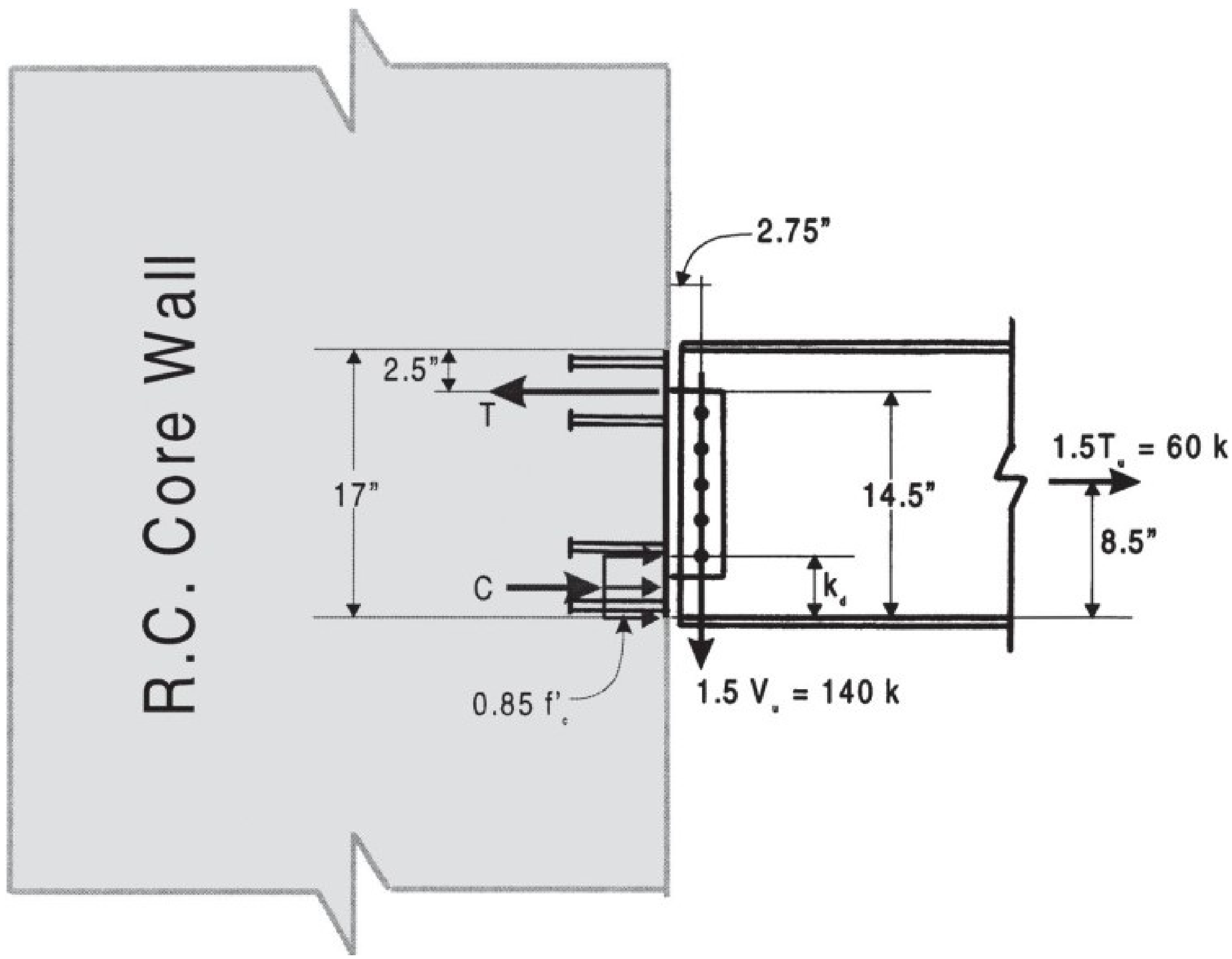


FIGURE 10.23 Free-body diagram to check final design.

$$\Sigma F_x = 0;$$

$$0.85 f'_c\, k_d\, b + 1.5 T_u - T = 0.85(6)(10)\, k_d + 1.5(40) - T = 0$$

$$T = 51\ k_d + 60$$

$$\Sigma M_{\text{about } T} = 0$$

$$1.5 T_u\,(14.5 - 8.5) + 0.85 f'_c\, k_d\, b \left(14.5 - \frac{k_d}{2}\right) - 1.5 V_u e = 0$$

$$1.5(40)(6) + 0.85(6)(k_d)10(14.5 - 0.5 k_d) - 1.5(93)(2.75) = 0$$

$$25.5\, k_d^2 - 739.5 k_d + 23.63 = 0$$

$$k_d = 0.032 \text{ in}$$

Therefore, $T = 51(0.0322) + 60 = 61.6$ kips

$$P_c = 4\sqrt{f'_c}\ (x + d_{e1} + d_{e2})(y + 21_e)$$

$$= 4\sqrt{6000}(6 + 6 + 6)[3 + 2(6.625)] = 906{,}280 \text{ lb}$$

$$= 90.6 \text{ kips}$$

$$P_s = 0.9(A_s f_y) n = 0.9(0.4418)(60)6$$

$$= 143 \text{ kips (six studs are in tension; } n = 6)$$

$$V_c = 800\ A_b \sqrt{f'_c}\ n = 800(0.4418)\sqrt{6000}(12) = 328{,}530 \text{ lb}$$

$$= 329 \text{ kips}$$

$$V_s = 0.75\ A_b f_y\ n = 0.75(0.4418)(60)(12)$$

$$= 239 \text{ kips (shear is resisted by all studs; } n = 12)$$

Therefore,

$$\frac{1}{0.85}\left[\left(\frac{61.6}{90.6}\right)^2 + \left(\frac{140}{329}\right)^2\right] = 0.76 < 1.0, \text{ ok}$$

$$\left[\left(\frac{61.6}{143}\right)^2 + \left(\frac{140}{239}\right)^2\right] = 0.53 < 1.0, \text{ ok}$$

The final design shown in Fig. 10.22 is adequate.

10.4 JOINTS BETWEEN STEEL BEAMS AND REINFORCED CONCRETE COLUMNS

10.4.1 Introduction

Composite frames consisting of steel beams and reinforced concrete columns constitute a very cost-effective structural system, especially in tall buildings where the columns have to sustain high axial loads. Concrete columns are known to be more cost-effective than structural steel columns under axial loads. On the other hand, steel beams have the advantages of faster construction and no formwork or shoring required. The combination of concrete columns and steel beams in one system results in the most efficient use of the materials. However, to achieve the full advantage of such system, the beam-column connection must be properly detailed and designed. Due to the current separation of the concrete and steel specifications, the need arises for guidelines to design such connections. The ASCE Task Committee (1994) on Design Criteria for Composite Structures in Steel and Concrete presented guidelines for the design moment resisting joints where the steel beams are continuous through the reinforced concrete column. These guidelines are based on the experimental study by Sheikh et al. (1989) and Deierlein et al. (1989), where 15 two-thirds scale joint specimens were tested under monotonic and cyclic loading. The recommendations were also based on relevant information from existing codes and standards. The following sections summarize the ASCE guidelines. For more information, the reader is referred to the paper by the ASCE Task Committee (1994).

10.4.2 Joint Behavior

The joint behavior depends on joint details that activate different internal force transfer mechanisms. Failure of the joint can happen in either one of the two primary failure modes shown in Fig. 10.24. The first mode is the panel shear failure, which results from the transmission of the horizontal flange forces through the joint. Both the steel web and concrete panel contribute to the horizontal shear resistance in the joint. Attachments that mobilize the concrete panel are discussed in the next section. The second mode is the vertical bearing failure, which results from the high bearing stresses of the compression flange against the column. The joint should be detailed and designed to eliminate the possibility of joint failure and force the failure to occur in the connected members.

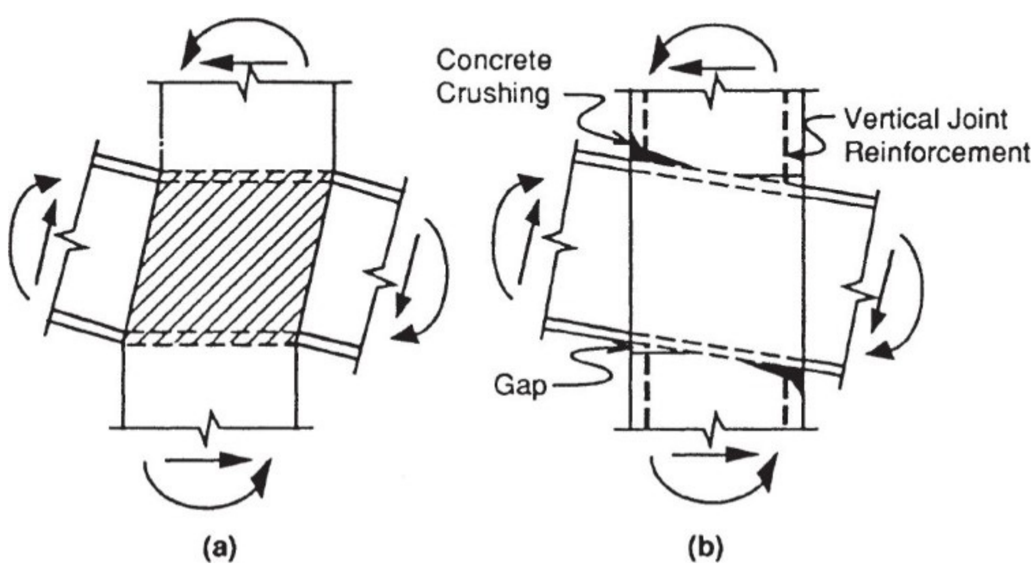


FIGURE 10.24 Joint failure modes: (*a*) panel shear and (*b*) vertical bearing (*ASCE, 1994*).

10.4.3 Joint Detailing

Several configurations of attachments can be used to improve the joint strength (see Fig. 10.25). Details shown in Fig. 10.25*a* and *b* enhance the joint shear capacity through mobilizing a greater portion of the concrete panel. The concrete panel is divided into inner and outer panels. The inner panel is mobilized by the formation of a compression strut through bearing against the FBPs between the beam flanges. Figure 10.26 shows the mobilization of the outer panel by the formation of compression field through bearing against the extended FBPs or steel columns above and below the joint. The FBP may vary in width and may be split for fabrication ease. The ASCE recommendations require that when significant moment is transferred through the beam-column connection, at least FBPs should be provided within the beam depth with the width no less than the flange width. The vertical joint reinforcement shown in Fig. 10.25*c* enhances the joint bearing capacity.

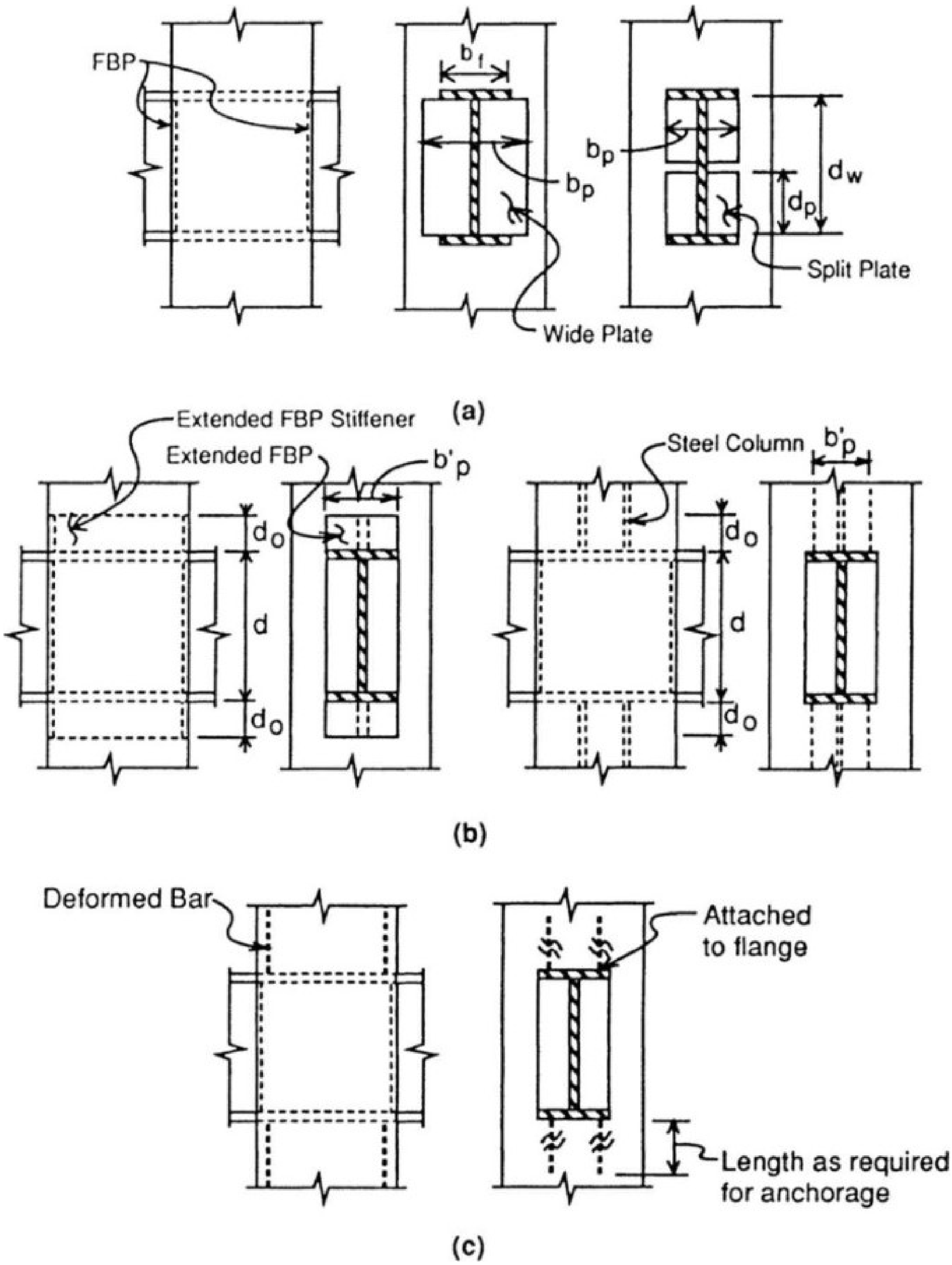


FIGURE 10.25 Joint details: (*a*) FBP; (*b*) extended FBP and steel column; (*c*) vertical joint reinforcement (*ASCE, 1994*).

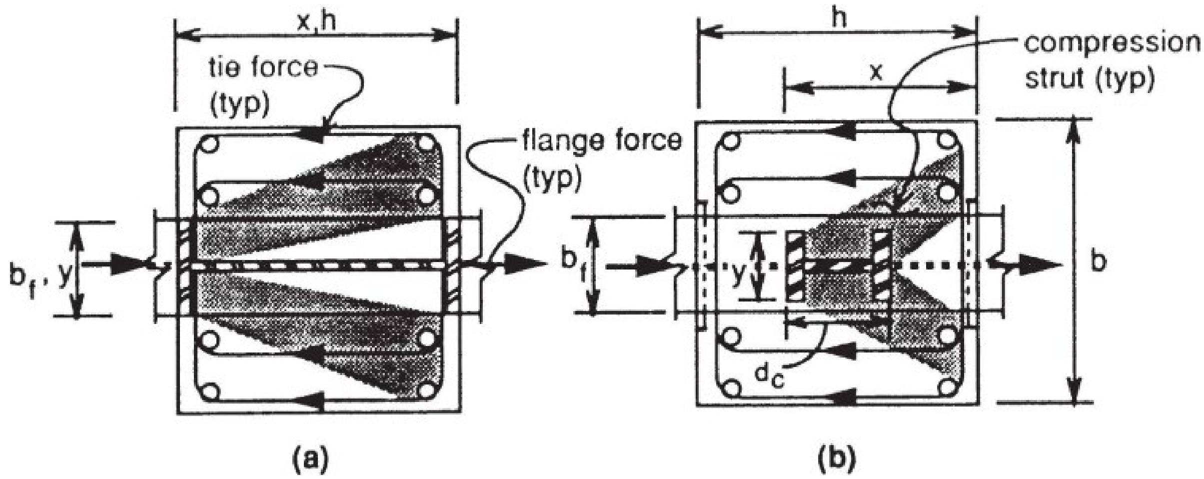


FIGURE 10.26 Transfer of horizontal force to outer concrete panel: (*a*) extended FBP and (*b*) steel column (*ASCE, 1994*).

10.4.4 Joint Forces

Various forces are transferred to the joint by adjacent members, including bending, shear, and axial loads as shown in Fig. 10.27. Existing data indicate that axial compressive forces in the column can improve the joint strength by delaying the formation of cracks. To simplify the design, and since it is conservative, the axial forces in the column are ignored. Since the axial forces in the beam are generally small, they are also neglected. Accordingly, the design forces are reduced to those shown in Fig. 10.28*a* and *b*. Considering moment equilibrium, the following equation is obtained:

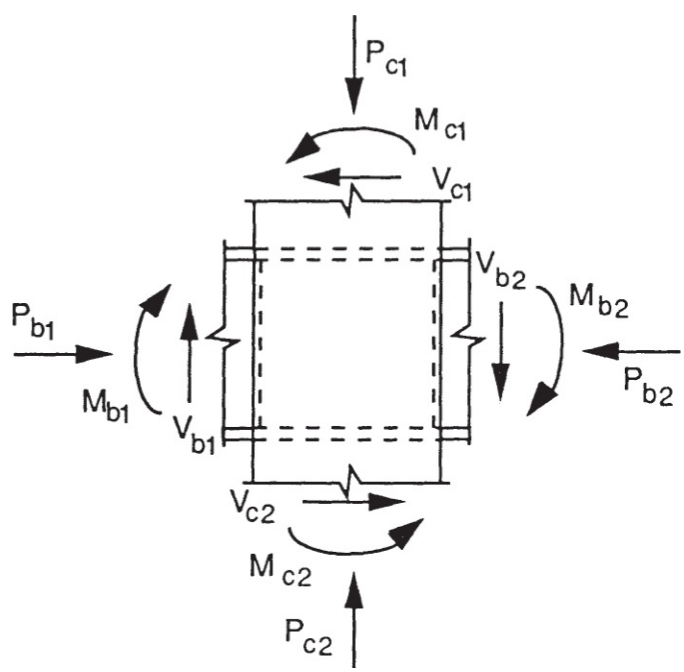


FIGURE 10.27 Forces acting on joint (*ASCE, 1994*).

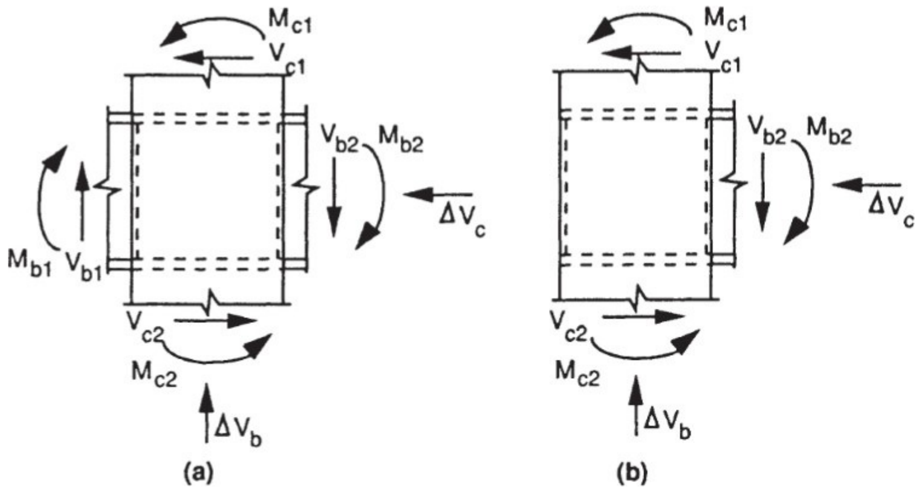


FIGURE 10.28 Joint design forces: (*a*) interior and (*b*) exterior (*ASCE, 1994*).

$$\sum M_c = \sum M_b + V_b h - V_c d \tag{10.10}$$

where

$$\sum M_b = (M_{b1} + M_{b2}) \tag{10.11}$$

$$V_b = \frac{V_{b1} + V_{b2}}{2} \tag{10.12}$$

$$V_c = \frac{V_{c1} + V_{c2}}{2} \tag{10.13}$$

$$\sum M_c = (M_{c1} + M_{c2}) \tag{10.14}$$

and

$$\Delta V_b = V_{b2} - V_{b1} \tag{10.15}$$

$$\Delta V_c = V_{c2} - V_{c1} \tag{10.16}$$

10.4.5 Effective Joint Width

The *effective width of the joint* is defined as the portion of the concrete panel effective in *resisting joint* shear. The concrete panel is divided into inner and outer panels. As shown in Fig. 10.29, the effective joint width, b_j, is equal to the sum of the inner and outer panel widths, b_i and b_o, and can be expressed as

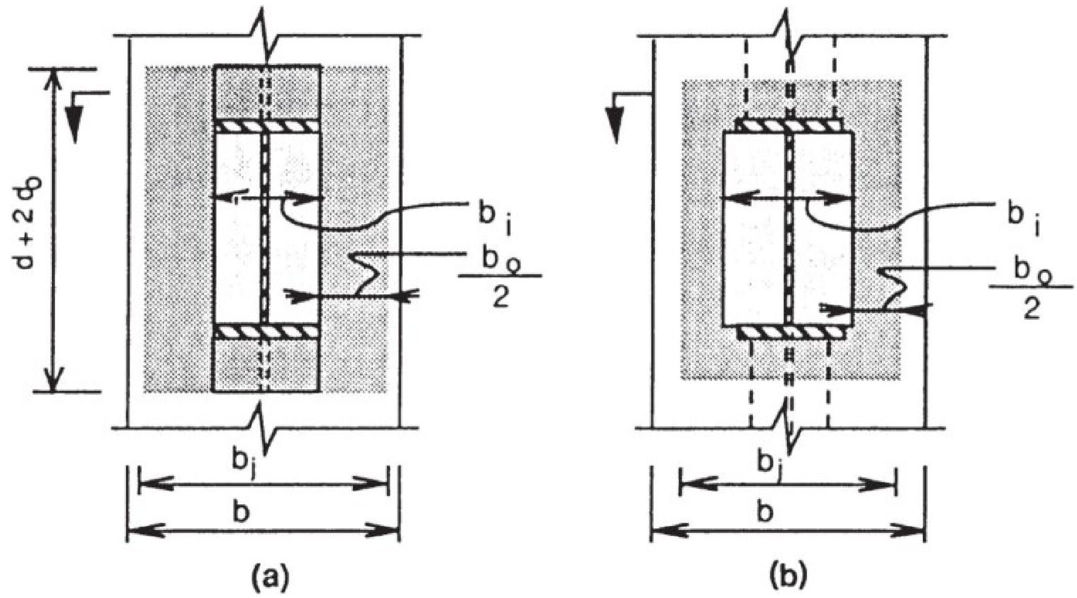


FIGURE 10.29 Effective joint width (*a*) extended FBP and (*b*) wide FBP and column (*ASCE, 1994*).

$$b_j = b_i + b_o \tag{10.17}$$

The inner width, b_i, is taken equal to the greater of the FBP width, b_p, or the beam flange width, b_f. Where neither the steel columns nor the extended FBPs are present, the outer panel width, b_o, is taken as zero. Where extended FBPs or steel columns are used, b_o is calculated according to the following:

$$b_o = C(b_m - b_i) < 2d_o \tag{10.18}$$

$$b_m = \frac{b_f + b}{2} < b_f + h < 1.75b_f \tag{10.19}$$

$$C = \frac{x}{h}\frac{y}{b_f} \tag{10.20}$$

where b = the concrete column width measured perpendicular to the beam
h = the concrete column depth
y = the greater of the steel column or extended FBP width
$x = h$, where extended FBPs are present or $x = h/2 + d_c/2$ when only the steel column is present (see Fig. 10.26)
d_c = steel column width
d_o = additional effective joint depth provided by attachment to the beam and is determined as follows: When a steel column is present, $d_o = 0.25d$, where d = beam depth; when extended FBPs are used, d_o should be taken as the lesser of $0.25d$ or the height of the extended FBPs

10.4.6 Strength Requirements

The joint strength is based on the two possible modes of failure mentioned earlier. Joint design strength is obtained by multiplying the nominal strength by a resistance factor, ϕ. Unless otherwise noted, ϕ should be taken equal to 0.70.

10.4.6.1 Vertical Bearing. Vertical forces in the joint are resisted by concrete bearing and by joint reinforcement. The equilibrium of the vertical bearing forces is shown in Fig. 10.30, where the moments in the upper and lower columns, M_{c1} and M_{c2}, are replaced with the corresponding forces in the joint reinforcement and the vertical bearing force. To obtain the joint bearing strength, the forces C_c, T_{vr}, and C_{vr} are replaced by their nominal values. The bearing strength of the joint is checked according to the following:

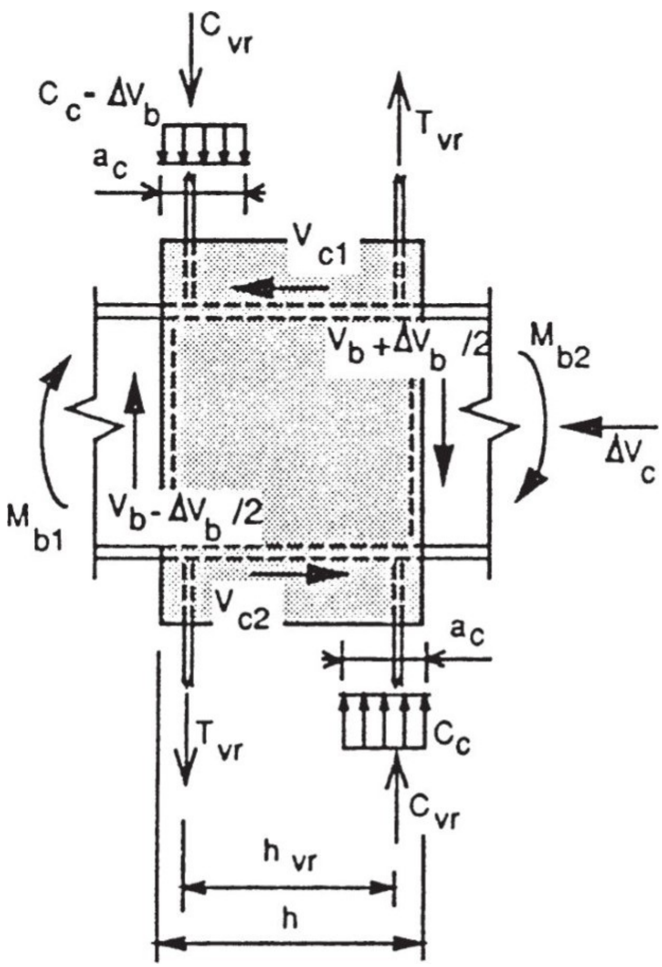


FIGURE 10.30 Vertical bearing forces (*ASCE, 1994*).

$$\sum M_c + 0.35h\Delta V_b \leq \phi[0.7hC_{cn} + h_{vr}(T_{vrn} + C_{vrn})] \quad (10.21)$$

where $\sum M_c$ = net column moments transferred through the joint
ΔV_b = net vertical beam shear transferred into the column
C_{cn} = the nominal concrete bearing strength
T_{vrn} = the nominal tension strength of the vertical joint reinforcement
C_{vrn} = the nominal compression strength of the vertical joint reinforcement
h_{vr} = the distance between the bars

C_{cn} is calculated using a bearing stress of $2f'_c$ over a bearing area with length $a_c = 0.3h$ and width b_j. The values of $2f'_c$ and $0.3h$ are based on test data. T_{vrn} and C_{vrn} are based on the connection between the reinforcement and steel beam, development of the reinforcement through bond or anchorage to concrete, and the material strength of reinforcement. To avoid overstressing the concrete within the joint, the contribution of the vertical reinforcement is limited by Eq. (10.22):

$$T_{vrn} + C_{vrn} \leq 0.3f'_c b_j h \quad (10.22)$$

To ensure adequate concrete confinement in bearing regions, three layers of ties should be provided within a distance of 0.4d above and below the beam (see Fig. 10.31). The minimum requirement for each layer is given by the following:

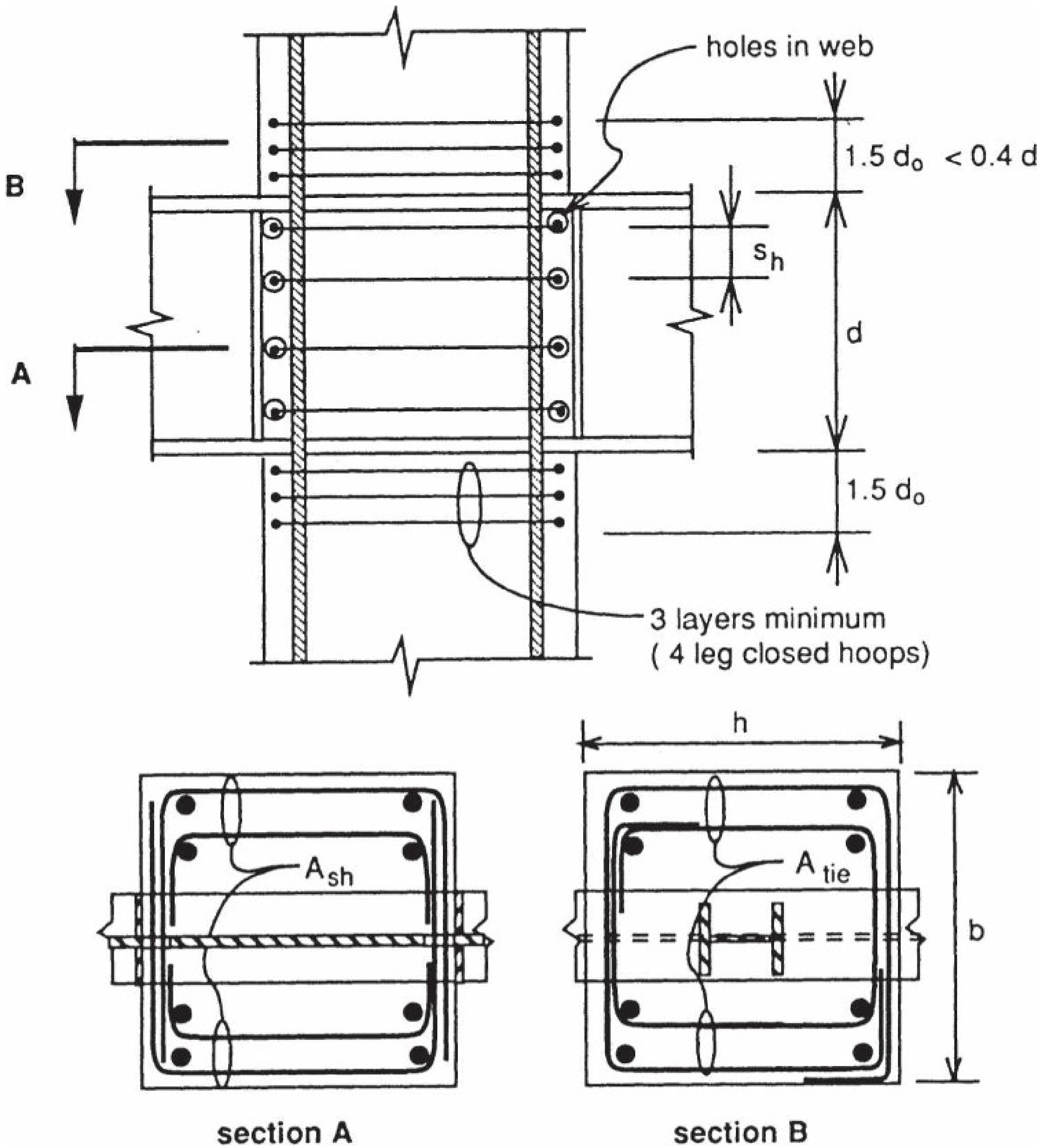


FIGURE 10.31 Column ties (*ASCE, 1994*).

$b \leq 500$ mm	four 10-mm bars
500 mm $< b \leq 750$ mm	four 12-mm bars
$b > 750$ mm	four 16-mm bars

These ties should be closed rectangular ties to resist tension parallel and perpendicular to the beam.

10.4.6.2 Joint Shear. As described in Secs. 10.4.2 and 10.4.3, shear forces in the joint are resisted by the steel web and the inner and outer concrete panels. The three different mechanisms are shown in Fig. 10.32. The horizontal shear strength is considered adequate if the following equation is satisfied:

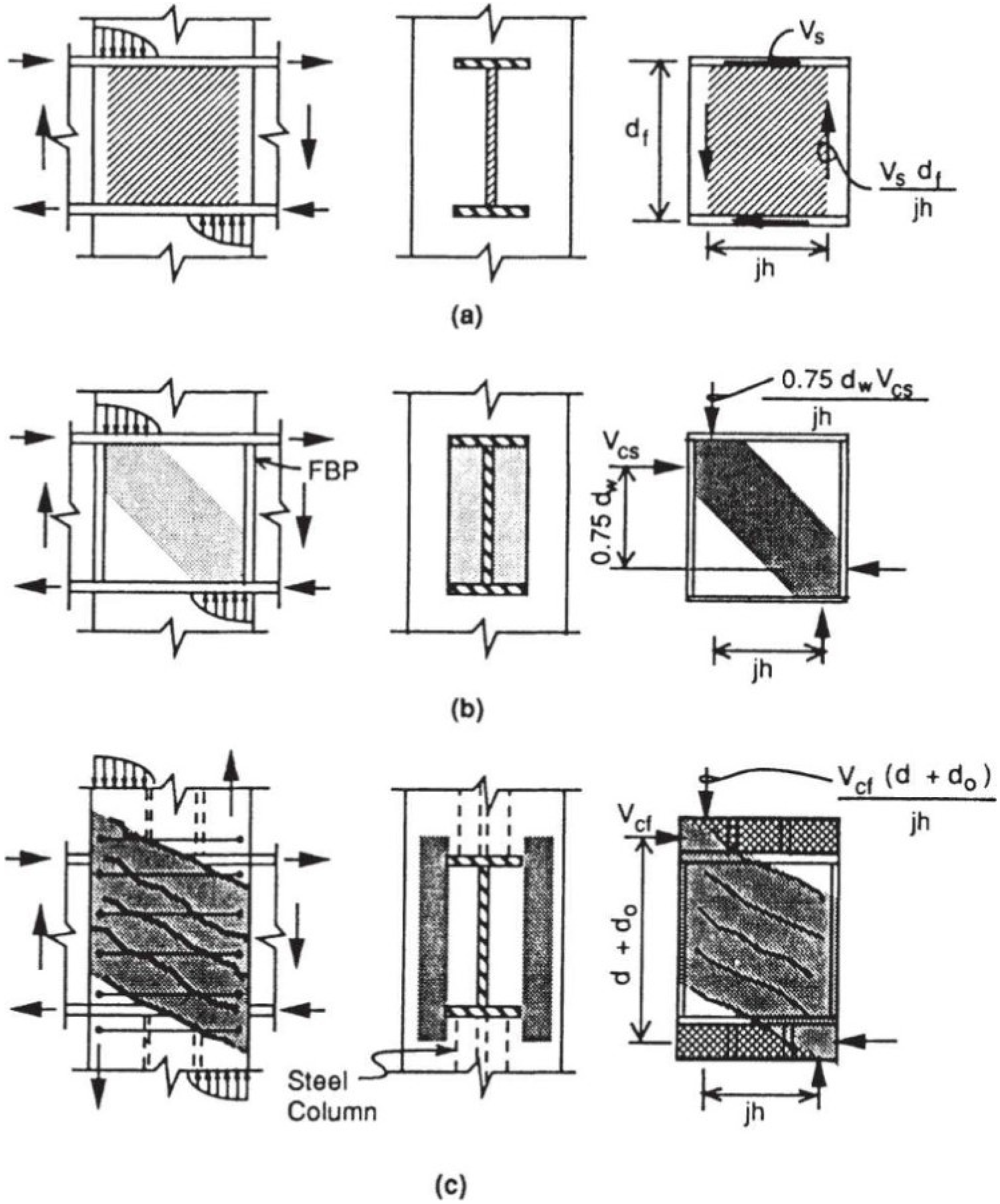


FIGURE 10.32 Joint shear mechanism (*ASCE, 1994*).

$$\sum M_c - V_b jh \leq \phi[V_{sn} d_f + 0.75 V_{csn} d_w + V_{cfn}(d + d_o)] \tag{10.23}$$

where V_{sn} = steel panel nominal strength
V_{csn} = the inner concrete compression strut nominal strength
V_{cfn} = the outer concrete compression field nominal strength
V_b = antisymmetric portion of beam shears
d_f = the center-to-center distance between the beam flanges
d_w = the depth of the steel web
d_o = additional effective joint depth provided by attachment to the beam
jh = horizontal distance between bearing force resultant and is given by the following:

$$jh = \frac{\Sigma M_c}{\phi(T_{vrn} + C_{vrn} + C_c) - \Delta V_b / 2} \geq 0.7h \tag{10.24}$$

in which

$$C_c = 2f'_c b_j a_c \tag{10.25}$$

$$a_c = \frac{h}{2} - \sqrt{\frac{h^2}{4} - K} \leq 0.3h \tag{10.26}$$

$$K = \frac{1}{\phi 2 f'_c b_j}\left[\Sigma M_c + \frac{\Delta V_b h}{2} - \phi(T_{vrn} + C_{vrn})h_{vr}\right] \tag{10.27}$$

In Eq. (10.23), it is assumed that the contributions of the mechanisms are additive. The following sections describe the individual contribution of each of the three different mechanisms.

Steel Panel. The steel contribution is given as the capacity of the beam web in pure shear. Assuming the effective panel length to be equal to jh and the average shear yield stress is $0.6F_{ysp}$, the nominal strength of the steel panel, V_{sn}, is expressed as follows:

$$V_{sn} = 0.6F_{ysp} t_{sp} jh \tag{10.28}$$

where F_{ysp} = the yield strength of the steel panel and t_{sp} = the thickness of the steel panel.

The vertical shear forces in the steel web cause the beam flanges to bend in the transverse direction. To prevent beam flanges failure, the thickness should satisfy the following:

$$t_f \geq 0.30\sqrt{\frac{b_f t_{sp} F_{ysp}}{hF_{yf}}} \tag{10.29}$$

where F_{yf} is the yield strength of the beam flanges.

Concrete Strut. The nominal strength of the concrete compression strut mechanism, V_{csn}, is calculated as follows:

$$V_{csn} = 1.7\sqrt{f'_c}\, b_p h \leq 0.5 f'_c b_p d_w \tag{10.30}$$

$$b_p \leq b_f + 5t_p \leq 1.5b_f \tag{10.31}$$

where f'_c = the concrete compressive strength, in MPa
b_p = the effective width of FBP, and is limited by Eq. (10.31)
t_p = the thickness of the FBP and should meet the following conditions:

$$t_p \geq \frac{\sqrt{3}}{b_f F_{up}}(V_{cs} - b_f t_w F_{yw}) \tag{10.32}$$

$$t_p \geq \frac{\sqrt{3}V_{cs}}{2b_f F_{up}} \tag{10.33}$$

$$t_p \geq 0.2\sqrt{\frac{V_{cs}b_p}{F_{yp}d_w}} \tag{10.34}$$

$$t_p \geq \frac{b_p}{22} \tag{10.35}$$

$$t_p \geq \frac{b_p - b_f}{5} \tag{10.36}$$

where F_{up} = the specified tensile strength of the bearing plate and V_{cs} = the horizontal shear force carried by the concrete strut.

Where split FBPs are used, the plate height, d_p, should not be less than $0.45d_w$.

Compression Field. The nominal strength of the concrete compression field mechanism, V_{cfn}, is calculated as follows:

$$V_{cfn} = V_c' + V_s' \leq 1.7\sqrt{f_c'}\,b_o h \tag{10.37}$$

$$V_c' = 0.4\sqrt{f_c'}\,b_o h \tag{10.38}$$

$$V_s' = \frac{A_{sh}F_{ysh}0.9h}{s_h} \tag{10.39}$$

where f_c' = the concrete compressive strength, in MPa
V_c' = the concrete contribution to nominal compression field, $V_c' = 0$, where the column is in tension
V_s' = the contribution provided by the horizontal ties to nominal compression field strength
A_{sh} = the cross-sectional area of reinforcing bars in each layer of ties spaced at s_h through the beam depth, $A_{sh} \geq 0.004bs_h$

Where extended FBP and/or steel columns are used, they should be designed to resist a force equal to the joint shear carried by the outer compression field, V_{cf}. The thickness of column flanges or the extended FBP is considered adequate if the following equation is satisfied:

$$t_f \geq 0.12\sqrt{\frac{V_{cf}b_p'}{d_o F_y}} \tag{10.40}$$

where V_{cf} = the horizontal shear force carried by the outer compression field
b_p' = the flange width of the steel column or the width of the extended FBP
F_y = the specified yield strength of the plate

In addition to the preceding requirement, the thickness of the extended FBP should not be less than the thickness of the FBP between the beam flanges.

Ties above and below the beam should be able to transfer the force, V_{cf}, from the beam flanges into the outer concrete panel. In addition to the requirements in Sec. 10.4.6.1, the minimum total cross-sectional area should satisfy the following:

$$A_{\text{tie}} \geq \frac{V_{cf}}{F_{ysh}} \tag{10.41}$$

where F_{ysh} = the yield strength of the reinforcement
A_{tie} = the total cross-sectional area of ties located within the vertical distance 0.4d of the beam (see Fig. 10.31)

10.4.6.3 Vertical Column Bars. To limit the slip of column bars within the joint, the size of the bar should satisfy the following requirements:

$$d_b < \frac{d + 2d_o}{20} \tag{10.42}$$

where, for single bars, d_b = the vertical bar diameter, and, for bundled bars, d_b = the diameter of a bar of equivalent area to the bundle.

Exceptions to Eq. (10.42) can be made where it can be shown that the change in force in vertical bars through the joint region, ΔF_{bar}, satisfies the following:

$$\Delta F_{\text{bar}} < 80(\text{d} + 2d_o)\sqrt{f_c'} \tag{10.43}$$

where f_c' is in MPa.

10.4.7 Limitations

The ASCE recommendations are limited to joints where the steel beams are continuous through the reinforced concrete column. Although this type of detail has been successfully used in practice, the guidelines do not intend to imply or recommend the use of this type over other possible details. Both interior and exterior joints can be designed using the recommendations; however, top-interior and top-corner joints are excluded because supporting test data are not available. For earthquake loading, the recommendations are limited to regions of low-to-moderate seismic zones. The ratio of depth of concrete column, *h*, to the depth of the steel beam, *d*, should be in the range of 0.75 to 2.0. For the purpose of strength calculation, the nominal concrete strength, f_c', is limited to 40 MPa (6 ksi) and only normal-weight concrete is allowed, the reinforcing bars yield stress is limited to 410 MPa (60 ksi), and the structural steel yield stress is limited to 345 MPa (50 ksi).

10.5 CONNECTIONS TO CONCRETE-FILLED TUBE (CFT) COLUMNS

10.5.1 Introduction

Steel tubes of relatively thin wall thickness filled with high-strength concrete have been used in building construction in the United States and far east Asian countries. This structural system allows the designer to maintain manageable column sizes while obtaining increased stiffness and ductility for wind and seismic loads. Column shapes can take the form of tubes or pipes as required by architectural restrictions. Additionally, shop fabrication of steel shapes helps ensure quality control.

In this type of construction, in general, at each floor level a steel beam is framed to these composite columns. Often, these connections are required to develop shear yield and plastic moment capacity of the beam simultaneously.

10.5.2 Current Practice

In current practices, there are very limited guidelines for selecting or designing connections for attaching steel beams to CFT columns. In these instances, heavy reliance is made on the judgment and experience of individual designers.

The majority of available information on steel beams to CFT columns has been developed as a result of the U.S.–Japan Cooperative Research Program on Composite/Hybrid Structures (1992). It should be noted that the information developed under this initiative is targeted toward highly seismic regions. Nevertheless, the information could be used to design connection details in nonseismic regions.

One of the distinct categories of connection details suggested is attaching the steel beams using full-penetration welds, as practiced in Japan. Japanese practice usually calls for a massive amount of field and shop welding. Figure 10.33 shows some of the connection details suggested in Japan. In general, the type of details that are used in Japan are not economical for U.S. practice.

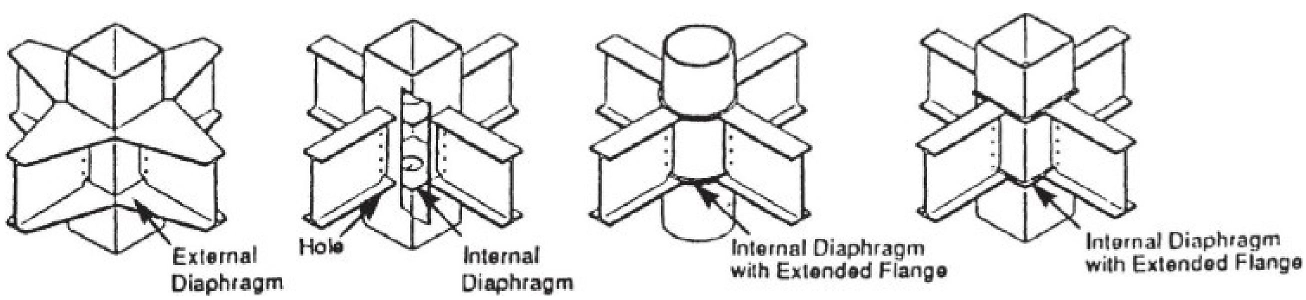


FIGURE 10.33 Typical connection details suggested in Japan.

10.5.3 Problems Associated with Welding Beams to CFT Columns

When beams are welded or attached to steel tubes through connection elements, complicated stiffener assemblies are required in the joint area within the column. However, welding of the steel beam or connecting element directly to the steel tube of composite columns could produce potential problems, some of which are outlined in the following:

1. Transfer of tensile forces to the steel tube can result in separation of the tube from the concrete core, thereby overstressing the steel tube. In addition, the deformation of the steel tube will increase connection rotation, decreasing its stiffness. This is especially

important if the connection is required to develop full plastic moment capacity of the beam.

2. Welding of the thin steel tube results in large residual stresses because of the restraint provided by other connection elements.
3. The steel tube is designed primarily to provide lateral confinement for the concrete. Further, in building construction where CFT columns are utilized, the steel tube portion of the column also acts as longitudinal reinforcement. Transferring additional forces from the beam to the steel tube could result in overstressing the steel tube portion of the column.

10.5.4 Possible Connection Detail

With these considerations in mind, Azizinamini and Parakash (1993) and Azizinamini et al. (1995) suggest two general types of connections. Figure 10.34 shows one alternative in which forces are transmitted to the core concrete via anchor bolts connecting the steel elements to the steel tube. In this alternative, all elements could be preconnected to the steel tube in the shop. The nut inside the steel tube is designed to accomplish this task. The capacity of this type of connection would be limited to the pull-out capacity of the anchor bolts and local capacity of the tube.

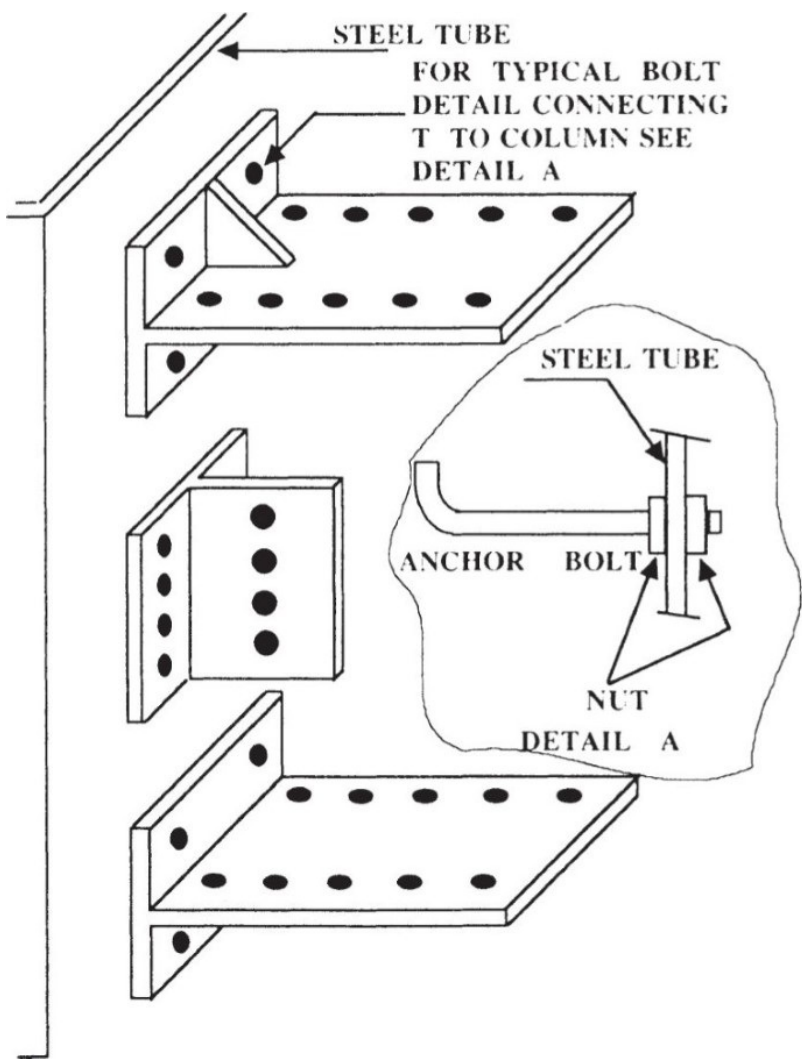


FIGURE 10.34 Connection detail using anchor bolts.

Another variation of the same idea is shown in Fig. 10.35, where connecting elements would be embedded in the core concrete via slots cut in the steel tube. In this variation, slots must be welded to connection elements after beam assembly for concrete confinement. The ultimate capacity of this detail also would be limited to the pull-out capacity of the connection elements and the concrete in the tube. The types of connections shown in Figs. 10.34 and 10.35 could be suitable to nonseismic applications, at the story levels, where the level of forces is relatively small.

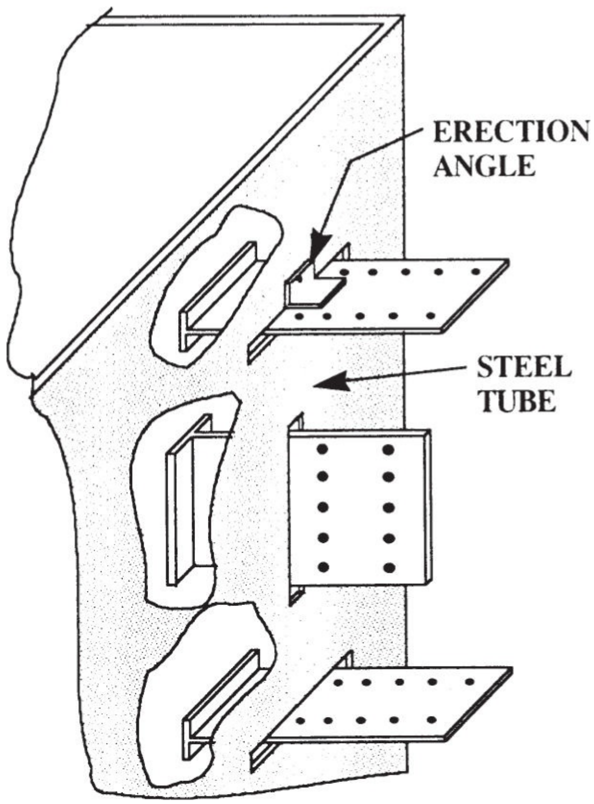


FIGURE 10.35 Connection detail using embedded elements.

Another suggested type of connection (Azizinamini and Parakash, 1993; Azizinamini et al., 1995) is to pass the beam completely through the column, as shown in Fig. 10.36. In this type of detail, a certain height of column tube, together with a short beam stub passing through the column and welded to the tube, could be shop-fabricated to form a "tree column" as shown in Fig. 10.37. The beam portion of the tree column could then be bolted to girders in the field.

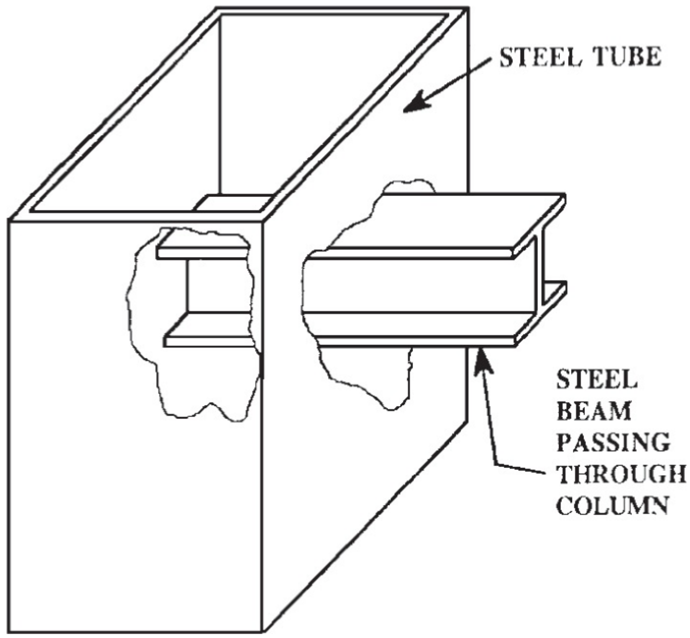


FIGURE 10.36 Through-connection detail.

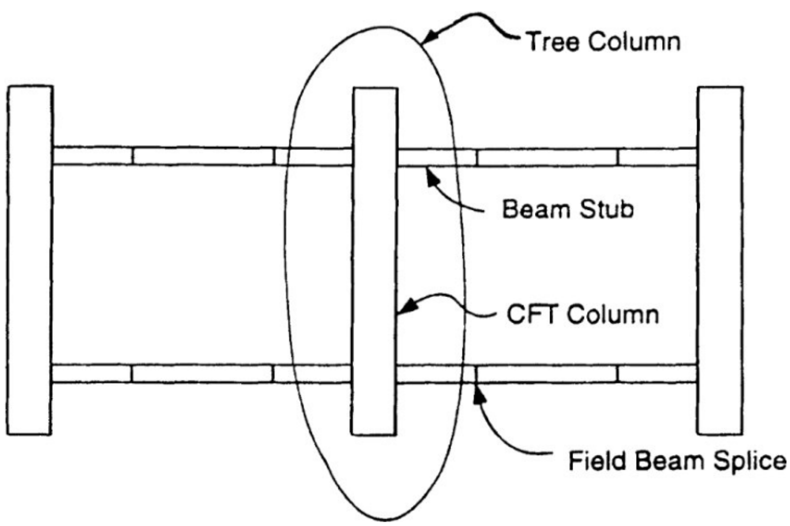


FIGURE 10.37 Tree column construction concept.

Alostaz and Schneider (1996) report tests on six different connection details for connecting steel beams to circular CFT columns. The objectives of these tests were to examine the feasibility of different connection details for use in highly seismic areas and suitable to U.S. practice.

These connections ranged from a very simple detail that attached the beam to the tube skin as in connection type I to a more rigid detail in which the girder was passed through the tube core as represented by connection type VII. All connections were designed with a beam stub. The beams were bolted and/or welded to these stubs. The specimens had a T configuration, thus representing an exterior joint in a building. Each specimen consisted of a 14 × ¼-in (356 × 6.4-mm)-diam pipe and W14 × 38 beam. The concrete compressive strength varied between 7.8 and 8.3 ksi (53.8 and 57.2 MPa). The pipe yield strength was 60 ksi (420 MPa). The stub flanges and web yield strengths were 50 and 40 ksi (350 and 280 MPa), respectively. This resulted in a column-to-beam bending capacity ratio of approximately 2.6. This relatively high column-to-beam capacity ratio is not desirable when one attempts to investigate connection behavior. At the extreme, very high column moment capacity will force the plastic hinge to form at the end of the beam, preventing the investigation of behavior of joints. Despite this shortcoming, Alostaz and Schneider's data provide valuable information that could be used to develop connection details suitable for seismic as well as nonseismic applications. Following is a brief discussion of the behavior of different connection details tested by Alostaz and Schneider (1996).

10.5.4.1 Simple Connection, Type I. Figure 10.38 illustrates the details of this specimen. The flange and web plates of the connection stub were welded directly to the steel pipe. At the tube face, the flange plates were flared to form a central angle of 120°, and the width of the plates was decreased gradually over a 10-in (254-mm) distance to match that of the girder flanges. Figure 10.39 shows the load-displacement relationship. Failure was due to fracture at the flange tip on the connection stub and pipe wall tearing. The connection survived a limited number of inelastic cycles and it could not develop the plastic flexural strength of the girder. This connection had the lowest flexural strength and was the most flexible of all connections tested. This connection had a ductility ratio of 1.88, which was the lowest of all connections tested. The flexural ductility ratio (FDR) was defined as

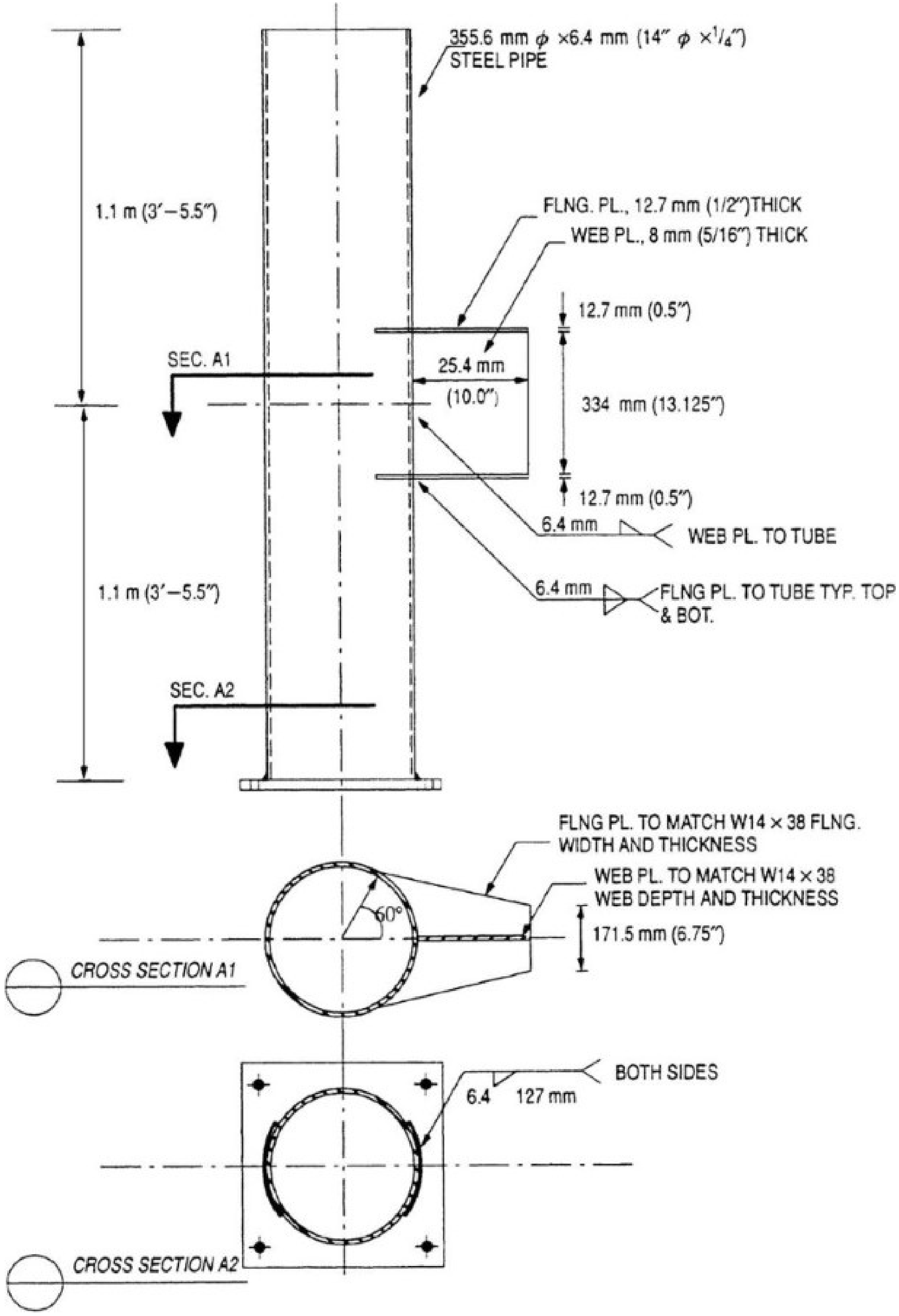


FIGURE 10.38 Simple connection, type I (*Alostaz and Schneider, 1996*).

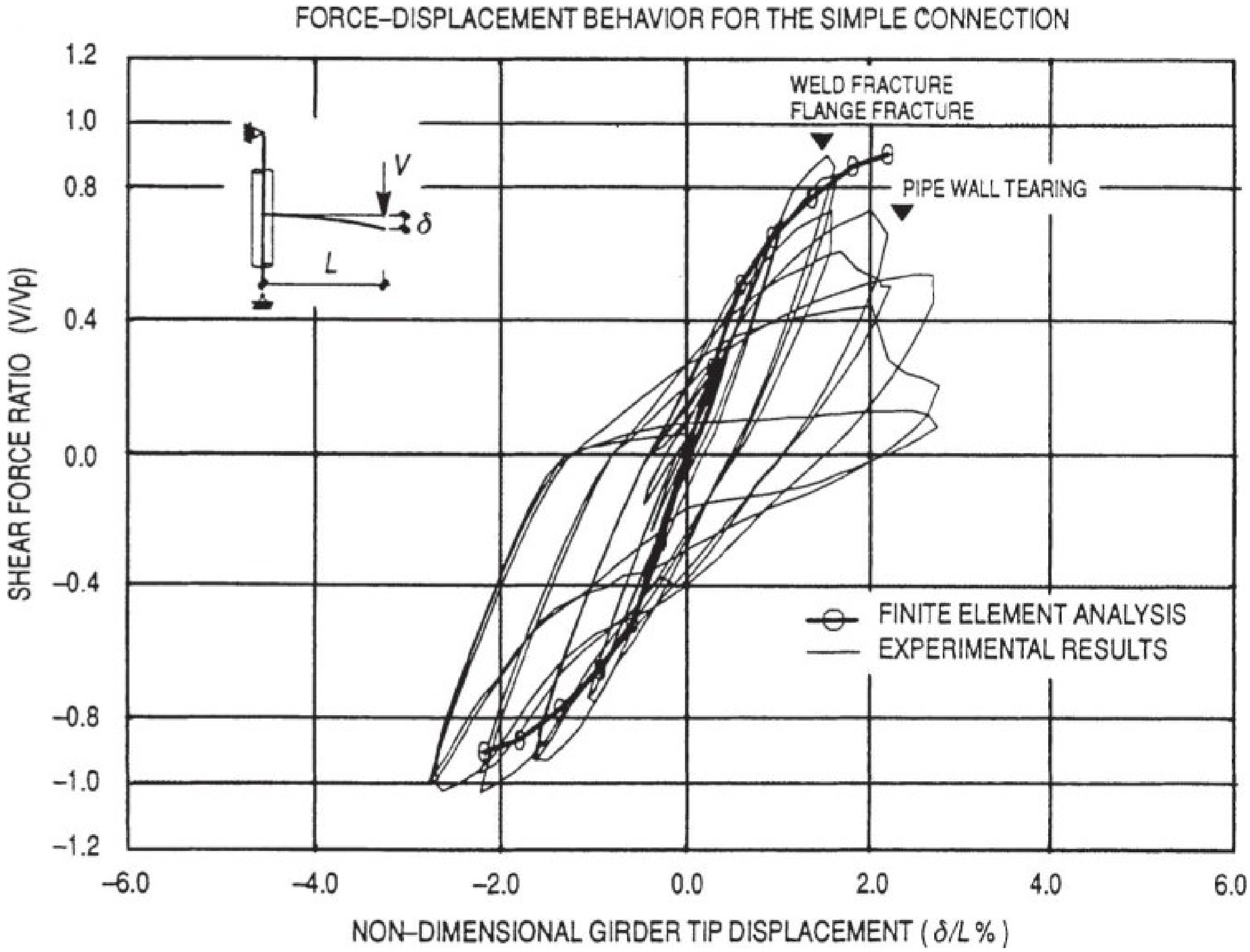


FIGURE 10.39 Load-displacement behavior of connection type I (*Alostaz and Schneider, 1996*).

$$\mathrm{FDR} = \frac{\delta_{\mathrm{max}}}{\delta_{\mathrm{yield}}} \tag{10.44}$$

where δ_{max} is the maximum displacement at the girder tip prior to failure and δ_{yield} is the yield displacement obtained experimentally.

10.5.4.2 Continuous Web Plate Connection, Type IA. In an attempt to improve the behavior of connection type I, the web plate was extended through the concrete core. To continue the web through the tube, a vertical slot was cut on opposite sides of the tube wall. The web plate was fillet-welded to the tube. Figure 10.40 illustrates the details of this specimen. Figure 10.41 shows the load-displacement relationship. The hysteretic behavior of this modified connection exhibited significant improvement compared to the original simple connection. This connection was able to develop approximately 1.26 times the flexural plastic strength of the girder and the initial stiffness was comparable to the ideal rigid connection. However, the strength deteriorated rapidly and only 50% of the girder bending strength remained at the end of the test. This connection had a ductility ratio of 2.55.

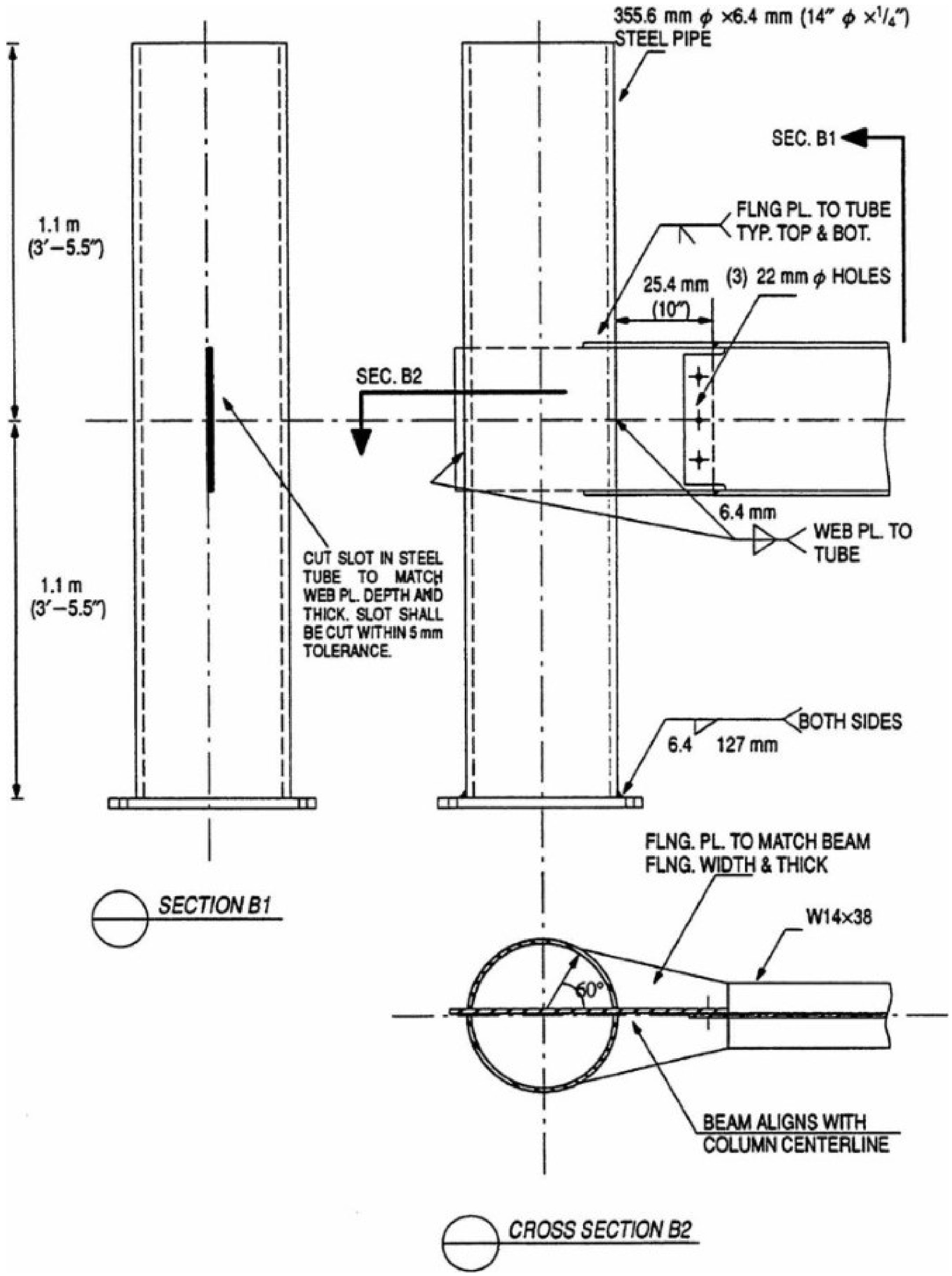


FIGURE 10.40 Simple connection with continuous web plate, type IA (*Alostaz and Schneider, 1996*).

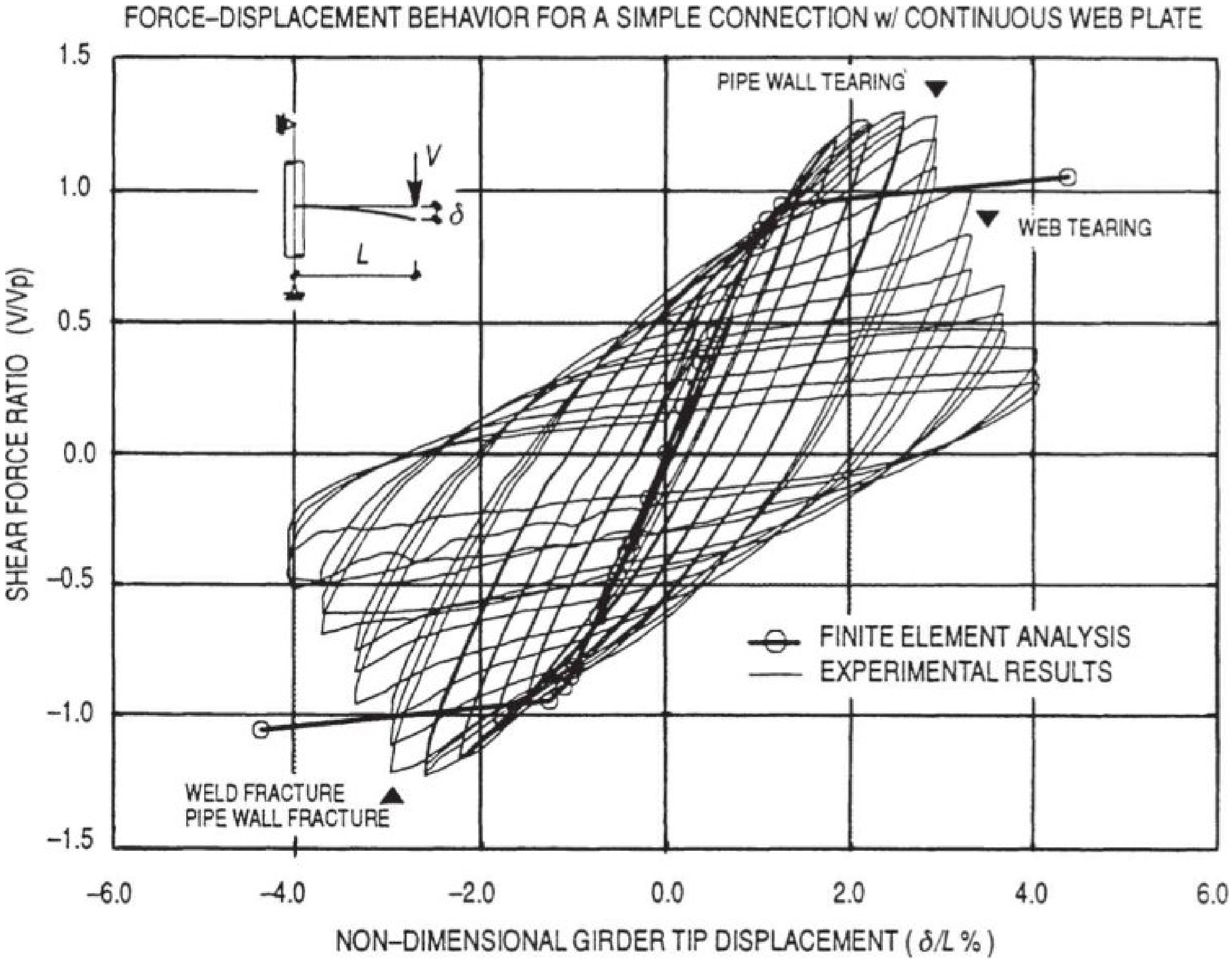


FIGURE 10.41 Load-displacement behavior of connection type IA (*Alostaz and Schneider, 1996*).

10.5.4.3 Connection with External Diaphragms, Type II. Behavior of the simple connection was improved by expanding the connection stub flanges to form external diaphragms. The diaphragm was fillet-welded to the pipe wall on both sides of the plate. Figure 10.42 illustrates the details of this specimen. Figure 10.43 shows the load-displacement relationship. The hysteretic performance of this connection improved relative to the simple connection type I. This resulted in a connection strength of approximately 17% higher than the girder bending strength. The geometry of the diaphragm was a critical issue in the behavior of this detail. The sharp reentrant corner between the diaphragm and the girder created a large stress concentration which initiated fracture in the diaphragm. This fracture caused rapid deterioration in the connection performance. Significant tearing was noted through the welded region of the diaphragm plates. Although connection type IA had higher strength, its strength deteriorated at a faster rate compared to the connection with external diaphragms. This connection had a ductility ratio of 2.88. Analytically, this detail exhibited significant improvement when the girder was shifted further away from the CFT column face.

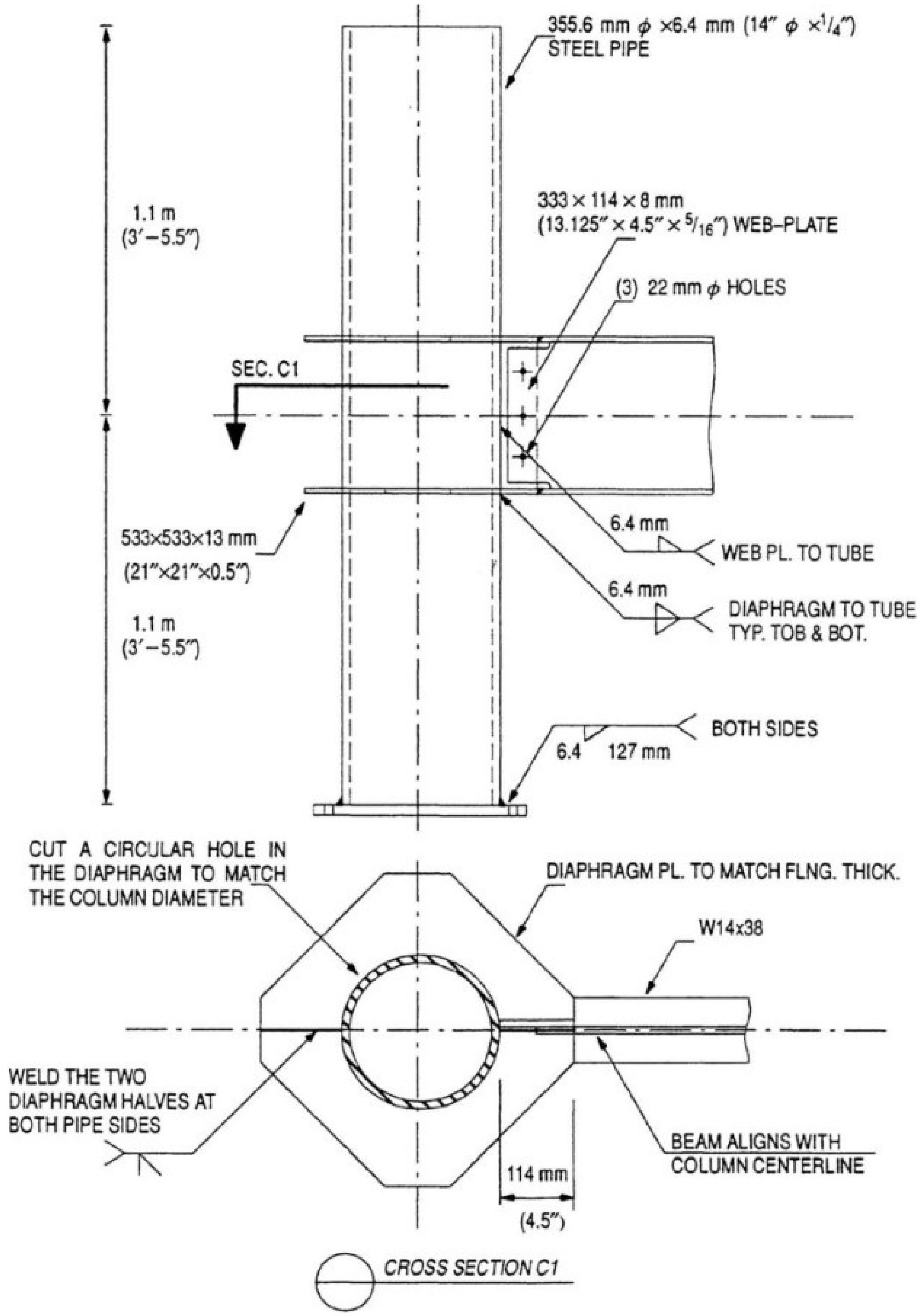


FIGURE 10.42 Connection with external diaphragms, type II (*Alostaz and Schneider, 1996*).

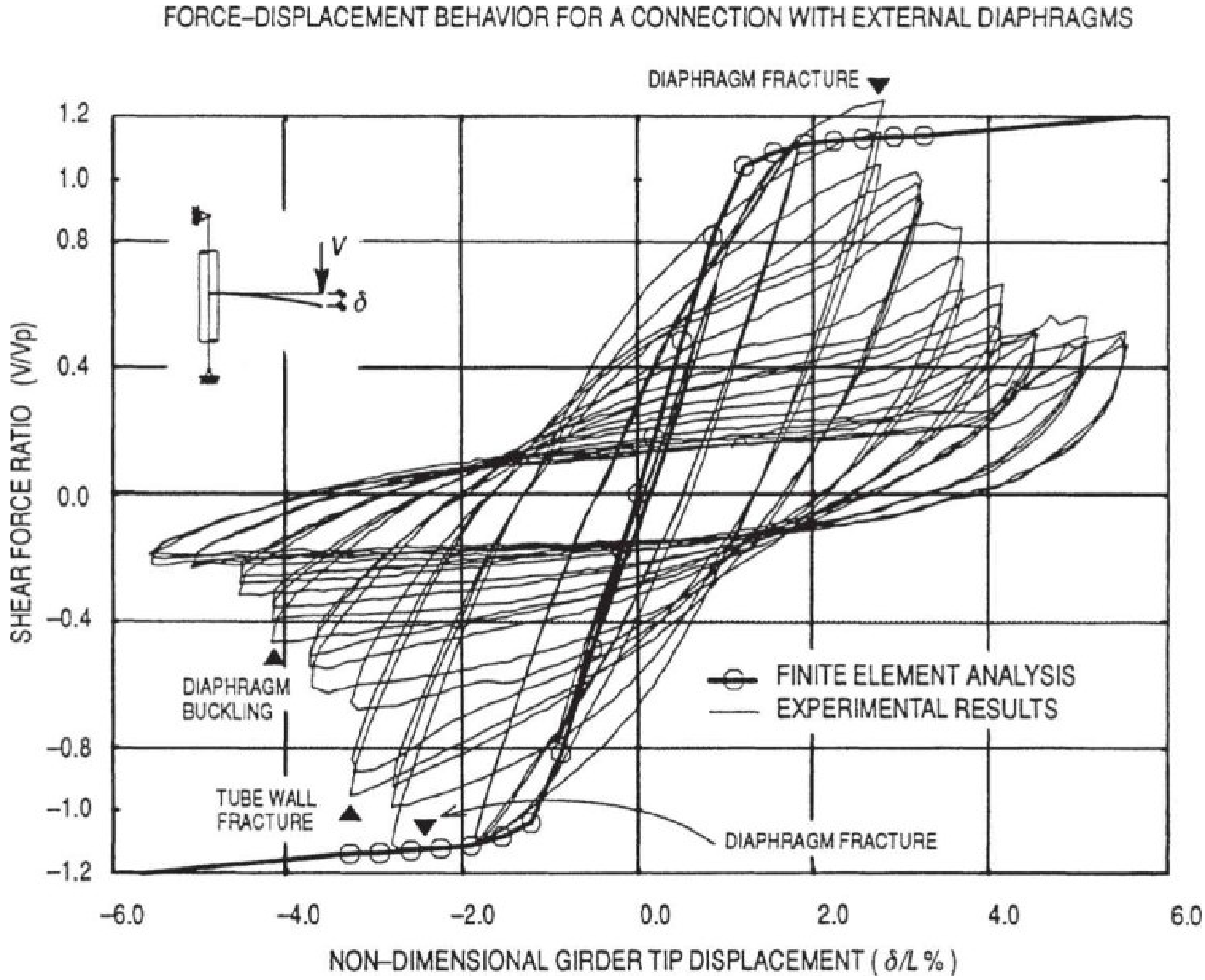


FIGURE 10.43 Load-displacement behavior of connection type II (*Alostaz and Schneider, 1996*).

10.5.4.4 Connection with Deformed Bars, Type III. This specimen is identical to connection type I, except that holes were drilled in the pipe to insert weldable deformed bars into the core of the tube. Four #6 (19-mm) deformed bars were welded to each flange. Figure 10.44 illustrates the details of this specimen. Figure 10.45 shows the force-displacement relationship. This connection exhibited stable strain-hardening behavior up to failure, and it developed approximately 1.5 times the girder bending strength. Failure was sudden and occurred by rupture of three of the four deformed bars in the connection detail, while the fourth bar failed by pull-out of the concrete core. The connection ductility was approximately 3.46 compared to only 1.88 for an identical connection without the deformed bars. The clearance, weldability of the deformed bars, and the configuration of the weld on the bars are critical issues in this detail.

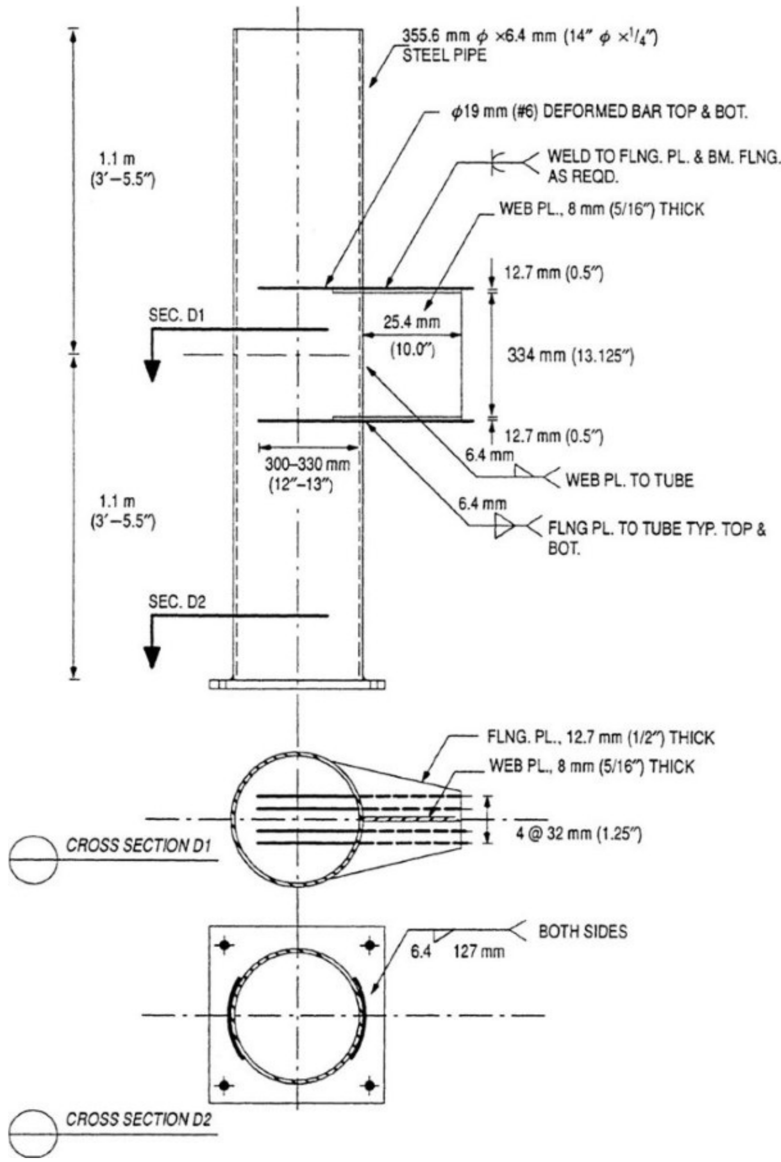


FIGURE 10.44 Connection with embedded deformed bars, type III (*Alostaz and Schneider, 1996*).

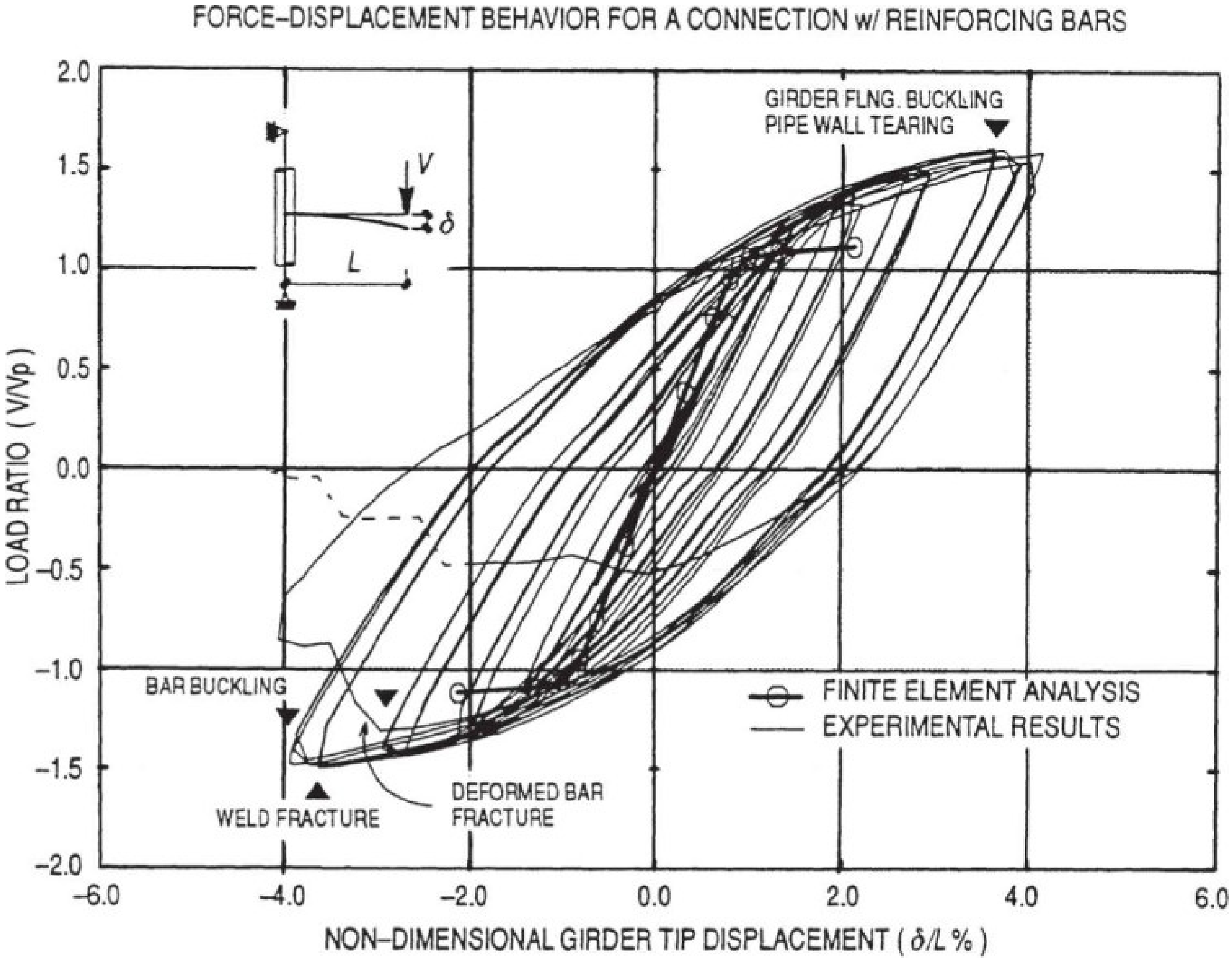


FIGURE 10.45 Load-displacement behavior of connection type III (*Alostaz and Schneider, 1996*).

10.5.4.5 Continuous Flanges, Type VI. To resolve the problems of connection type III, the connection stub flanges were continued through the pipe and fillet-welded to the pipe wall. A shear tab was fillet-welded to the tube skin. No effort was made to enhance the bond between the embedded flanges and the concrete core. Figure 10.46 illustrates the details of this specimen. Figure 10.47 shows the force-displacement relationship. The fillet weld attaching the flanges to the tube wall fractured at low amplitude cyclic deformations. The embedded flanges slipped through the concrete core without significant resistance. The hysteretic curves were quite pinched and it is likely that this connection may not perform well during a severe seismic event.

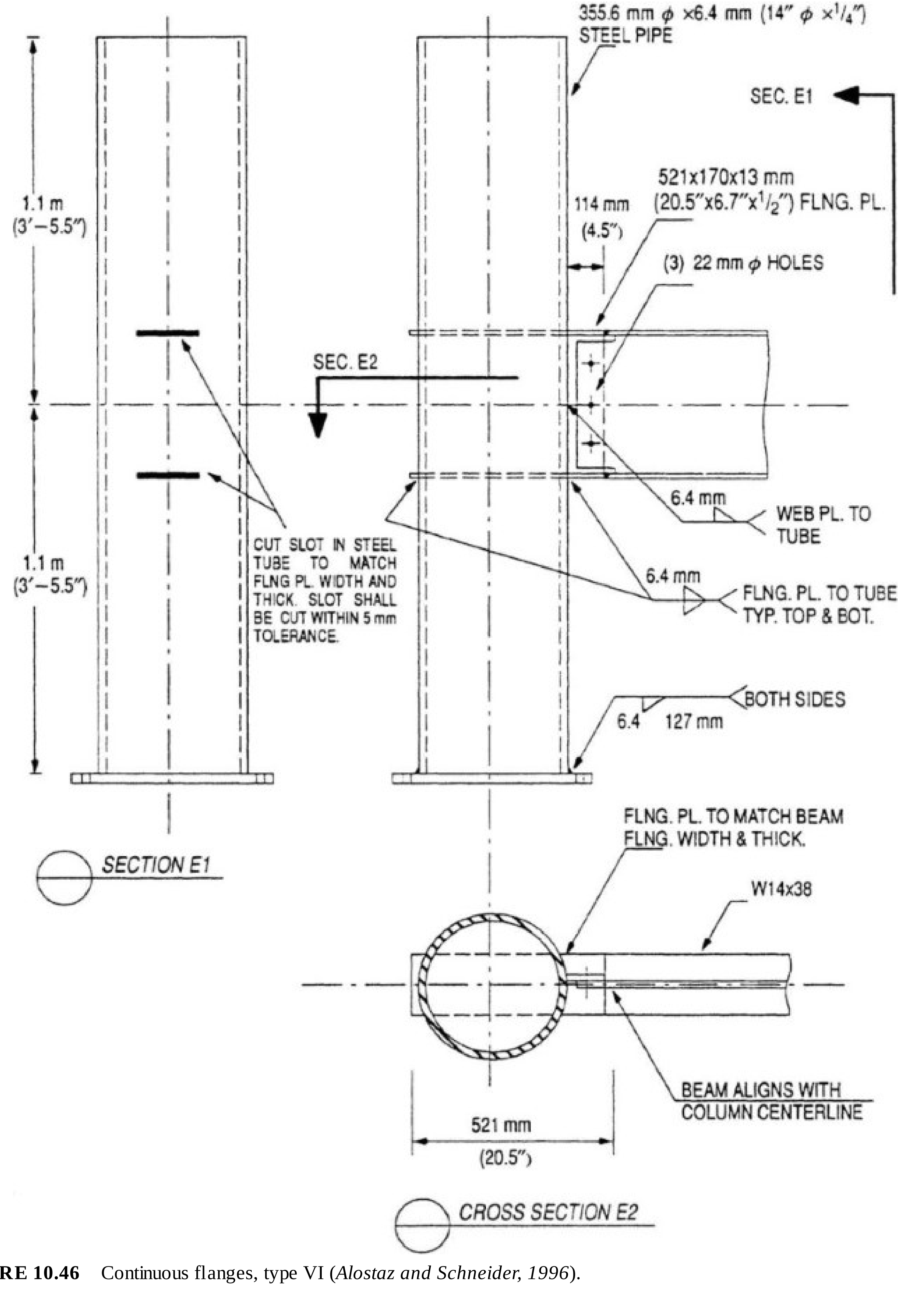


FIGURE 10.46 Continuous flanges, type VI (*Alostaz and Schneider, 1996*).

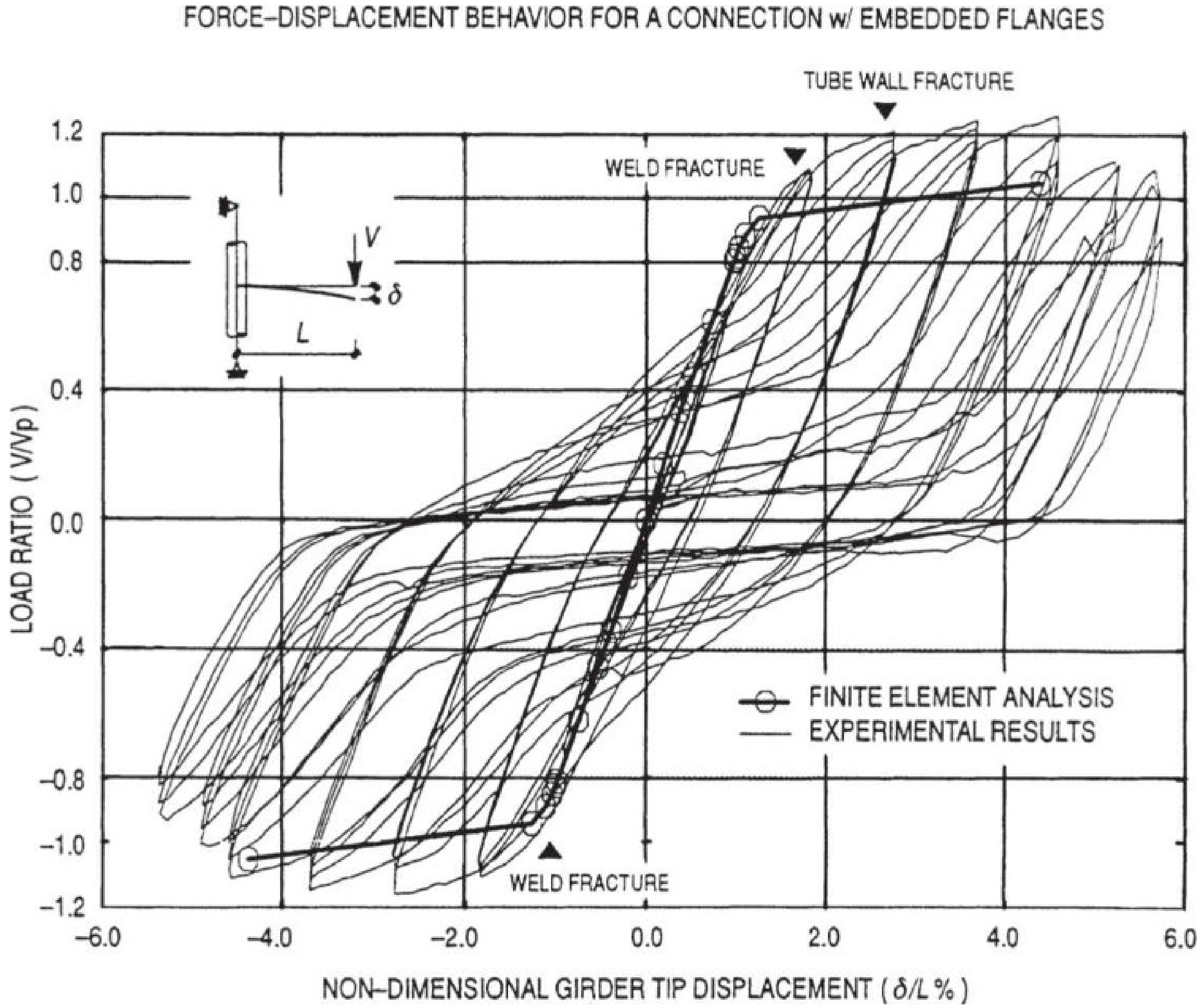


FIGURE 10.47 Load-displacement behavior of connection type VI (*Alostaz and Schneider, 1996*).

10.5.4.6 Through-Beam Connection Detail, Type VII. Alostaz and Schneider (1996) also tested one specimen with the through-beam connection detail suggested by Azizinamini and Parakash (1993) and Azizinamini et al. (1995). In this detail, the full cross section of the girder was continued through the tube core. An I-shaped slot was cut in the tube wall and the beam stub was passed through the pipe. The beam stub was fillet-welded to the pipe. Figure 10.48 illustrates the details of this specimen. Figure 10.49 shows the force-displacement relationship. The flexural strength of this connection exceeded 1.3 times the plastic bending strength of the girder. This detail had a ductility ratio of 4.37, the highest of all connections tested. It also had a satisfactory hysteretic performance. Table 10.1 shows a summary of the flexural characteristics of the tested connections.

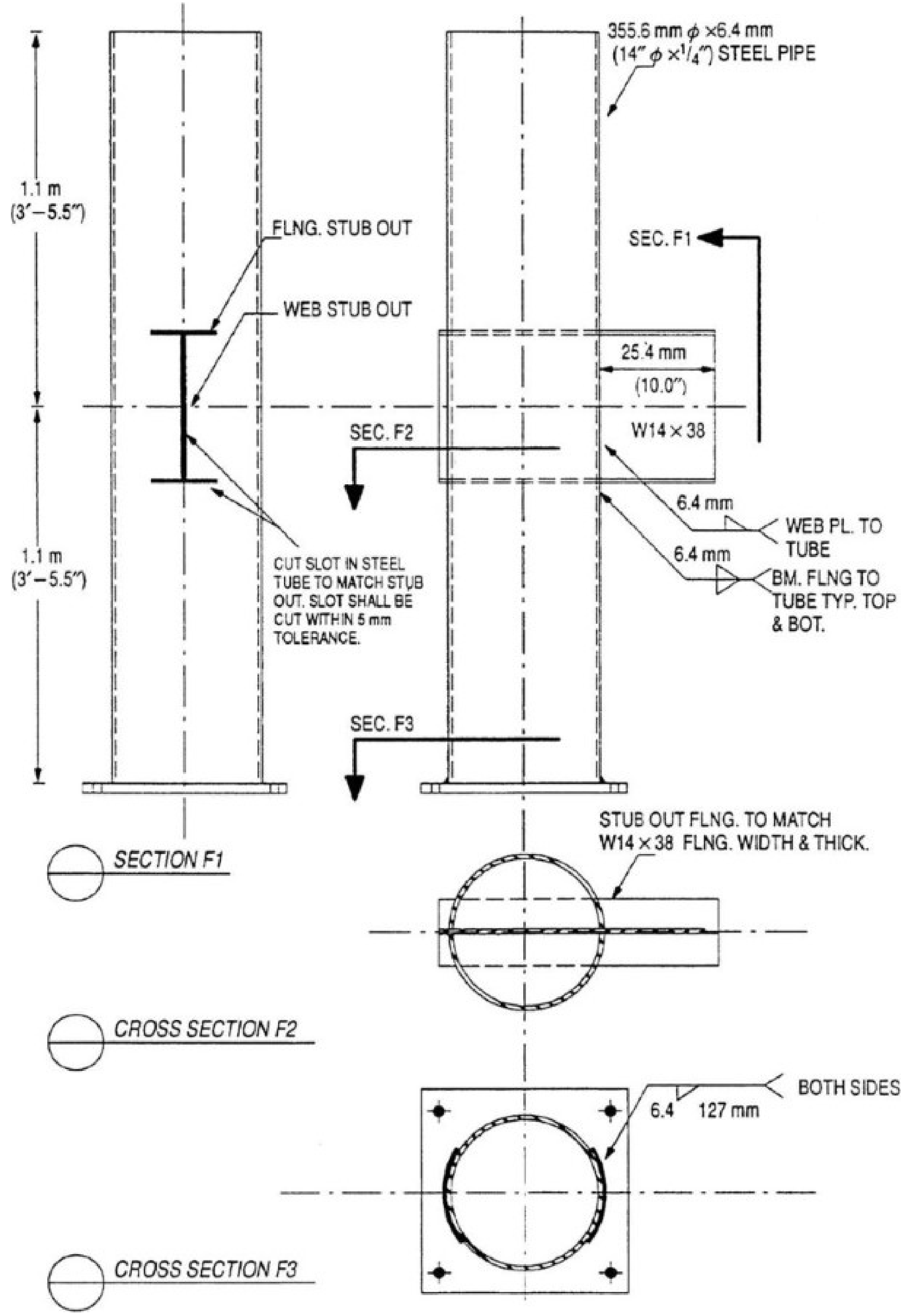


FIGURE 10.48 Continuation of the girder through the column, type VII (*Alostaz and Schneider, 1996*).

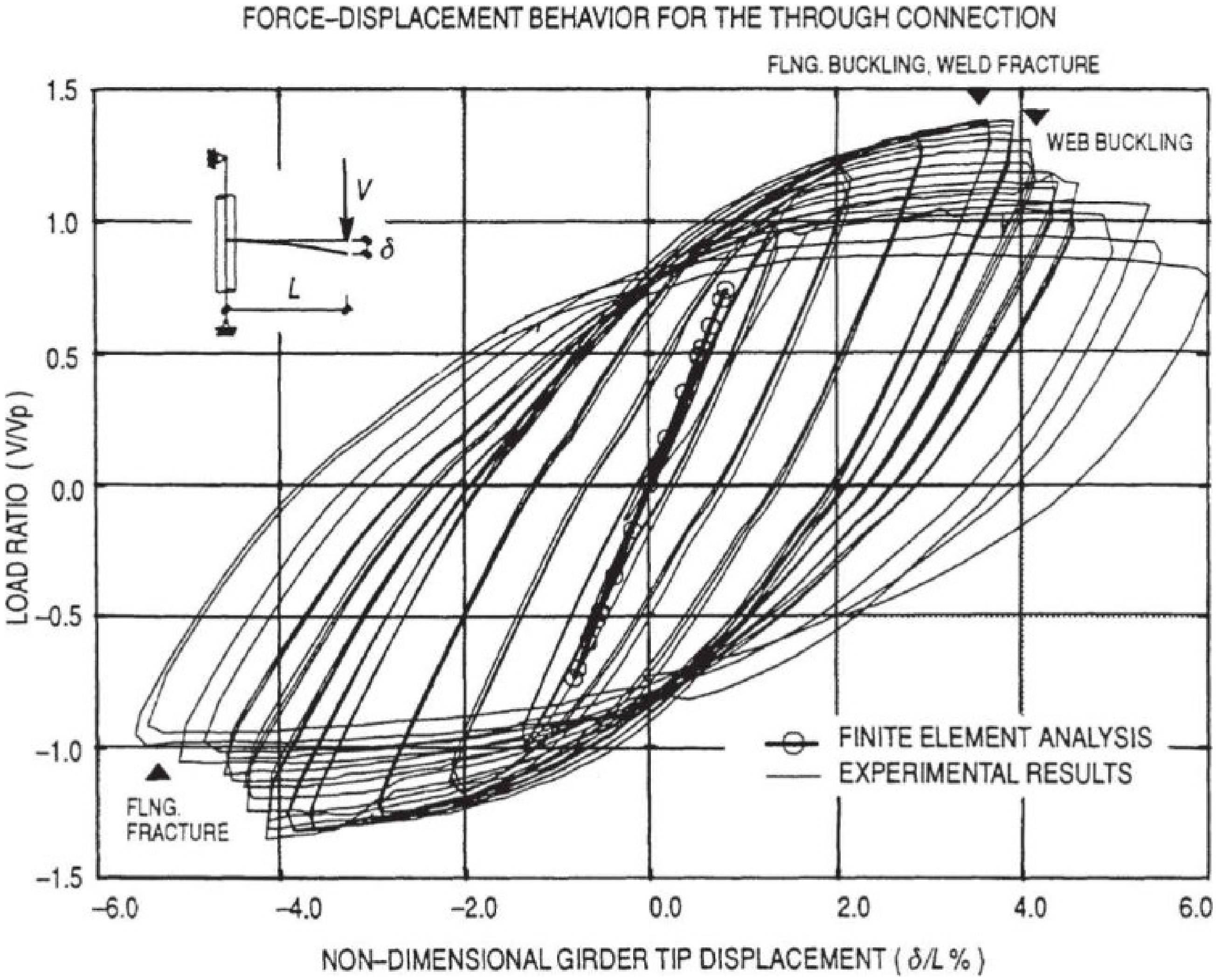


FIGURE 10.49 Load-displacement behavior of connection type VII (*Alostaz and Schneider, 1996*).

TABLE 10.1 Flexural Characteristics of the Tested Connections

Detail	Ductility	M_{max}/M_p	Initial Stiffness Ratio
I	1.88	0.97	85
IA	2.55	1.26	100
II	2.83	1.17	100
III	3.46	1.56	106
VI	3.76	1.23	100
VII	4.37	1.37	100

Results of Alostaz and Schneider tests indicated that the through-beam connection detail had the best performance, especially for seismic regions.

10.5.4.7 Other Connection Details. Ricles et al. (1997) report results of cyclic tests conducted on beams attached to rectangular CFT columns using bolted or welded tees. Figures 10.50 through 10.52 show connection details for three of the specimens tested (specimens C4, C5, and C6). The split tees in these specimens were posttensioned to the column using 14-A490 bolts after curing of the concrete. These bolts were passed through the column using PVC conduits placed prior to casting concrete. In specimens C4 and C5, 22-mm-diam A325 bolts with 2-mm oversized bolt holes were used to attach the beam flanges to split tees. In specimen C6, however, 12-mm fillet welds were used to attach the beam flanges to split tees. In specimens C5 and C6, the shear tabs for attaching the beam web to CFT column were omitted.

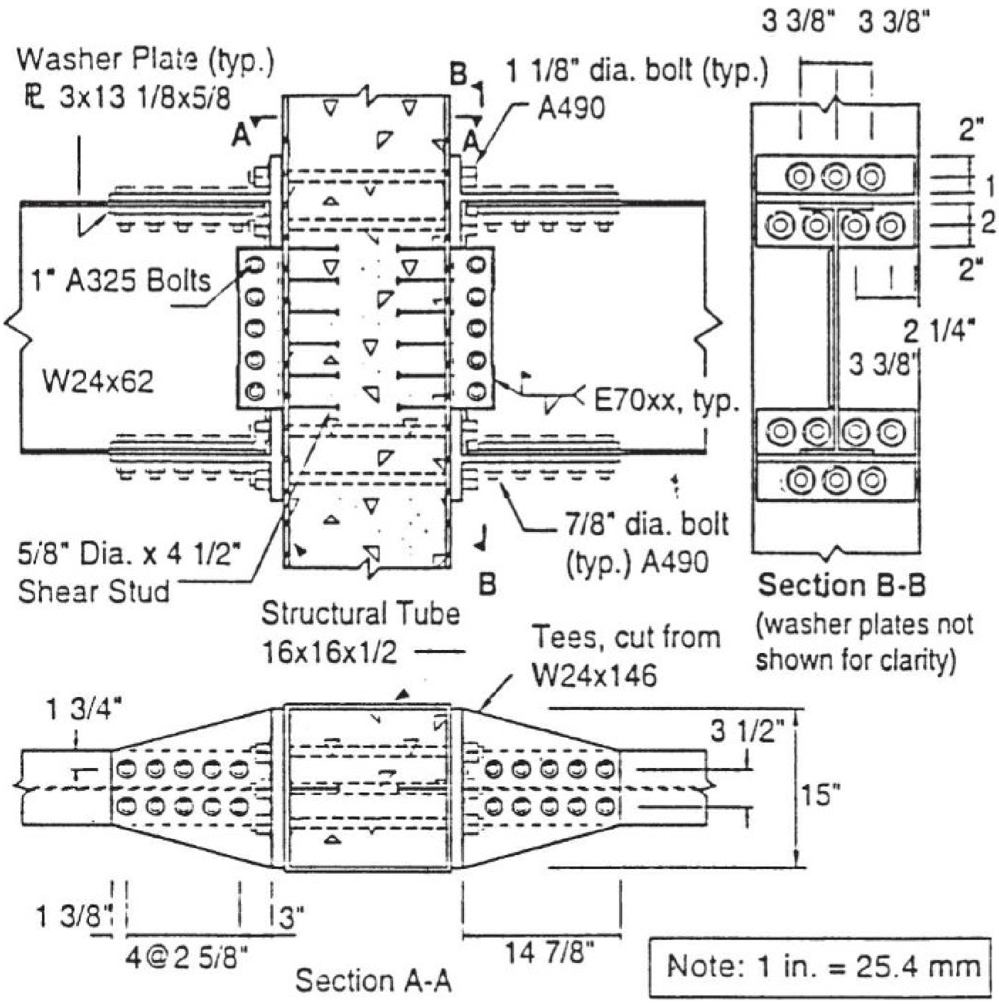


FIGURE 10.50 Split tee connection detail, specimen C4 (*Ricles et al., 1997*).

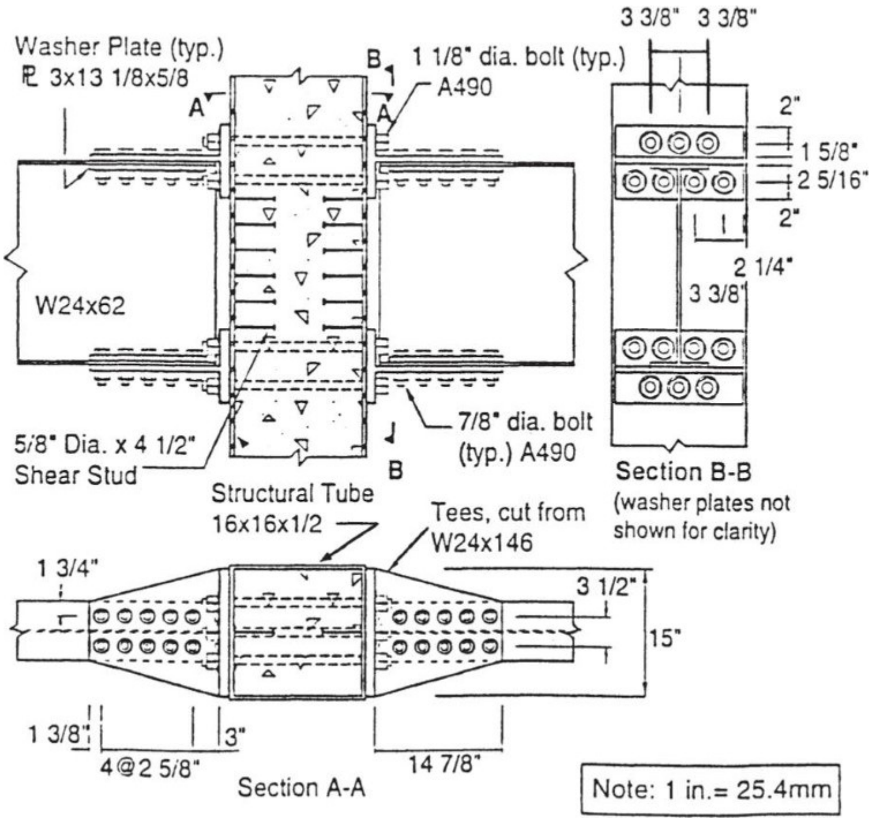


FIGURE 10.51 Split tee connection detail, specimen C5 (*Ricles et al., 1997*).

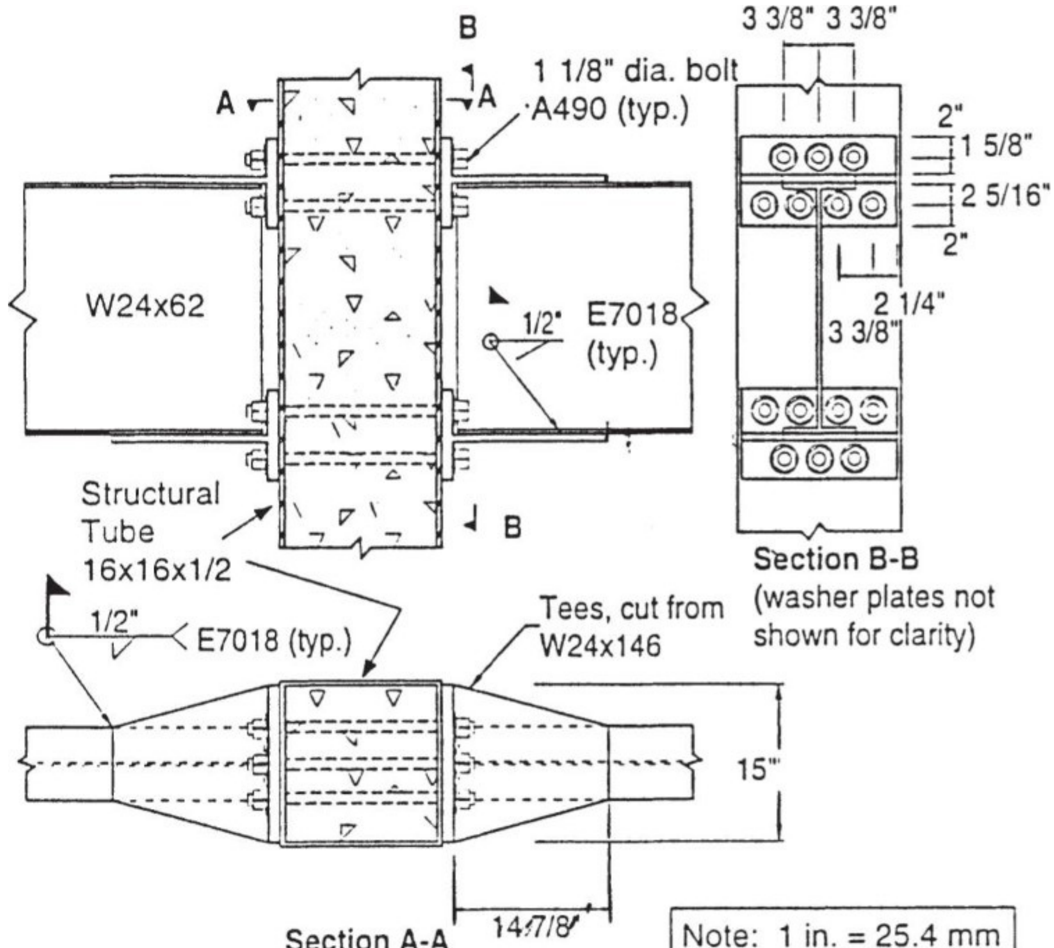


FIGURE 10.52 Split tee connection detail, specimen C6 (*Ricles et al., 1997*).

Figures 10.53 through 10.55 give plots of applied beam moment versus the resulting plastic rotation at the connection level for the three test specimens. These specimens were designed based on AISC LRFD seismic provisions following the weak beam-strong column configuration.

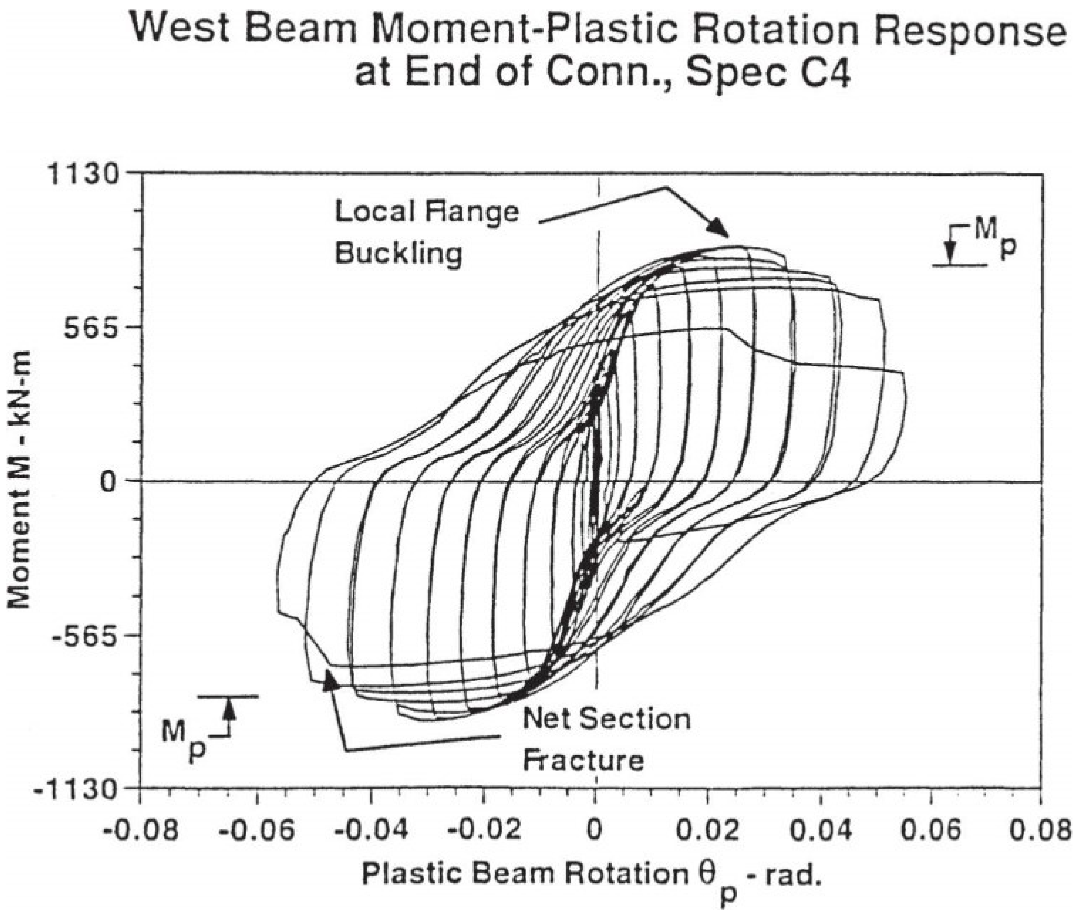


FIGURE 10.53 Moment-plastic rotation response, specimen C4 (*Ricles et al., 1997*).

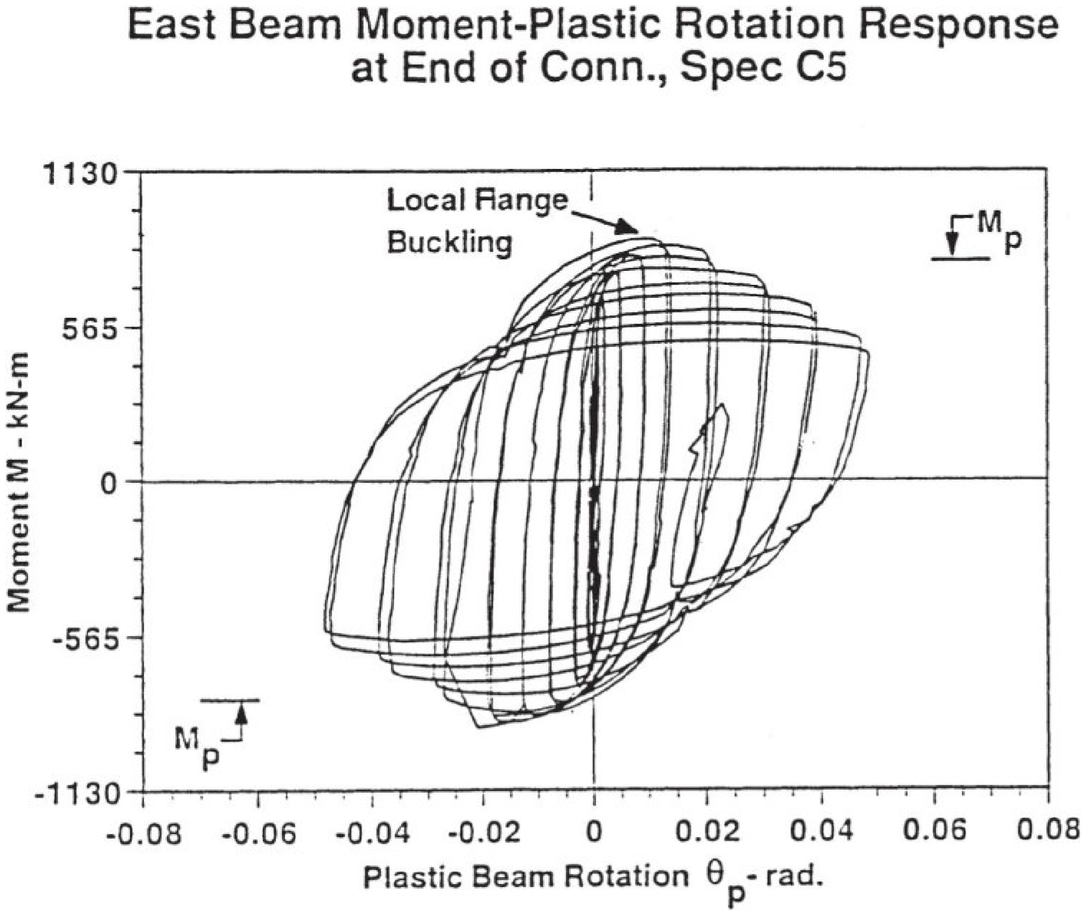


FIGURE 10.54 Moment-plastic rotation response, specimen C5 (*Ricles et al., 1997*).

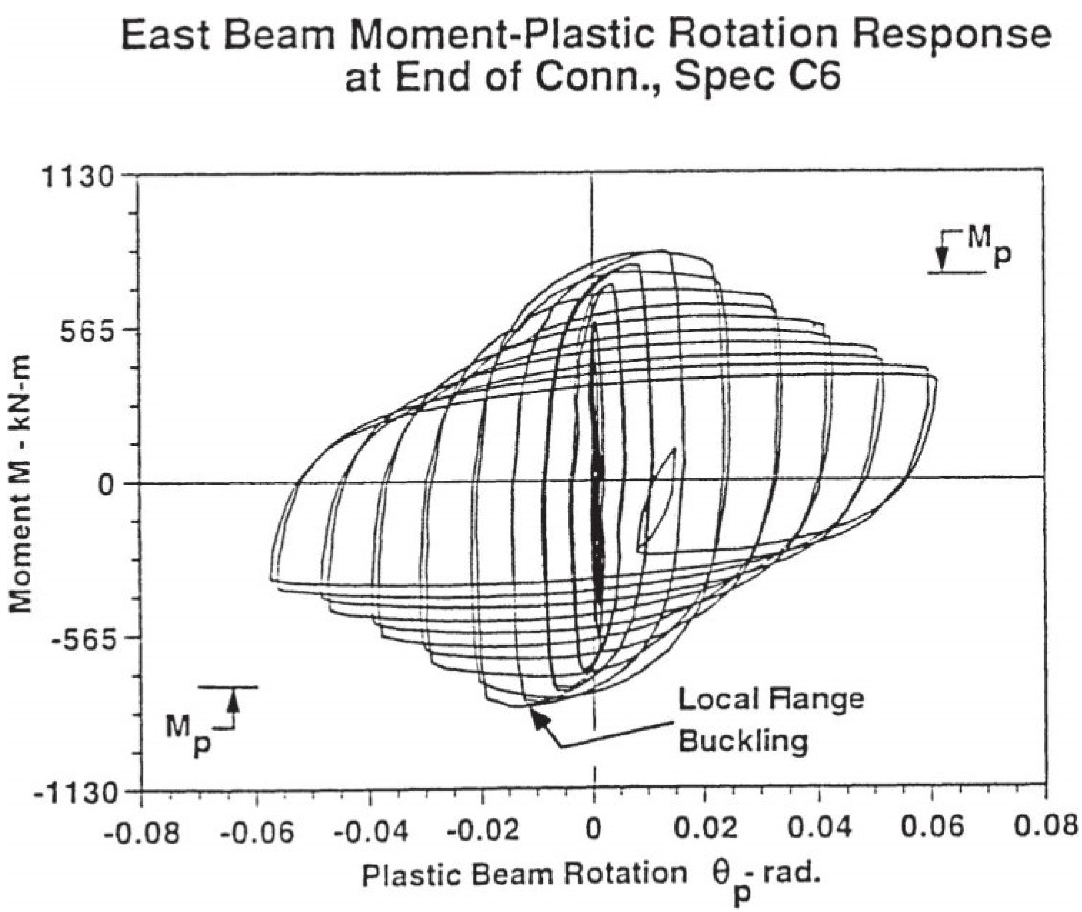


FIGURE 10.55 Moment-plastic rotation response, specimen C6 (*Ricles et al., 1997*).

Test observations indicated that damage to the joint area was eliminated. Some elongation of A490 bolts was observed. This was attributed to compressive bearing forces transferred from split tees to CFT columns, causing distortion of the joint area in CFT columns. Another major observation was the slippage of the stem of split tees with respect to beam flanges in specimens C4 and C5. Ricles et al. (1997) were able to eliminate this slippage by welding washers to the beam flanges. The washers, acting as reinforcing material around the bolt hole, prevented bolt hole elongation and elimination of the slippage.

In this type of detail, attention should be directed to shear transfer between the beam end and CFT column. The load path for transferring the beam shear force to the CFT column is as follows. The beam end shear is first transferred as axial force from the beam end to the steel tube portion of the CFT column. This axial compressive or tensile force could only be transferred to the concrete portion of the CFT column if composite action between the steel tube and the concrete core exists. There are several ways through which this composite action could be developed. Friction due to bending or use of shear studs are two possible mechanisms. The guidelines for such shear-transfer mechanisms are still lacking. Ongoing research by Roeder (1997) attempts to resolve this issue.

10.5.5 Force Transfer Mechanism for Through-Beam Connection Detail

A combination of analytical and experimental investigations were undertaken to approximate the force transfer mechanism for the through-beam connection detail utilizing both circular and rectangular CFT columns (Azizinamini and Parakash, 1993; Azizinamini, et al., 1995).

Figure 10.56 shows the force transfer mechanism. A portion of the steel tube between the beam flanges acts as a stiffener, resulting in a concrete compression strut which assists the beam web within the joint in carrying shear. The effectiveness of the compression strut increases to a limit by increasing the thickness of the steel plate.

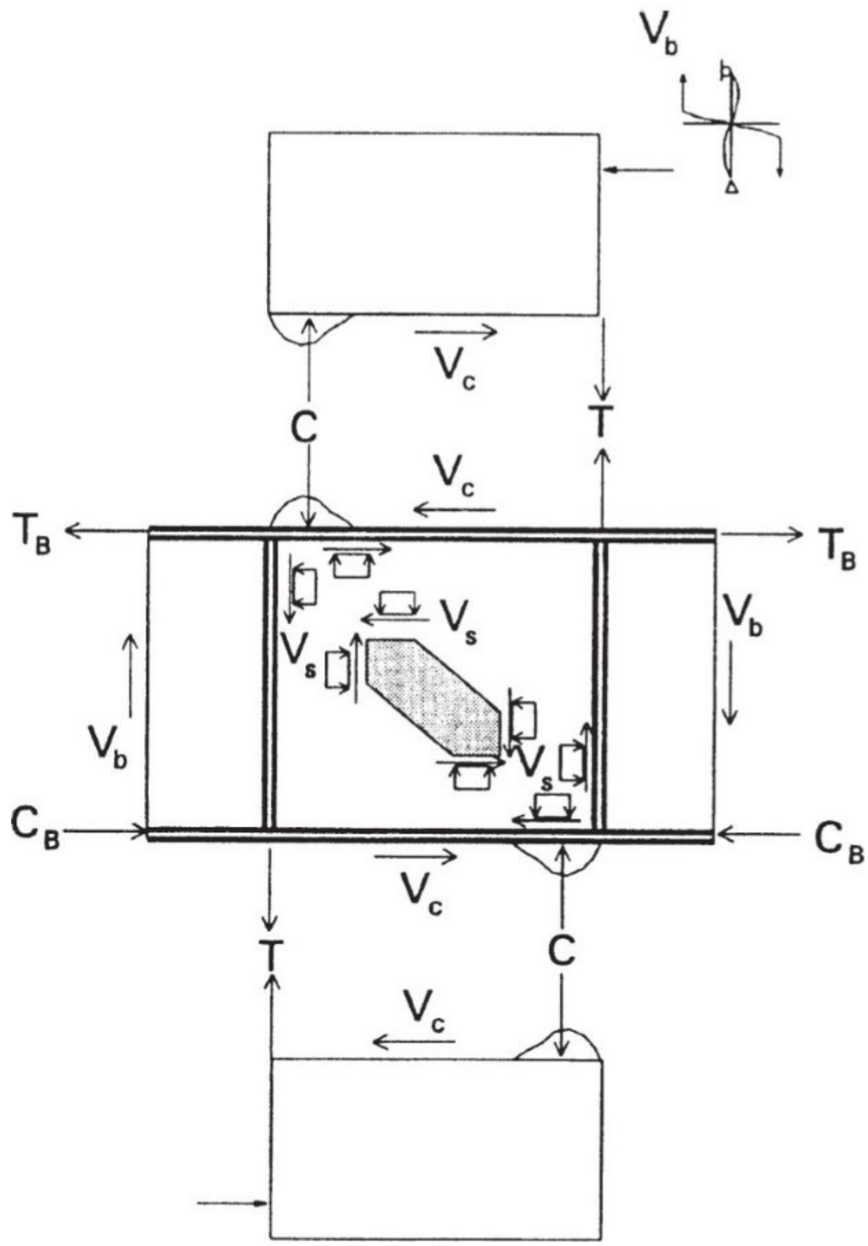


FIGURE 10.56 Force transfer mechanism for through-beam connection detail.

The width of the concrete compression strut on each side of the beam web in the direction normal to the beam web was approximately equal to half the beam flange width.

A compressive force block was created when beam flanges were compressed against the upper and lower columns (Fig. 10.56). The width of this compression block was approximately equal to the width of the beam flange. In the upper and lower columns, shown in Fig. 10.56, the compressive force *C* is shown to be balanced by the tensile force in the steel pipe. In Azizinamini and Parakash (1993), rods embedded in the concrete and welded to the beam flanges were provided to assist the steel tube in resisting the tensile forces and to minimize the tensile stresses in the steel tube. For small columns this may be necessary, however, for relatively larger columns there may not be a need for placing such rods. Ongoing research at the University of Nebraska—Lincoln is investigating this and other aspects of the force transfer mechanism. The next section suggests design provisions for the through-beam connection detail. These provisions are tentative and are applicable for both circular and rectangular CFT columns.

10.5.6 Tentative Design Provisions for Through-Beam Connection Detail

This tentative design procedure is in the form of equations relating the applied external forces to the connection details such as the thickness of the steel pipe. The design procedure follows the general guidelines in the AISC LRFD manual. In developing the design equations the following assumptions were made:

1. Externally applied shear forces and moments at the joints are known.
2. At the ultimate condition, the concrete stress distribution is linear and the maximum concrete compressive stress is below its limiting value.

The joint forces implied in assumption (1) could be obtained from analysis and require the knowledge of the applied shear and moment at the joint at failure. These quantities are assumed to be related as follows:

$$V_c = \alpha V_b$$
$$M_b = l_1 V_b$$
$$M_c = l_2 V_c$$

where V_b and M_b are the ultimate beam shear and moment, respectively, and V_c and M_c are the ultimate column shear and moment, respectively. Figure 10.57 shows these forces for an isolated portion of a structure subjected to lateral loads.

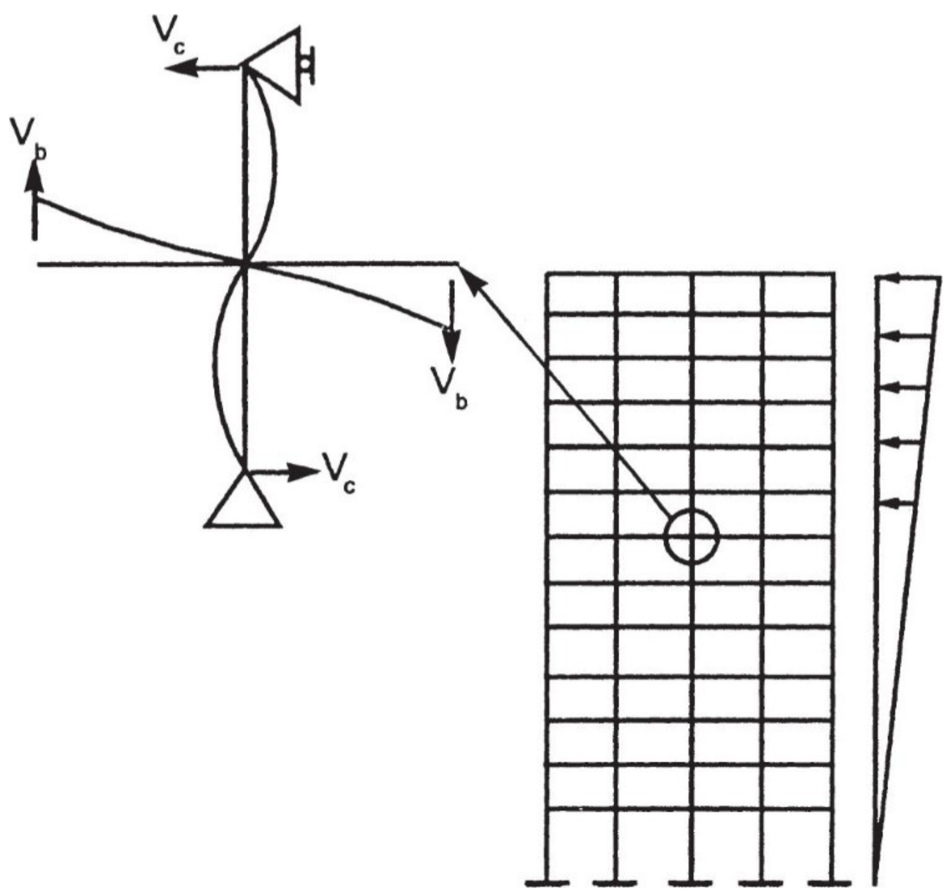


FIGURE 10.57 Assumed forces on an interior joint in a frame subjected to lateral loads.

Assumption (2) is valid for the cases where the moment capacity of columns is relatively larger than the beam capacity.

Figure 10.58 shows the free-body diagram (FBD) of the beam web within the joint and upper column at ultimate load. With reference to Fig. 10.58 the following additional assumptions are made in deriving the design equations:

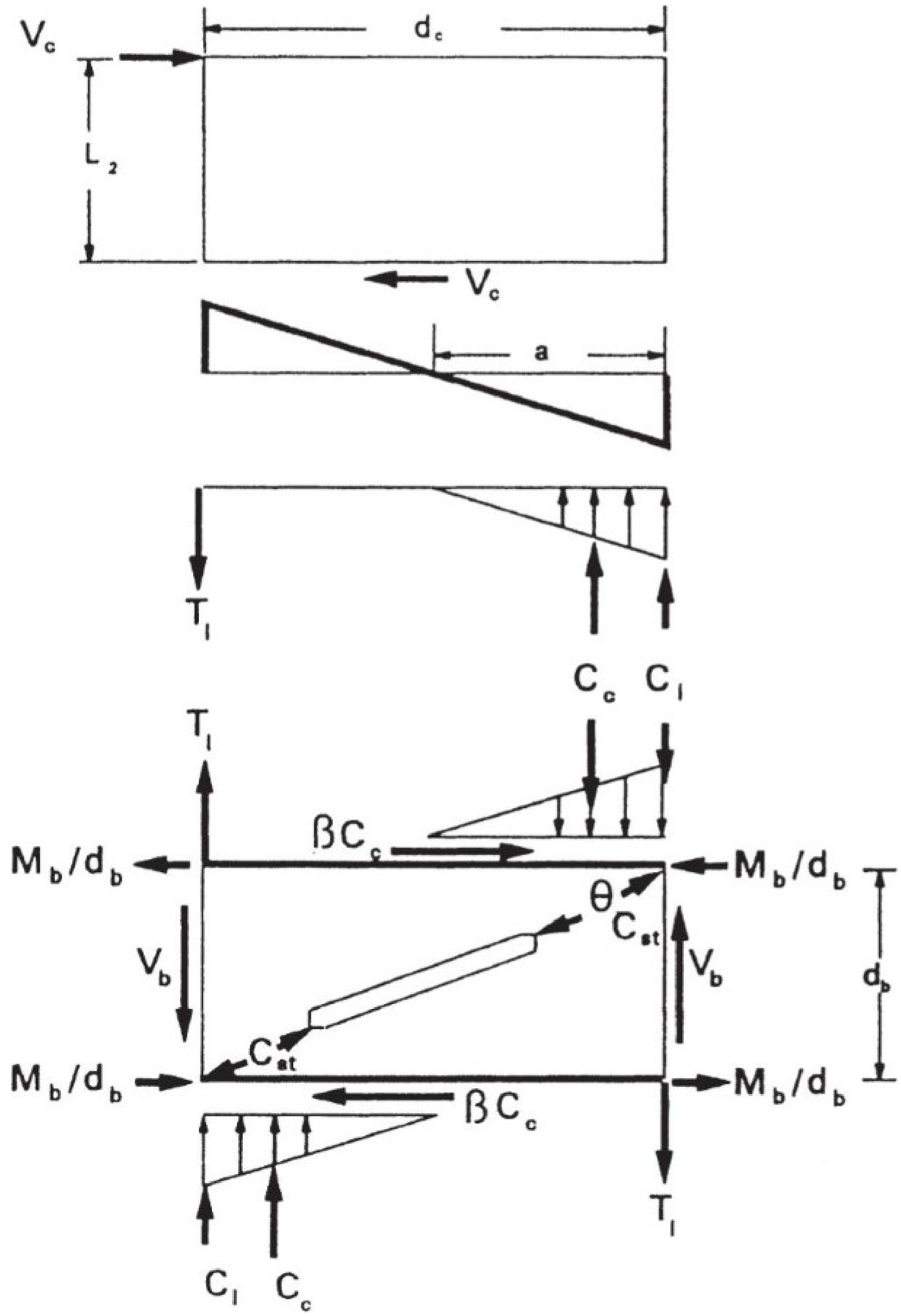


FIGURE 10.58 FBD of upper column and beam web within the joint.

1. The concrete stress distribution is assumed to be linear. The width of the concrete stress block is assumed to equal b_f, the beam flange width.
2. As shown in Fig. 10.58, the strain distribution over the upper column is assumed to be linear.
3. The steel tube and concrete act compositely.

4. The portion of the upper column shear, V_c, transferred to the steel beam is assumed to be ßC_c, where C_c is the resultant concrete compressive force bearing against the beam flange and ß is the coefficient of friction.
5. Applied beam moments are resolved into couples concentrated at beam flanges.
6. The resultant of the concrete compression strut is along a diagonal as shown in Fig. 10.58.

Considering the preceding assumptions and the strain distribution shown for the upper column in Fig. 10.58, the maximum strain in concrete, ε_c, could be related to ε_l, the steel pipe strain in tension:

$$\varepsilon_c = \frac{a}{d_c - a}\varepsilon_l \tag{10.45}$$

The maximum stress in the concrete and the stresses in the steel tube can be calculated as follows:

$$f_c = E_c \varepsilon_c \tag{10.46}$$

$$f_{lc} = E_s \varepsilon_c \tag{10.47}$$

$$f_{lt} = E_s \varepsilon_l \tag{10.48}$$

where f_c, f_{lc}, and f_{lt} are the maximum concrete compressive stress, the stress in the steel pipe in compression, and the stress in the steel pipe in tension, respectively.

Substituting Eq. (10.45) into Eqs. (10.46) to (10.48) and multiplying Eqs. (10.46) to (10.48) by the corresponding area, the resultant forces for different connection elements can be calculated as follows:

$$C_c = \left(\frac{1}{2}\right)\left(\frac{1}{\eta}\right)\xi b_f \frac{a^2}{d_c - a} f_{yl} \tag{10.49}$$

$$C_l = \gamma \xi b_f t_l \frac{a}{d_c - a} f_{yl} \tag{10.50}$$

$$T_l = \xi \gamma b_f t_l f_{yl} \tag{10.51}$$

Using the FBD of the upper column, shown in Fig. 10.58, Eqs. (10.49) to (10.51), and satisfying the vertical force equilibrium, the following equation could be obtained:

$$t_l = \frac{a^2}{d_c - 2a}\frac{1}{2\gamma\eta} \tag{10.52}$$

where d_c = diameter of the steel tube
a = depth of neutral axis
η = ratio of modulus of elasticity for steel over modulus of elasticity of concrete
t_l = thickness of steel tube
γ = factor reflecting portion of steel tube effective in carrying tensile forces. Experimental data for square tubes indicated that it could be assumed that $\gamma = 2$. The same value is assumed for circular columns.

Considering the moment equilibrium of the FBD of the upper column shown in Fig. 10.58, the following expression can be derived:

$$\frac{1}{d_c - a}\left[\frac{a^3 d_c}{d_c - 2a} + a^2\left(d_c - \frac{a}{3}\right)\right] = \frac{2\eta}{\xi}\frac{\alpha l_2}{b_f f_{yl}} V_b \tag{10.53}$$

where f_{yl} is the yield strength of the steel tube.

In Eq. (10.53), ξf_{yl}, is the stress level that the steel tube is allowed to approach at the ultimate condition. Based on the experimental data and until further research is conducted, it is suggested that a value of 0.75 be used for ξ.

Equations (10.52) and (10.53) relate the externally applied force, V_b, directly and the externally applied forces, V_c and M_c, indirectly (through the coefficients α and l_2) to different connection parameters.

10.5.6.1 Design Approach. Before designing the through-beam connection detail, additional equations will be derived to relate the shear stress in the beam web within the joint to the compressive force in the concrete compression strut and the externally applied forces.

Considering the FBD of a portion of the beam web within the joint area as shown in Fig. 10.59 and satisfying the horizontal force equilibrium, the following equation can be derived:

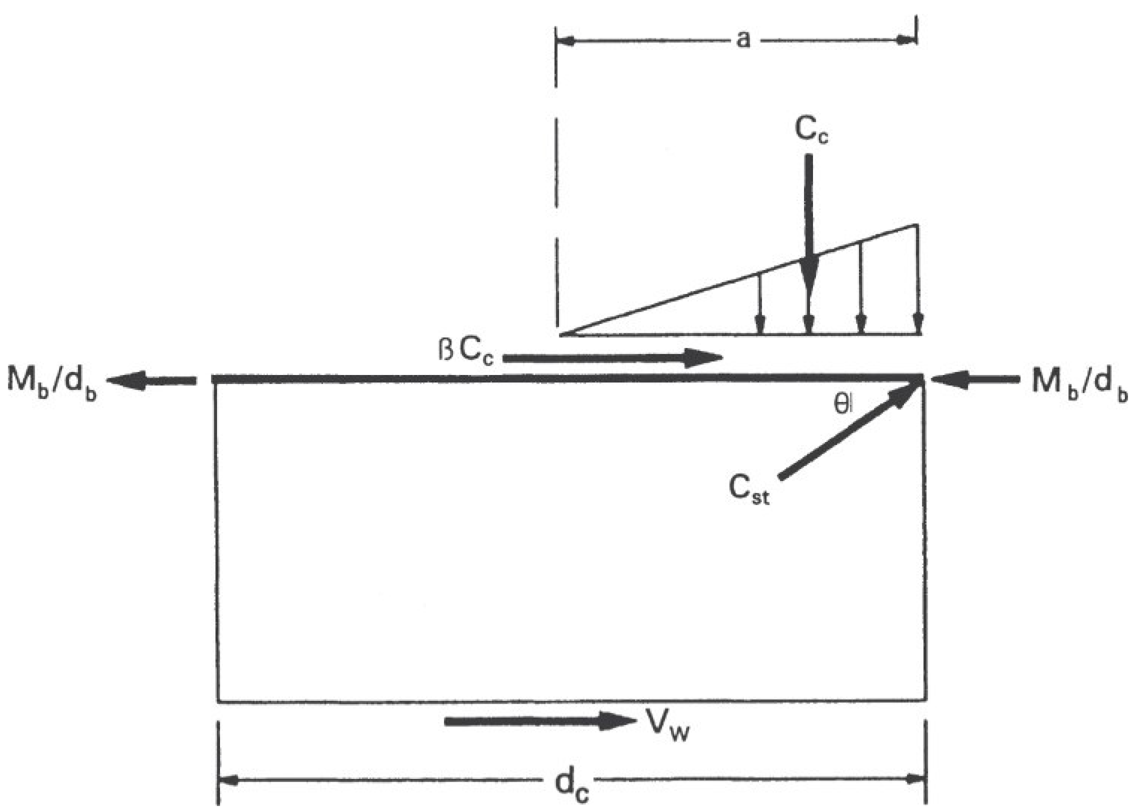


FIGURE 10.59 FBD of portion of web within joint area.

$$V_w + C_{st}\cos\theta + ßC_c - \frac{2M_b}{d_b} = 0 \tag{10.54}$$

where V_w is the shear force in the beam web at the ultimate condition and θ = arctan (d_b/d_c). Equations (10.52) to (10.54) can be used to proportion the through-beam connection detail.

Until further research is conducted, the following steps are suggested for designing the through-beam connection detail following the LRFD format:

1. From analysis, obtain the factored joint forces.
2. Select b_f, d_c, and f_{yl}.
3. Solving Eq. (10.53), obtain *a*, the depth of the neutral axis.
4. Solving Eq. (10.52), obtain t_l, the required thickness of the pipe steel.
5. Check stress in different connection elements.
6. From the vertical equilibrium requirement of the FBD shown in Fig. 10.59:

$$C_{st} = \frac{C_c}{\sin\theta} \tag{10.55}$$

Using Eq. (10.49), calculate C_c and then using Eq. (10.55) calculate C_{st}.

7. Using Eq. (10.54), calculate V_w, the shear force in the beam at the ultimate condition and compare it to V_{wy}, the shear yield capacity of the beam web given by

$$V_{wy} = 0.6F_{yw}t_w d_c \tag{10.56}$$

where F_{yw} is the beam web yield stress and t_w is the thickness of the beam web. If necessary increase the thickness of the web within the joint region. In this design procedure the assumption is that at the factored load level, the web starts to yield.

8. Check the shear stress in the concrete in the joint area. The limiting shear force could be assumed to be as suggested by ACI 352:

$$V_u = \phi R\sqrt{f'_c}\,A_e \tag{10.57}$$

where $\phi = 0.85$

$R = 20$, 15, and 12 for interior, exterior, and corner joints, respectively

f'_c = concrete compressive strength

It is suggested that the value of f'_c be limited to 70 MPa, implying that in the case of 100-MPa concrete, for instance, f'_c be taken as 70 MPa rather than 100 MPa.

10.5.6.2 Design Example. Design a through-beam connection detail with the following geometry and properties:

Given steps 1 and 2:

$$b_f = 139.7\text{ mm}$$
$$d_b = 368.3\text{ mm}$$
$$d_c = 406.4\text{ mm}$$
$$f_{yl} = 248.22\text{ MPa}$$
$$F_{yw} = 248.22\text{ MPa}$$
$$t_w = 6.35\text{ mm}$$
$$\alpha = 0.85$$
$$l_2 = 812.8\text{ mm}$$
$$V_b = 351.39\text{ kN}$$
$$M_b = 187.54\text{ kN}\cdot\text{m}$$
$$ß = 0.5$$
$$\xi = 0.75$$
$$\eta = 4.3$$
$$f'_c = 96.53\text{ MPa}$$
$$E_s = 200\text{ GPa (modulus of elasticity of steel)}$$
$$E_c = 46\text{ GPa (modulus of elasticity of concrete)}$$

Step 3: Using Eq. (10.53), calculate *a*, the depth of the neutral axis. Equation (10.53) will result in a third-degree polynomial which can be shown to have only one positive, real root. For this example Eq. (10.53) results in a = 149.35 mm.

Step 4: Using Eq. (10.52), calculate the required thickness of the steel pipe (use t_l = 12.0 mm):

$$t_l = \frac{149.35^2}{406.4 - 2(149.35)}\,\frac{1}{2(2)(4.3)} = 12.04\,\text{mm}$$

Step 5: Check stresses in different connection elements against their limit values. First calculate the tensile strain in the steel tube

$$\varepsilon_l = \frac{\xi f_{yl}}{E_s} = \frac{0.75(248.22)}{200{,}000} = 0.000931\text{ mm/mm}$$

Using Eqs. (10.45) and (10.46), calculate f_c

$$f_c = 24.90\text{ MPa} < f'_c = 96.53\text{ MPa}$$

Using Eqs. (10.47) and (10.48), calculate the stresses in the other connection elements. This yields

$$f_{lc} = 108.1 \text{ MPa} < \phi_c F_y = 0.85 \times 248.22 = 211 \text{ MPa}$$
$$f_{lt} = 186.2 \text{ MPa} < \phi_t F_y = 0.9 \times 248.22 = 223.4 \text{ MPa}$$

Step 6: Using Eqs. (10.49) and (10.55), calculate the compressive force in the concrete compression strut

$$\theta = \arctan\frac{368.3}{406.4} = 42.2°$$

$$C_c = \left(\frac{1}{2}\right)(1/\eta)\zeta b_f\left(\frac{a^2}{d_c - a}\right) f_{yl}$$

$$C_c = \frac{½(0.23)(0.75)(139.7)(149.35^2)/(406.4 - 149.35) \times 248.22}{1000}$$
$$= 262.42 \text{ kN}$$

$$C_{st} = \frac{C_c}{\sin\theta} = \frac{262.42}{\sin 42.2} = 390.67 \text{ kN}$$

Step 7: Using Eq. (10.54), compute V_w

$$V_w + C_{st}\cos\theta + ßC_c - \frac{2M_b}{d_b} = 0$$

$$V_w + 390.67\cos(42.2) + 0.5(262.42) - \frac{2 \times 187.54 \times 10^3}{368.3} = 0$$

$$V_w = 597.79 \text{ kN}$$

From Eq. (10.56) the shear yield capacity of the beam is

$$V_{wy} = \frac{0.6 \times 248.22 \times 6.35 \times 406.4}{1000} = 384.3 \text{ kN} < 597.79 \text{ kN}$$

Since the shear yield capacity of the web within the joint is not sufficient, using Eq. (10.56), increase the web thickness to

$$t_w = \frac{597.79}{0.6 \times 248.22 \times 406.4/1000} = 9.88 \text{ mm}$$

$$t_w = 10 \text{ mm}$$

Step 8: The shear force carried by concrete within the joint between the beam flanges is assumed to be the horizontal component, C_{st}

$$V_c = C_{st} \cos\theta$$

$$V_c = 390.67 \cos (42.2) = 289.41 \text{ kN}$$

For the interior joint the shear capacity is

$$V_u = \phi(20)\sqrt{f_c}\,(2b_f)\,(d_c)$$

$$V_u = 0.85(20)6.895 \times 10^{-3} \times 100 \times \frac{(2 \times 139.7)(406.4)}{1000}$$

$$= 1330.95 \text{ kN} > 289.41 \text{ kN}$$

10.6 NOTATIONS (FOR SEC. 10.3)

a	shear span taken as one-half of coupling beam, in
A_b	cross-sectional area of stud
A_v	total area of web reinforcement in concrete encasement around steel coupling beam
A_{vd}	total area of reinforcement in each group of diagonal bars
b	width of embedded steel plate
b_f	steel coupling beam flange width
b_w	web width of encasing element around steel coupling beam
d	distance from the extreme compression fiber to centroid of longitudinal tension reinforcement in the encasing element around steel coupling beam
d'	distance from the extreme compression fiber to centroid of longitudinal compression reinforcement
d_e	distance from the stud axis to the edge of wall
d_{e1} and d_{e2}	distance from the axis of extreme studs to the edge of wall
d_h	diameter of stud head
e	eccentricity of gravity shear measured from centerline of bolts to face of wall
f'_c	concrete compressive strength, psi [for Eq. (10.2) this is for the concrete used in wall piers]
f_y	yield stress of reinforcing bars or studs
F_y	yield strength of web
h	steel coupling beam depth/distance from the center of resistance of tension studs to edge of the embedded steel plate
H	overall depth of coupling beam
I_g	gross concrete section moment of inertia
k_d	depth of concrete compression block
l_e	embedment length of studs (stud length—thickness of head)
l_n	clear span of coupling beam measured from face of wall piers
L	distance between centerlines of wall piers

L_b	clear distance between wall piers
L_e	embedment length of steel coupling beams inside wall piers
M_u	ultimate coupling beam moment
n	number of studs
P_c	tensile strength of studs based on concrete
P_s	tensile strength of studs based on steel
R	code-specified response modification factor
s	spacing of web reinforcement in encasing element around steel coupling beam
t_f	flange thickness
t_w	web thickness
t_{wall}	wall thickness, in
T_{capacity}	tensile capacity of studs
T_u	calculated tensile force in collector/outrigger beam
V_P	plastic shear capacity
V_s	shear strength of stud governed by steel
V_u	ultimate coupling beam shear force/calculated gravity shear in outrigger or collector beam
x	horizontal distance between outermost studs
y	vertical distance between studs
α	angle between diagonal reinforcement and longitudinal steel
$ß_1$	ratio of the average concrete compressive strength to the maximum stress as defined by ACI Building Code
γ_p	coupling beam plastic shear angle
θ_e	elastic interstory drift angle
θ_p	plastic interstory drift angle
ϕ	strength reduction factor taken as 0.85

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APPENDIX A

STRUCTURAL SHAPES—DIMENSIONS AND GENERAL INFORMATION[1]

Structural Shape Size Groupings
(Structural Shape Sizes per Tensile Group Classifications)

Group 1	Group 2	Group 3	Group 4	Group 5
		W shapes		
W 24 × 55	W 44 × 198	W 44 × 248	W 40 × 362 to W 40 × 655	W 36 × 848
W 24 × 62	W 44 × 224	W 44 × 285	W 36 × 328 to W 36 × 798	W 14 × 605 to W 14 × 730
W 21 × 44 to W 21 × 57	W 40 × 149 to W 40 × 268	W 40 × 277 to W 40 × 328	W 33 × 318 to W 33 × 619	
W 18 × 35 to W 18 × 71	W 36 × 135 to W 36 × 210	W 36 × 230 to W 36 × 300	W 30 × 292 to W 30 × 581	
W 16 × 26 to W 16 × 57	W 33 × 118 to W 33 × 152	W 33 × 201 to W 33 × 291	W 27 × 281 to W 27 × 539	
W 14 × 22 to W 14 × 53	W 30 × 90 to W 30 × 211	W 30 × 235 to W 30 × 261	W 24 × 250 to W 24 × 492	
W 12 × 14 to W 12 × 58	W 27 × 84 to W 27 × 178	W 27 × 194 to W 27 × 258	W 21 × 248 to W 21 × 402	
W 10 × 12 to W 10 × 45	W 24 × 68 to W 24 × 162	W 24 × 176 to W 24 × 229	W 18 × 211 to W 18 × 311	
W 8 × 10 to W 8 × 48	W 21 × 62 to W 21 × 147	W 21 × 166 to W 21 × 223	W 14 × 233 to W 14 × 550	
W 6 × 9 to W 6 × 25	W 18 × 76 to W 18 × 143	W 18 × 158 to W 18 × 192	W 12 × 21 to W 12 × 336	
W 5 × 16 W 5 × 19	W 16 × 67 to W 16 × 100	W 14 × 145 to W 14 × 211		

W 4 × 13	W 14 × 61 to W 14 × 132	W 12 × 120 to W 12 × 190
	W 12 × 65 to W 12 × 106	
	W 10 × 49 to W 10 × 112	
	W 8 × 58	
	W 8 × 67	
	M shapes	
To 37.7 lb/ft		
	S shapes	
To 35 lb/ft		
	HP shapes	
	To 102 lb/ft	>102 lb/ft
	Standard channel	
To 20.7 lb/ft	>20.7 lb/ft	
	Miscellaneous channel	
To 28.5 lb/ft	>28.5 lb/ft	
	Angle iron	
To ½ in	>½ to ¾ in	>¾ in

Wide-Flange Dimensions, in

Designation	Depth	Web thickness	Flange	
			Width	Thickness
W-shape dimensions				
W 44 × 285	44	1	11¾	1¾
× 248	43⅝	⅞	11¾	1-9/16
× 224	43¼	13/16	11¾	1-7/16
× 198	42⅞	11/16	11¾	1¼
W 40 × 328	40	15/16	17⅞	1¾
× 298	39¾	13/16	17⅞	1-9/16
× 268	39⅜	¾	17¾	1-7/16
× 244	39	11/16	17¾	1¼
× 221	38⅝	11/16	17¾	1-1/16
× 192	38¼	11/16	17¾	13/16
W 40 × 655	43⅝	2	16⅞	3-9/16
× 593	43	1-13/16	16¾	3-1/5
× 531	42⅜	1⅝	16½	2-15/16
× 480	41¾	1-7/16	16⅜	2-⅝
× 436	41⅜	1-5/16	16¼	2⅜
× 397	41	1¼	16⅛	2-3/16
× 362	40½	1⅛	16	2
× 324	40⅛	1	15⅞	13/16
× 297	39⅞	15/16	15⅞	1-⅝
× 277	39¾	13/16	15⅞	1-9/16
× 249	39⅜	¾	15¾	1-7/16
× 215	39	⅝	15¾	1¼
× 199	38⅝	⅝	15¾	1-1/16
W 40 × 183	39	⅝	11¾	1¼
× 167	38⅝	⅝	11¾	1
× 149	38¼	⅝	11¾	13/16
W 36 × 848	42½	2½	18⅛	4½
× 798	42	2⅜	18	4-5/16
× 720	41¼	2-3/16	17-3/5	3⅞
× 650	40½	2	17⅝	3-9/16
× 588	39⅞	1-3/16	17⅜	3¼
× 527	39¼	1⅝	17¼	2-15/16
× 485	38¾	1½	17⅛	2-11/16
× 439	38¼	1⅜	17	2-7/16
× 393	37¾	1¼	16⅞	2-3/16
× 359	37⅜	1⅛	16¾	2
× 328	37⅛	1	16⅝	1⅞
× 300	36¾	15/16	16⅝	1-11/16
× 280	36½	⅞	16⅝	1-9/16
× 260	36¼	13/16	16½	1-7/16
× 245	36⅛	13/16	16½	1⅜
× 230	35⅞	¾	16½	1¼
W 36 × 256	37⅜	1	1¾	1¾
× 232	37⅛	⅞	1-9/16	1-9/16
× 210	36¾	13/16	1⅜	1⅜
× 194	36½	¾	1¼	1¼
× 182	36⅜	¾	1-3/16	1-3/16
× 170	36⅛	11/16	1⅛	1⅛
× 160	36	⅝	1	1
× 150	35⅞	⅝	15/16	15/16
× 135	35½	⅝	15/16	13/16

W 33 × 619	38½	2	16⅞	3-9/16
× 567	37⅞	1-13/16	16¾	3¼
× 515	37⅜	1⅝	16⅝	3
× 468	36¾	1½	16½	2¾
× 424	36⅜	1⅜	16⅜	2½
× 387	36	1¼	16¼	2¼
× 354	35½	1-13/16	16⅛	2-1/16
× 318	35⅛	1-1/16	16	1⅞
× 291	34⅞	1	15⅞	1¾
× 263	34½	⅞	15¾	1-9/16
× 241	34⅛	13/16	15⅞	1⅜
× 221	33⅞	¾	15¾	1¼
× 201	33⅝	11/16	15¾	1⅛
W 33 × 169	33⅞	11/16	11½	1¼
× 152	33½	⅝	11⅝	1-1/16
× 141	33¼	⅝	11½	15/16
× 130	33⅛	9/16	11½	⅞
× 118	32⅞	9/16	11½	¾
W 30 × 581	35⅜	2	16¼	3-9/16
× 526	34¾	1-13/16	16	3¼
× 477	34¼	1⅝	15⅞	3
× 433	33⅝	1½	15¾	2-11/16
× 391	33¼	1⅜	5⅝	2⅞
× 357	32¾	1¼	15½	2¼
× 326	32⅜	1⅛	15⅜	2-1/16
× 292	32	1	15¼	1⅞
× 261	31⅝	15/16	15⅛	1⅝
× 235	31¼	13/16	15	1½
× 211	31	¾	15⅛	1-5/16
× 191	30⅝	11/16	15	1-3/16
× 173	30½	⅝	15	1-1/16
W 30 × 148	30⅝	⅝	10½	1-3/16
× 132	30¼	⅝	10½	1
× 124	30⅛	9/16	10½	15/16
× 116	30	9/16	10½	⅞
× 108	29⅞	9/16	10½	¾
× 99	29⅝	½	10½	11/16
× 90	29½	½	10⅜	9/16

W 27 × 539	32½	2	15¼	3-9/16
× 494	32	1-13/16	15⅛	3¼
× 448	31⅜	1⅝	15	3
× 407	30⅞	1½	14¾	2¾
× 368	30⅜	1⅜	14⅝	2½
× 336	30	1¼	14½	2¼
× 307	29⅝	1-3/16	14½	2-1/16
× 281	29¼	1-1/16	14⅜	1-15/16
× 258	29	1	14¼	1¾
× 235	28⅝	15/16	14¼	1⅝
× 217	28⅜	13/16	14⅛	1½
× 194	28⅛	¾	14	1-5/16
× 178	27¾	¾	14⅛	1-3/16
× 161	27⅝	11/16	14	1-1/16
× 146	27⅜	⅝	14	1
W 27 × 129	27⅝	⅝	10	1⅛
× 114	27¼	9/16	10⅛	15/16
× 102	27⅛	½	10	13/16
× 94	26⅞	½	10	¾
× 84	26¾	7/16	10	⅝
W 24 × 492	29⅝	2	14⅛	3-9/16
× 450	29⅛	1-13/16	14	3¼
× 408	28½	1⅝	13¾	3
× 370	28	1½	13⅝	2¾
× 335	27½	1⅜	13½	2½
× 306	27⅛	1¼	13⅜	2¼
× 279	26¾	1-3/16	13¼	2-1/16
× 250	26⅜	1-1/16	13⅛	1⅞
× 229	26	1	13⅛	1¾
× 207	25¾	⅞	13	1-9/16
× 192	25½	13/16	13	1-7/16
× 176	25¼	¾	12⅞	1-5/16
× 162	25	11/16	13	1¼
× 146	24¾	⅝	12⅞	1-1/16
× 131	24½	⅝	12⅞	15/16
× 117	24¼	9/16	12¾	⅞
× 104	24	½	12¾	¾

W 24 × 103	24½	9/16	9	1
× 94	24¼	½	9⅛	⅞
× 84	24⅛	½	9	¾
× 76	23⅞	7/16	9	11/16
× 68	23¾	7/16	9	9/16
W 24 × 62	23¾	7/16	7	9/16
× 55	23⅝	⅜	7	½
W 21 × 402	26	1¾	13⅜	3⅛
× 364	25½	1-9/16	13¾	2⅞
× 333	25	1-7/16	13⅛	2⅝
× 300	24½	1-5/16	13	2⅜
× 275	24⅛	1¼	12⅞	2-3/16
× 248	23¾	1⅛	12¾	2
× 223	23⅜	1	12⅝	1-13/16
× 201	23	15/16	12⅝	1⅝
× 182	22¾	13/16	12½	1½
× 166	22½	¾	12⅜	1⅜
× 147	22	¾	12½	1⅛
× 132	21⅞	⅝	12½	1-1/16
× 122	21⅝	⅝	12⅜	15/16
× 111	21½	9/16	12⅜	⅞
× 101	21⅜	½	12¼	13/16
W 21 × 93	21⅝	9/16	8⅜	15/16
× 83	21⅜	½	8⅜	13/16
× 73	21¼	7/16	8¼	¾
× 68	21⅛	7/16	8¼	11/16
× 62	21	⅜	8¼	⅝
W 21 × 57	21	⅜	6½	⅝
× 50	20⅞	⅜	6½	9/16
× 44	20⅝	⅜	6½	7/16
W 18 × 311	22⅜	1½	12	2¾
× 283	21⅞	1⅜	11⅞	2½
× 258	21½	1¼	11¾	2-5/16
× 234	21	1-3/16	11⅝	2⅛
× 211	20⅝	1-1/16	11½	1-15/16
× 192	20⅜	1	11½	1¾
× 175	20	⅞	11⅜	1-9/16
× 158	10¾	13/16	11¼	1-7/16
× 143	19½	¾	11¼	1-5/16
× 130	19¼	11/16	11⅛	1-3/16
W 18 × 119	19	⅝	11¼	1-1/16
× 106	18¾	9/16	11¼	15/16
× 97	18⅝	9/16	11⅛	⅞
× 86	18⅜	½	11⅛	¾
× 76	18¼	7/16	11	11/16

W 18 × 71	18½	½	7⅝	13/16
× 65	18⅜	7/16	7⅝	¾
× 60	18¼	7/16	7½	11/16
× 55	18⅛	⅜	7½	⅝
× 50	18	⅜	7½	9/16
W 18 × 46	18	⅜	6	⅝
× 40	17⅞	5/16	6	½
× 35	17¾	5/16	6	7/16
W 16 × 100	17	9/16	10⅜	1
× 89	16¾	½	10⅜	⅞
× 77	16½	7/16	10¼	¾
× 67	16⅜	⅜	10¼	11/16
W 16 × 57	16⅜	7/16	7⅛	11/16
× 50	16¼	⅜	7⅛	⅝
× 45	16⅛	⅜	7	9/16
× 40	16	5/16	7	½
× 36	15⅞	5/16	7	7/16
W 16 × 31	15⅞	¼	5½	7/16
× 26	15¾	¼	5½	⅜
W 14 × 730	22⅜	3-1/16	17⅞	4-15/16
× 665	21⅝	2-13/16	17⅝	4½
× 605	20⅞	2⅝	17⅜	4-3/16
× 550	20¼	2⅜	17¼	3-13/16
× 500	19-6/8	2-3/16	17	3½
× 455	19	2	16⅞	3-3/16
W 14 × 426	18⅝	1⅞	16¾	3-1/16
× 398	18¼	1¾	16⅝	2⅞
× 370	17⅞	1⅝	16½	2-11/16
× 342	17½	1-9/16	16⅜	2½
× 311	17⅛	1-7/16	16¼	1¼
× 283	16¾	1-5/16	16⅛	2-1/16
× 257	16⅜	1-3/16	16	1⅞
× 233	16	1-1/16	15⅞	1¾
× 211	15¾	1	15¾	1-9/16
× 193	15½	⅞	15¾	1-7/16
× 176	15¼	13/16	15⅝	1-5/16
× 159	15	¾	15⅝	1-3/16
× 145	14¾	11/16	15½	1-1/16
W 14 × 132	14⅝	⅝	14¾	1
× 120	14½	9/16	14⅝	15/16
× 109	14⅜	½	14⅝	⅞
× 99	14⅛	½	14⅝	¾
× 90	14	7/16	14½	11/16
W 14 × 82	14¼	½	10⅛	⅞
× 74	14⅛	7/16	10⅛	13/16
× 68	14	7	10	¾
× 61	13⅞	16	10	⅝

W 14 × 53	13⅞	⅜	8	11/16
× 48	13¾	⅜	8	⅝
× 43	13⅝	5/16	8	½
W 14 × 38	14⅛	5/16	6¾	½
× 34	14	5/16	6¾	7/16
× 30	13⅞	¼	6¾	⅜
W 14 × 26	13⅞	¼	5	7/16
× 22	13¾	¼	5	5/16
W 12 × 336	16⅞	1¾	13⅜	2-15/16
× 305	16⅜	1⅝	13¼	2-11/16
× 279	15⅞	1½	13⅛	2½
× 252	15⅜	1⅜	13	2¼
× 230	15	1-5/16	12⅞	2-1/16
× 210	14¾	1-3/16	12¾	1⅞
× 190	14⅜	1-1/16	12⅝	1¾
× 170	14	15/16	12⅝	1-9/16
× 152	13¾	⅞	12½	1⅜
× 136	13⅜	13/16	12⅜	1¼
× 120	13⅛	11/16	12⅜	1⅛
× 106	12⅞	⅝	12¼	1
× 96	12¾	9/16	12⅛	⅞
× 87	12½	½	12⅛	13/16
× 79	12⅜	½	12⅛	¾
× 72	12¼	7/16	12	11/16
× 65	12⅛	⅜	12	⅝
W 12 × 58	12¼	⅜	10	⅝
× 53	12	⅜	10	9/16
W 12 × 50	12¼	⅜	8⅛	⅝
× 45	12	5/16	8	9/16
× 40	12	5/16	8	½
W 12 × 35	12½	5/16	6½	½
× 30	12⅜	¼	6½	7/16
× 26	12¼	¼	6½	⅜
W 12 × 22	12¼	¼	4	7/16
× 19	12⅛	¼	4	⅜
× 16	12	¼	4	¼
× 14	11⅞	3/16	4	¼
W 10 × 112	11⅜	¾	10⅜	1¼
× 100	11⅛	11/16	10⅜	1⅛
× 88	10⅞	⅝	10¼	1
× 77	10⅝	½	10¼	⅞
× 68	10⅜	½	10⅛	¾
× 60	10¼	7/16	10⅛	11/16
× 54	10⅛	⅜	10	⅝
× 49	10	5/16	10	9/16

W 10 × 45	10⅛	⅜	8	⅝
× 39	9⅞	5/16	8	½
× 33	9¾	5/16	8	7/16
W 10 × 30	10½	5/16	5¾	½
× 26	10⅜	¼	5¾	7/16
× 22	10⅛	¼	5¾	⅜
W 10 × 19	10¼	¼	4	⅜
× 17	10⅛	¼	4	5/16
× 15	10	¼	4	¼
× 12	9⅞	3/16	4	3/16
W 8 × 67	9	9/16	8¼	15/16
× 58	8¾	½	8¼	13/16
× 48	8½	⅜	8⅛	11/16
× 40	8¼	⅜	8⅛	9/16
× 35	8⅛	5/16	8	½
× 31	8	5/16	8	7/16
× 28	8	5/16	6½	7/16
× 24	7⅞	¼	6½	⅜
× 21	8¼	¼	5¼	⅜
× 18	8⅛	¼	5¼	5/16
× 15	8⅛	¼	4	5/16
× 13	8	¼	4	¼
× 10	7⅞	3/16	4	3/16
W 6 × 25	6⅜	5/16	6⅛	7/16
× 20	6¼	¼	6	⅜
× 15	6	¼	6	¼
× 16	6¼	¼	4	⅜
× 12	6	¼	4	¼
× 9	5⅞	3/16	4	3/16
W 5 × 19	5⅛	¼	5	7/16
× 16	5	¼	5	⅜
W 4 × 13	4⅛	¼	4	⅜
M-shape dimensions				
M 14 × 18	14	3/16	4	¼
M 12 × 11.8	12	3/16	3⅛	¼
M 12 × 10.8	12	3/16	3⅛	¼
M 12 × 10	12	3/16	3¼	3/16
M 10 × 9	10	3/16	2¾	3/16
M 10 × 8	10	3/16	2¾	3/16
M 10 × 7.5	10	⅛	2¾	3/16
M 8 × 6.5	8	⅛	2¼	3/16
M 6 × 4.4	6	⅛	1⅞	3/16
M 5 × 18.9	5	5/16	5	7/16

		S-shape dimensions		
S 24 × 121	24½	13/16	8	1-1/16
× 106	24½	⅝	7⅞	1-1/16
S 24 × 100	24	¾	7¼	⅞
× 90	24	⅝	7⅛	⅞
× 80	24	½	7	⅞
S 20 × 96	20¼	13/16	7¼	15/16
× 86	20¼	11/16	7	15/16
S 20 × 75	20	⅝	6⅜	13/16
× 66	20	½	6¼	13/16
S 18 × 70	18	11/16	6¼	11/16
× 54.7	18	7/16	6	11/16
S 15 × 50	15	9/16	5⅝	⅝
× 42.9	15	7/16	5½	⅝
S 12 × 50	12	11/16	5½	11/16
× 40.8	12	7/16	5¼	11/16
S 12 × 35	12	7/16	5⅛	9/16
× 25.4	12	⅜	5	9/16
S 10 × 35	10	⅝	5	½
× 25.4	10	5/16	4⅝	½
S 8 × 23	8	7/16	4⅛	7/16
× 18.4	8	¼	4	7/16
S 7 × 20	7	7/16	3⅞	⅜
× 15.3	7	¼	3⅝	⅜
S 6 × 17.25	6	7/16	3⅝	⅜
× 12.5	6	¼	3⅜	⅜
S 5 × 14.75	5	½	3¼	5/16
× 10	5	3/16	3	5/16
S 4 × 9.5	4	5/16	2¾	5/16
× 7.7	4	3/16	2⅝	5/16
S 3 × 7.5	3	⅜	2½	¼
× 5.7	3	3/16	2⅜	¼
		HP-shape dimensions		
HP 14 × 117	14½	13/16	14⅞	13/16
× 102	14	11/16	14¾	11/16
× 89	13⅞	⅝	14¾	⅝
× 73	13⅝	½	14⅝	½
HP 13 × 100	13⅛	¾	13¼	¾
× 87	13	11/16	13⅛	11/16
× 73	12¾	9/16	13	9/16
× 60	12½	7/16	12⅞	7/16

HP 12 × 84	12¼	11/16	12¼	11/16
× 74	12⅛	⅝	12¼	⅝
× 63	12	½	12⅛	½
× 53	11¾	7/16	12	7/16
HP 10 × 57	10	9/16	10¼	9/16
× 42	9¾	7/16	10⅛	7/16
HP 8 × 36	8	7/16	8⅛	7/16
Channel iron—American Standard dimensions				
C 15 × 50	15	11/16	3¾	⅝
× 40	15	½	3½	⅝
× 33.9	15	⅜	3⅜	⅝
C 12 × 30	12	½	3⅛	½
× 25	12	⅜	3	½
× 20.7	12	5/16	3	½
C 10 × 30	10	11/16	3	7/16
× 25	10	½	2⅞	7/16
× 20	10	⅜	2¾	7/16
× 15.3	10	¼	2⅝	7/16
C 9 × 20	9	7/16	2⅝	7/16
× 15	9	5/16	2½	7/16
× 13.4	9	¼	2⅜	7/16
C 8 × 18.75	8	½	2½	⅜
× 13.75	8	5/16	2⅜	⅜
× 11.5	8	¼	2¼	⅜
C 7 × 14.75	7	7/16	2¼	⅜
× 12.25	7	5/16	1¼	⅜
× 9.8	7	3/16	1⅛	⅜
C 6 × 13	6	7/16	2⅛	5/16
× 10.5	6	5/16	2	5/16
× 8.2	6	3/16	1⅞	5/16
C 5 × 9	5	5/16	1⅞	5/16
× 6.7	5	3/16	1¾	5/16
C 4 × 7.25	4	5/16	1¾	5/16
× 5.4	4	3/16	1⅝	5/16
C 3 × 6	3	⅜	1⅝	¼
× 5	3	¼	1½	¼
× 4.1	3	3/16	1⅜	¼
Channel iron—miscellaneous dimensions				
MC 18 × 58	18	11/16	4¼	⅝
× 51.9	18	⅝	4⅛	⅝
× 45.8	18	½	4	⅝
× 42.7	18	7/16	4	⅝
MC 13 × 50	13	13/16	4⅜	⅝
× 40	13	9/16	4⅛	⅝
× 35	13	7/16	4⅛	⅝
× 31.8	13	⅜	4	⅝

MC 12 × 50	12	13/16	4⅛	11/16
× 45	12	11/16	4	11/16
× 40	12	9/16	3⅞	11/16
× 35	12	7/16	3¾	11/16
× 31	12	⅜	3⅝	11/16
MC 12 × 10.6	12	3/16	1½	5/16
MC 10 × 41.1	10	13/16	4⅜	9/16
× 33.6	10	9/16	4⅛	9/16
× 28.5	10	7/16	4	9/16
MC 10 × 25	10	⅜	3⅜	9/16
× 22	10	5/16	3⅜	9/16
MC 10 × 8.4	10	3/16	1½	¼
MC 10 × 6.5	10	⅛	1⅛	3/16
MC 9 × 25.4	9	7/16	3½	9/16
× 23.9	9	⅜	3½	9/16
MC 8 × 22.8	8	7/16	3½	½
× 21.4	8	⅜	3½	½
MC 8 × 20	8	⅜	3	½
× 18.7	8	⅜	3	½
MC 8 × 8.5	8	3/16	1⅞	5/16
MC 7 × 22.7	7	½	3⅝	½
× 19.1	7	⅜	3½	½
MC 6 × 18	6	⅜	3½	½
× 15.3	6	5/16	3½	⅜
MC 6 × 16.3	6	⅜	3	½
× 15.1	6	5/16	3	½
MC 6 × 12	6	5/16	2½	⅜

Pipe Dimensions—Standard Weight, in

Nominal diameter	Outside diameter	Inside diameter	Wall thickness	Weight per foot
½	0.840	0.622	0.109	0.85
¾	1.050	0.824	0.113	1.13
1	1.315	1.049	0.133	1.68
1¼	1.660	1.380	0.140	2.27
1½	1.900	1.610	0.145	2.72
2	2.375	2.067	0.154	3.65
2½	2.875	2.469	0.203	5.79
3	3.500	3.068	0.216	7.58
3½	4.000	3.548	0.226	9.11
4	4.500	4.026	0.237	10.79
5	5.563	5.047	0.258	14.62
6	6.625	6.065	0.280	18.97
8	8.625	7.981	0.322	28.55
10	10.750	10.020	0.365	40.48
12	12.750	12.000	0.375	49.56

Pipe Dimensions—Extra Strong, in

Nominal diameter	Outside diameter	Inside diameter	Wall thickness	Weight per foot
½	0.840	0.546	0.147	1.09
¾	1.050	0.742	0.154	1.47
1	1.315	0.957	0.179	2.17
1¼	1.660	1.278	0.191	3.00
1½	1.900	1.500	0.200	3.63
2	2.375	1.939	0.218	5.02
2½	2.875	2.323	0.276	7.66
3	3.500	2.900	0.300	10.25
3½	4.000	3.364	0.318	12.50
4	4.500	3.826	0.337	14.98
5	5.563	4.813	0.375	20.78
6	6.625	5.761	0.432	28.57
8	8.625	7.625	0.500	43.39
10	10.750	9.750	0.500	54.74
12	12.750	11.750	0.500	65.42

Pipe Dimensions—Double Extra Strong, in

Nominal diameter	Outside diameter	Inside diameter	Wall thickness	Weight per foot
2	2.375	1.503	0.436	9.03
2½	2.875	1.771	0.552	13.69
3	3.500	2.300	0.600	18.58
4	4.500	3.152	0.674	27.54
5	5.563	4.063	0.750	38.55
6	6.625	4.897	0.864	53.16
8	8.625	6.875	0.875	72.42

Structural Tubing Dimensions (Square and Rectangular)

Nominal size, in	Wall thickness, in	Weight per foot
Square tubing dimensions		
16 × 16	⅝	127.37
	½	103.30
	⅜	78.52
	5/16	65.87
14 × 14	⅝	110.36
	½	89.68
	⅜	68.31
	5/16	57.36
12 × 12	⅝	93.34
	½	76.07
	⅜	58.10
	5/16	48.86
	¼	39.43
	3/16	29.84
10 × 10	⅝	76.33
	9/16	69.48
	½	62.46
	⅜	47.90
	5/16	40.35
	¼	32.63
	3/16	24.73
9 × 9	⅝	67.82
	9/16	61.83
	½	55.66
	⅜	42.79
	5/16	36.10
	¼	29.23
	3/16	22.18
8 × 8	⅝	59.32
	9/16	54.17
	½	48.85
	⅜	37.69
	5/16	31.84
	¼	25.82
	3/16	19.63
7 × 7	9/16	46.51
	½	42.05
	⅜	32.58
	5/16	27.59
	¼	22.42
	3/16	17.08
6 × 6	9/16	38.86
	½	35.24

	⅜	27.48
	5/16	23.34
	¼	19.02
	3/16	14.53
5 × 5	½	28.43
	⅜	22.37
	5/16	19.08
	¼	15.62
	3/16	11.97
4.5 × 4.5	¼	13.91
	3/16	10.70
4 × 4	½	21.63
	⅜	17.27
	5/16	14.83
	¼	12.21
	3/16	9.42
3.5 × 3.5	5/16	12.70
	¼	10.51
	3/16	8.15
3 × 3	5/16	10.58
	¼	8.81
	3/16	6.87
2.5 × 2.5	5/16	8.45
	¼	7.11
	3/16	5.59
2 × 2	5/16	6.32
	¼	5.41
	3/16	4.32
	Rectangular tubing dimensions	
20 × 12	½	103.30
	⅜	78.52
	5/16	65.87
20 × 8	½	89.68
	⅜	68.31
	5/16	57.36
20 × 4	½	76.07
	⅜	58.10
	5/16	48.86

18 × 6	½	76.07
	⅜	58.10
	5/16	48.86
16 × 12	⅝	110.36
	½	89.68
	⅜	68.31
	5/16	57.36
16 × 8	½	76.07
	⅜	58.10
	5/16	48.86
16 × 4	½	62.46
	⅜	47.90
	5/16	40.35
14 × 10	⅝	93.34
	½	76.07
	⅜	58.10
	5/16	48.86
14 × 16	½	62.46
	⅜	47.90
	5/16	40.35
	¼	32.63
14 × 4	½	55.66
	⅜	42.79
	5/16	36.10
	¼	29.23
12 × 8	⅝	76.33
	9/16	69.48
	½	62.46
	⅜	47.90
	5/16	40.35
	¼	32.63
	3/16	24.73
12 × 6	⅝	67.82
	9/16	61.83
	½	55.66
	⅜	42.79
	5/16	36.10
	¼	29.23
	3/16	22.18

12 × 4	⅝	59.32
	9/16	54.17
	½	48.85
	⅜	37.69
	5/16	31.84
	¼	25.82
	3/16	19.63
12 × 2	¼	22.42
	3/16	17.08
10 × 8	⅝	67.82
	9/16	61.83
	½	55.66
	⅜	42.79
	5/16	36.10
	¼	29.23
	3/16	22.18
	Square tubing dimensions	
10 × 6	⅝	59.32
	9/16	54.17
	½	48.85
	⅜	37.69
	5/16	31.84
	¼	25.82
	3/16	19.63
10 × 5	⅝	55.06
	9/16	50.34
	½	45.45
	⅜	35.13
	5/16	29.72
	¼	24.12
	3/16	18.35
10 × 4	9/16	46.51
	½	42.05
	⅜	32.58
	5/16	27.59
	¼	22.42
	3/16	17.08

10 × 2	⅜	27.48
	5/16	23.34
	¼	19.02
	3/16	14.53
9 × 7	⅝	59.32
	9/16	54.17
	½	48.85
	⅜	37.69
	5/16	31.84
	¼	25.82
	3/16	19.63
9 × 6	⅝	55.06
	9/16	50.34
	½	45.45
	⅜	35.13
	5/16	29.72
	¼	24.12
	3/16	18.35
9 × 5	9/16	46.51
	½	42.05
	⅜	32.58
	5/16	27.59
	¼	22.42
	3/16	17.08
	Rectangular tubing dimensions	
9 × 3	½	35.24
	⅜	27.48
	5/16	23.34
	¼	19.02
	3/16	14.53
8 × 6	9/16	46.51
	½	42.05
	⅜	32.58
	5/16	27.59
	¼	22.42
	3/16	17.08
8 × 4	9/16	38.86
	½	35.24
	⅜	27.48
	5/16	23.34
	¼	19.02
	3/16	14.53

8 × 3	½	31.84
	⅜	24.93
	5/16	21.21
	¼	17.32
	3/16	13.25
8 × 2	⅜	22.37
	5/16	19.08
	¼	15.62
	3/16	11.97
7 × 5	½	35.24
	⅜	27.48
	5/16	23.34
	¼	19.02
	3/16	14.53
7 × 4	½	31.84
	⅜	24.93
	5/16	21.21
	¼	17.32
	3/16	13.25
7 × 3	½	28.43
	⅜	22.37
	5/16	19.08
	¼	15.62
	3/16	11.97
7 × 2	¼	13.91
	3/16	10.70
6 × 5	½	31.84
	⅜	24.93
	5/16	21.21
	¼	17.32
	3/16	13.25

6 × 4	½	28.43
	⅜	22.37
	5/16	19.08
	¼	15.62
	3/16	11.97
6 × 3	⅜	19.82
	5/16	16.96
	¼	13.91
	3/16	10.70
6 × 2	⅜	17.27
	5/16	14.83
	¼	12.21
	3/16	9.42
5 × 4	⅜	19.82
	5/16	16.96
	¼	13.91
	3/16	10.70
5 × 3	½	21.63
	⅜	17.27
	5/16	14.83
	¼	12.21
	3/16	9.42
5 × 2	5/16	12.70
	¼	10.51
	3/16	8.15
4 × 3	5/16	12.70
	¼	10.51
	3/16	8.15
4 × 2	5/16	10.58
	¼	8.81
	3/16	6.87
3.5 × 3.5	¼	8.81
	3/16	6.87
3 × 2	¼	7.11
	3/16	5.59

Sheet Steel Thickness

Gage numbers and equivalent thicknesses, hot-rolled and cold-rolled sheet			Gage numbers and equivalent thicknesses, galvanized sheet		
Manufacturers' standard gage number	Thickness equivalent		Galvanized sheet gage number	Thickness equivalent	
	in	mm		in	mm
3	0.2391	6.073			
4	0.2242	5.695	8	0.1681	4.270
5	0.2092	5.314	9	0.1532	3.891
6	0.1943	4.935	10	0.1382	3.510
7	0.1793	4.554	11	0.1233	3.132
8	0.1644	4.176	12	0.1084	2.753
9	0.1495	3.800	13	0.0934	2.372
10	0.1345	3.416	14	0.0785	1.993
11	0.1196	3.038	15	0.0710	1.803
12	0.1046	2.657	16	0.0635	1.613
13	0.0897	2.278	17	0.0575	1.460
14	0.0747	1.900	18	0.0516	1.311
15	0.0673	1.709	19	0.0456	1.158
16	0.0598	1.519	20	0.0396	1.006
17	0.0538	1.366	21	0.0366	0.930
18	0.0478	1.214	22	0.0336	0.853
19	0.0418	1.062	23	0.0306	0.777
20	0.0359	0.912	24	0.0276	0.701
21	0.0329	0.836	25	0.0247	0.627
22	0.0299	0.759	26	0.0217	0.551
23	0.0269	0.660	27	0.0202	0.513
24	0.0239	0.607	28	0.0187	0.475
25	0.0209	0.531	29	0.0172	0.437
26	0.0179	0.455	30	0.0157	0.399
27	0.0164	0.417	31	0.0142	0.361
28	0.0149	0.378	32	0.0134	0.340

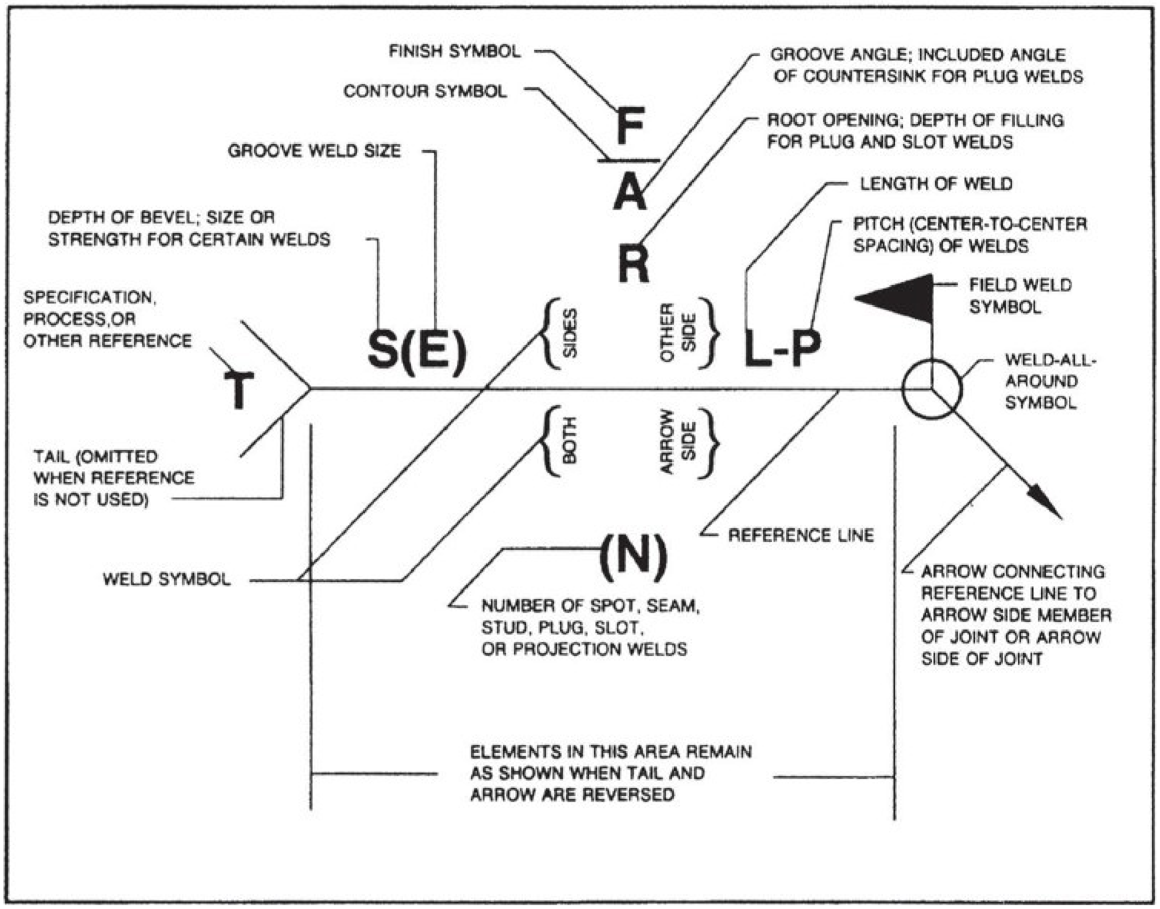
This table is for information only. This product is commonly specified to decimal thickness, not to gage number.

[1]From Mouser, *Welding Codes, Standards, and Specifications, pp. 309–374.*

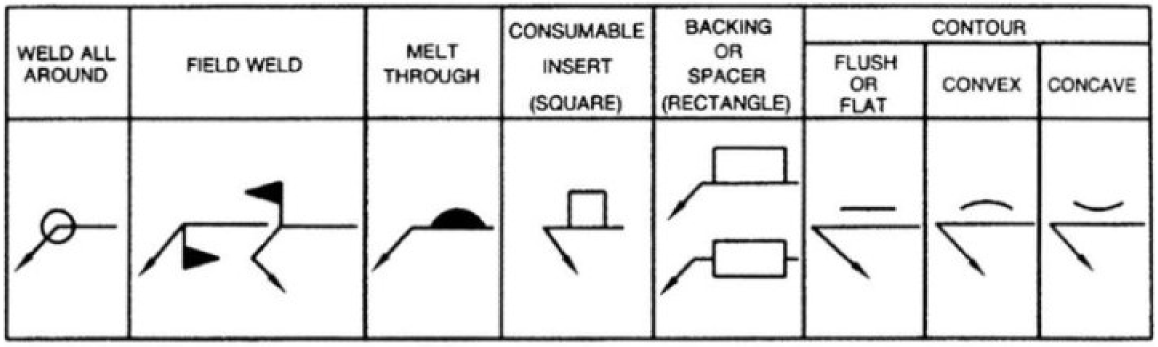
APPENDIX B
WELDING SYMBOLS

Welding Symbols

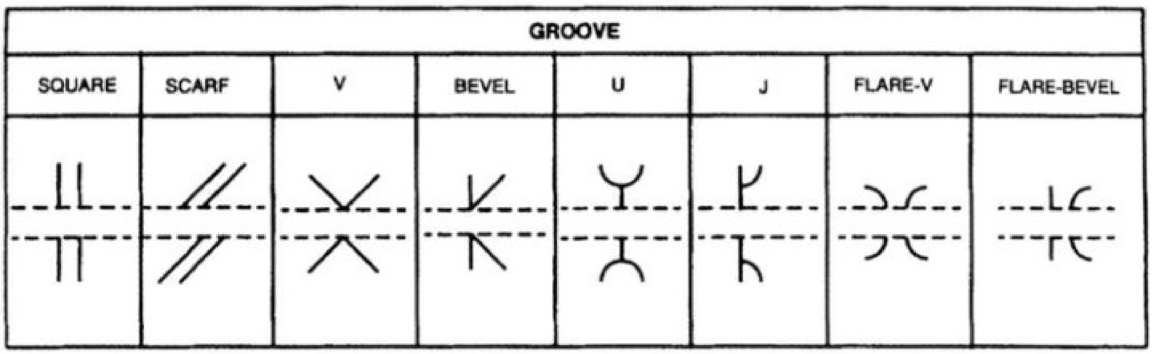
Standard Location of Elements of a Welding Symbol

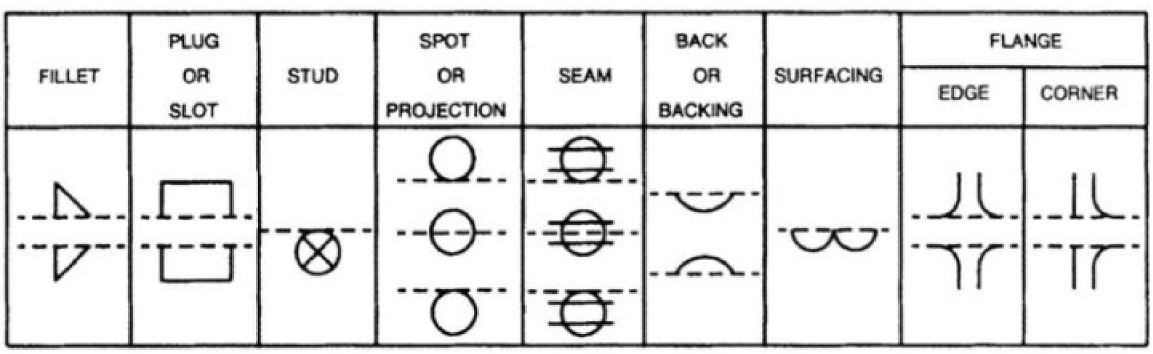


Supplementary Symbols



Welding Symbols





NOTE: THE REFERENCE LINE IS SHOWN DASHED FOR ILLUSTRATIVE PURPOSES.

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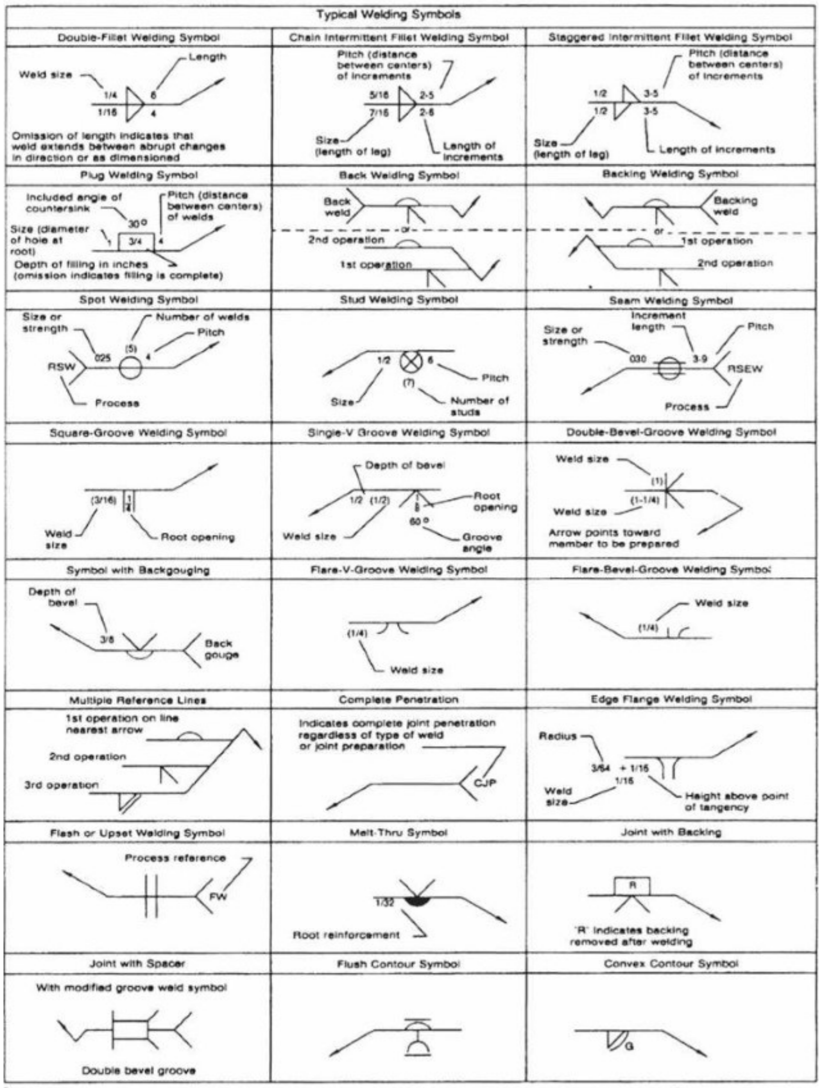

* It should be understood that these charts are intended only as shop aids. The only complete and official presentation of the standard welding symbols is in A2.4

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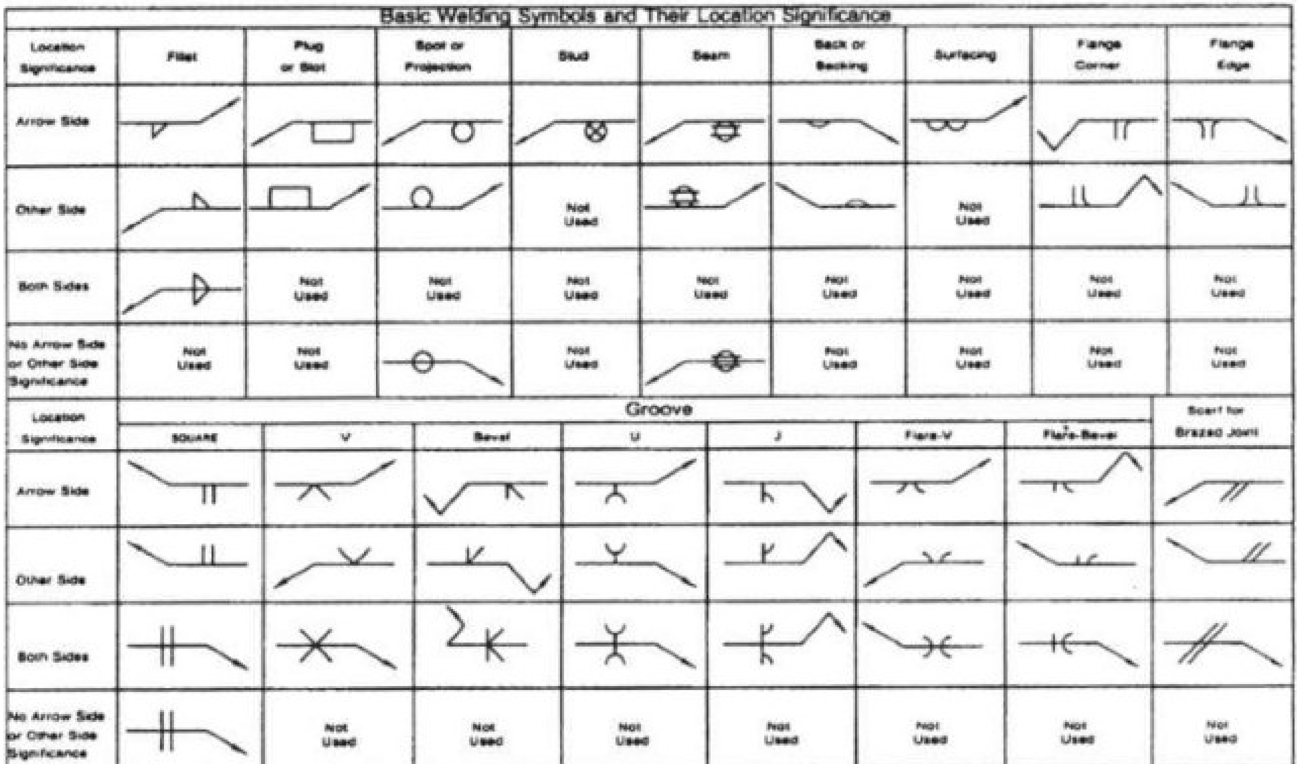
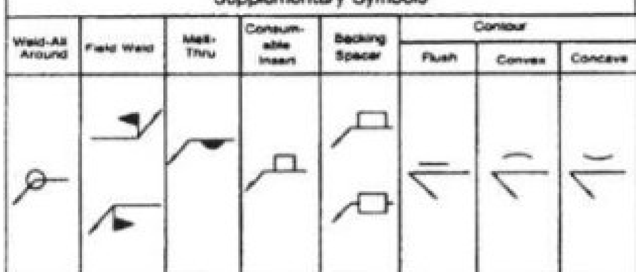
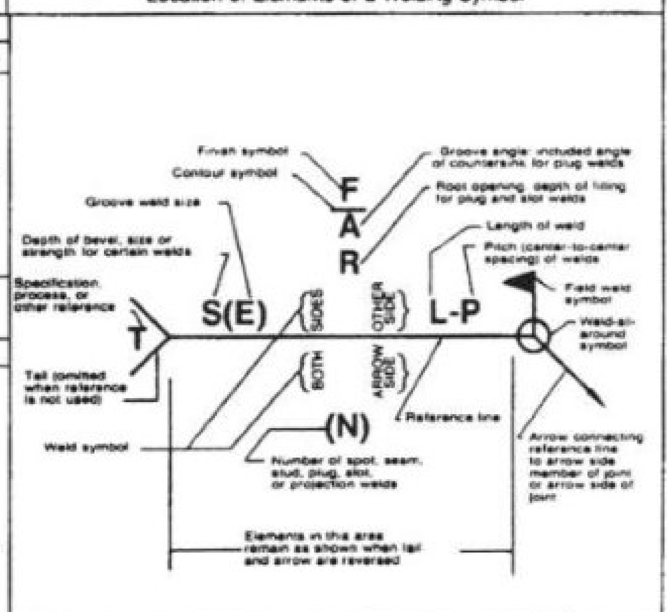
Basic Welding Symbols and Their Location Significance



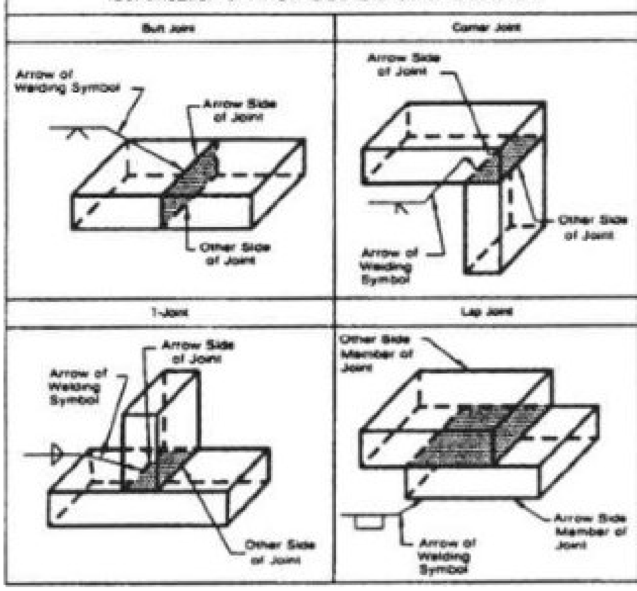
Supplementary Symbols

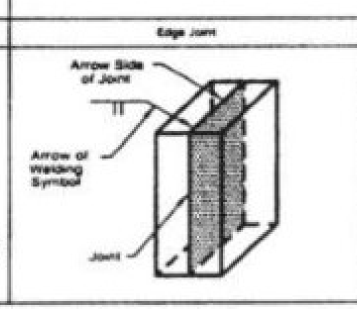


Location of Elements of a Welding Symbol



Basic Joints
Identification of Arrow Side and Other Side Joint





Process Abbreviations

Where process abbreviations are to be included in the tail of the welding symbol, reference is made to Table 1, Designation of Welding and Allied Processes by Letters, of ANSI/AWS A2.4-93

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APPENDIX C

SI METRIC CONVERSION TABLE[1]

Some Conversion Factors, between U.S. Customary and SI Metric Units, Useful in Structural-Steel Design

	To convert	To	Multiply by
Forces	kip force	kN	4.448
	lb	N	4.448
	kN	kip	0.2248
Stresses	ksi	MPa (i.e., N/mm^2)	6.895
	psi	MPa	0.006895
	MPa	ksi	0.1450
	MPa	psi	145.0
Moments	ft-kip	$kN \cdot m$	1.356
	$kN \cdot m$	ft-kip	0.7376
Uniform loading	kip/ft	kN/m	14.59
	kN/m	kip/ft	0.06852
	kip/ft^2	kN/m^2	47.88
	psf	N/m^2	47.88
	kN/m^2	kip/ft^2	0.02089

Note: For proper use of SI, see "Standard for Metric Practice (ASTM E380)," American Society for Testing and Materials, Philadelphia. Also see "Standard Practice for the Use of Metric (SI) Units in Building Design and Construction" (Committee E-6 Supplement to E380) (ANSI/ASTM E621), American Society for Testing and Materials, Philadelphia.

Basis of conversions (ASTM E380): 1 in = 25.4 mm; 1 lb force = 4.448 221 615 260 5 N.

Basic SI Units Relating to Structural Steel Design

Quantity	Unit	Symbol
Length	Meter	m
Mass	Kilogram	kg
Time	Second	s

Derived SI Units Relating to Structural Steel Design

Quantity	Unit	Symbol	Formula
Force	Newton	N	$kg \cdot m/s^2$
Pressure, stress	Pascal	Pa	N/m^2
Energy, or work	Joule	J	$N \cdot m$

[1] From *Steel Design Handbook: LRFD Method*.

APPENDIX D

NOMENCLATURE

A	cross-sectional area, in^2
A_B	loaded area of concrete, in^2
A_b	nominal body area of a fastener, in^2
A_c	area of concrete, in^2
A_c	area of concrete slab within effective width, in^2
A_D	area of an upset rod based on the major diameter of its threads, in^2
A_e	effective net area, in^2
A_f	area of flange, in^2
A_{fe}	effective tension flange area, in^2
A_{fg}	gross area of flange, in^2
A_{fn}	net area of flange, in^2
A_g	gross area, in^2
A_{gt}	gross area subject to tension, in^2
A_{gv}	gross area subject to shear, in^2
A_n	net area, in^2
A_{nt}	net area subject to tension, in^2
A_{nv}	net area subject to shear, in^2
A_{pb}	projected bearing area, in^2
A_r	area of reinforcing bars, in^2
A_s	area of steel cross section, in^2
A_{sc}	cross-sectional area of stud shear connector, in^2
A_{sf}	shear area on the failure path, in^2
A_w	web area, in^2
A_1	area of steel bearing concentrically on a concrete support, in^2
A_2	total cross-sectional area of a concrete support, in^2
B	factor for bending stress in tees and double angles
B	factor for bending stress in web-tapered members
B_1, B_2	factors used in determining M_u for combined bending and axial forces when first-order analysis is employed
C_{PG}	plate-girder coefficient
C_b	bending coefficient dependent on moment gradient

C_m	coefficient applied to bending term in interaction formula for prismatic members and dependent on column curvature caused by applied moments
C'_m	coefficient applied to bending term in interaction formula for tapered members and dependent on axial stress at the small end of the member
C_p	ponding flexibility coefficient for primary member in a flat roof
C_s	ponding flexibility coefficient for secondary member in a flat roof
C_v	ratio of "critical" web stress, according to linear buckling theory, to the shear yield stress of web material
C_w	warping constant, in^2
D	outside diameter of circular hollow section, in
D	dead load due to the weight of the structural elements and permanent features on the structure
D	factor dependent on the type of transverse stiffeners used in a plate girder
E	modulus of elasticity of steel ($E = 29{,}000$ ksi)
E	earthquake load
E_c	modulus of elasticity of concrete, ksi
E_m	modified modulus of elasticity, ksi
F_{BM}	nominal strength of the base material to be welded, ksi
F_{by}	flexural stress for tapered members
F_{cr}	critical stress, ksi
F_{crft}, F_{cry}, F_{crz}	flexural-torsional buckling stresses for double-angle and tee-shaped compression members, ksi
F_e	elastic buckling stress, ksi
F_{ex}	elastic flexural buckling stress about the major axis, ksi

F_{EXX}	classification number of weld metal (minimum specified strength), ksi
F_{ey}	elastic flexural buckling stress about the minor axis, ksi
F_{ez}	elastic torsional buckling stress, ksi
F_L	smaller of $(F_{yf}\ F_r)$ or F_{yw}, ksi
F_{my}	modified yield stress for composite columns, ksi
F_n	nominal shear rupture strength, ksi
F_r	compressive residual stress in flange (10 ksi for rolled, 16.5 ksi for welded), ksi
F_{sy}	stress for tapered members, ksi
F_u	specified minimum tensile strength of the type of steel being used, ksi
F_w	nominal strength of the weld electrode material, ksi
F_{wy}	stress for tapered members, ksi
F_y	specified minimum yield stress of the type of steel being used, ksi; *yield stress* denotes either the specified minimum yield point (for those steels that have a yield point) or specified yield strength (for those steels that do not have a yield point)
F_{yf}	specified minimum yield stress of the flange, ksi
F_{yr}	specified minimum yield stress of reinforcing bars, ksi
F_{yst}	specified minimum yield stress of the stiffener materials, ksi
F_{yw}	specified minimum yield stress of the web, ksi
G	shear modulus of elasticity of steel, ksi ($G = 11{,}200$)
H	horizontal force, kips
H	flexural constant
H_s	length of stud connector after welding, in
I	moment of inertia, in^4
I_d	moment of inertia of the steel deck supported on secondary members, in^4
I_p	moment of inertia of primary members, in^4
I_s	moment of inertia of secondary members, in^4
I_{st}	moment of inertia of a transverse stiffener, in^4
I_{yc}	moment of inertia about y axis referred to compression flange, or if reverse curvature bending referred to smaller flange, in^4
J	torsional constant for a section, in^4
K	effective length factor for prismatic member
K_z	effective length factor for torsional buckling
K_γ	effective length factor for a tapered member
L	story height, in
L	length of connection in the direction of loading, in
L	live load due to occupancy and movable equipment
L_b	laterally unbraced length; length between points which are either braced against lateral displacement of compression flange or braced against twist of the cross section, in
L_c	length of channel shear connector, in

L_e edge distance, in

L_p limiting laterally unbraced length for full plastic bending capacity, uniform moment case ($C_b = 1.0$), in

L_p column spacing in direction of girder, ft

L_{pd} limiting laterally unbraced length for plastic analysis, in

L_r limiting laterally unbraced length for inelastic lateral-torsional buckling, in

L_r roof live load

L_s column spacing perpendicular to direction of girder, ft

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

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

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

M_{cr} elastic buckling moment, kip-in

M_{lt} required flexural strength in member due to lateral frame translation only, kip-in

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

M_n nominal flexural strength, kip-in

M_{nt} required flexural strength in member assuming there is no lateral translation of the frame, kip-in

M'_{nx}, M_{ny} flexural strength for use in alternate interaction equations for combined bending and axial force, kip-in or kip-ft as indicated

M_p plastic bending moment, kip-in

M'_p moment for use in alternate interaction equations for combined bending and axial force, kip-in

M_r	limiting buckling moment, M_{cr}, when $\lambda = \lambda_r$ and $C_b = 1.0$, kip-in
M_u	required flexural strength, kip-in
M_y	moment corresponding to onset of yielding at the extreme fiber from an elastic stress distribution ($= F_y S$ for homogeneous sections), kip-in
M_1	smaller moment at end of unbraced length of beam or beam-column, kip-in
M_2	larger moment at end of unbraced length of beam or beam-column, kip-in
N	length of bearing, in
N_r	number of stud connectors in one rib at a beam intersection
P_{e1}, P_{e2}	elastic Euler buckling load for braced and unbraced frame, respectively, kips
P_n	nominal axial strength (tension or compression), kips
P_p	bearing load on concrete, kips
P_u	required axial strength (tension or compression), kips
P_y	yield strength, kips
Q	full reduction factor for slender compression elements
Q_a	reduction factor for slender stiffened compression elements
Q_n	nominal strength of one stud shear connector, kips
Q_s	reduction factor for slender unstiffened compression elements
R	load due to initial rainwater or ice exclusive of the ponding contribution
R_{PG}	plate girder bending strength reduction factor
R_e	hybrid girder factor
R_n	nominal strength
R_v	web shear strength, kips
S	elastic section modulus, in^3
S	spacing of secondary members, ft
S	snow load

S_{eff}	effective section modulus about major axis, in^3
S'_x	elastic section modulus of larger end of tapered member about its major axis, in^3
S_{xt}, S_{xc}	elastic section modulus referred to tension and compression flanges, respectively, in^3
T	tension force due to service loads, kips
T_b	specified pretension load in high-strength bolt, kips
T_u	required tensile strength due to factored loads, kips
U	reduction coefficient, used in calculating effective net area
V_n	nominal shear strength, kips
V_u	required shear strength, kips
W	wind load
X_1	beam buckling factor
X_2	beam buckling factor
Z	plastic section modulus, in^3
a	clear distance between transverse stiffeners, in
a	distance between connectors in a built-up member, in
a	shortest distance from edge of pin hole to edge of member measured parallel to direction of force, in
a_r	ratio of web area to compression flange area
a'	weld length, in
b	compression element width, in
b_e	reduced effective width for slender compression elements, in
b_{eff}	effective edge distance, in
b_f	flange width, in
c_1, c_2, c_3	numerical coefficients
d	nominal fastener diameter, in
d	overall depth of member, in
d	pin diameter, in
d	roller diameter, in
d_b	beam depth, in
d_c	column depth, in

d_L	depth at larger end of unbraced tapered segment, in
d_o	depth at smaller end of unbraced tapered segment, in
e	base of natural logarithm $= 2.71828\ldots$
f	computed compressive stress in the stiffened element, ksi
f_{b1}	smallest computed bending stress at one end of a tapered segment, ksi
f_{b2}	largest computed bending stress at one end of a tapered segment, ksi
f'_c	specified compressive strength of concrete, ksi
f_o	stress due to $1.2D + 1.2R$, ksi
f_{un}	required normal stress, ksi
f_{uv}	required shear stress, ksi
f_v	required shear stress due to factored loads in bolts or rivets, ksi
g	transverse center-to-center spacing (gage) between fastener gage line, in
h	clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, in
h	distance between centroids of individual components perpendicular to the member axis of buckling, in
h_c	twice the distance from the centroid to the following: the inside face of the compression flange less the fillet or corner radius, for rolled shapes; the nearest line of fasteners at the compression flange or the inside faces of the compression flange when welds are used, for built-up sections, in
h_r	nominal rib height, in
h_s	factor used for web-tapered members
h_w	factor used for web-tapered members
j	factor for minimum moment of inertia for a transverse stiffener
k	distance from outer face of flange to web toe of fillet, in
k_v	web plate buckling coefficient

l	laterally unbraced length of member at the point of load, in
l	length of bearing, in
l	length of connection in the direction of loading, in
l	length of weld, in
m	ratio of web to flange yield stress or critical stress in hybrid beams
r	governing radius of gyration, in
r_i	minimum radius of gyration of individual component in a built-up member, in
r_{ib}	radius of gyration of individual component relative to centroidal axis parallel to member axis of buckling, in
r_m	radius of gyration of the steel shape, pipe, or tubing in composite columns; for steel shapes it may not be less than 0.3 times the overall thickness of the composite section, in
$\bar{r}_o$	polar radius of gyration about the shear center, in
r_{ox}, r_{oy}	radius of gyration about x and y axes at the smaller end of a tapered member, respectively, in
r_{To}	for the smaller end of a tapered member, the radius of gyration, considering only the compression flange plus one-third of the compression web area, taken about an axis in the plane of the web, in
r_x, r_y	radius of gyration about x and y axes, respectively, in
r_{yc}	radius of gyration about y axis referred to compression flange, or if reverse curvature bending, referred to smaller flange, in
s	longitudinal center-to-center spacing (pitch) of any two consecutive holes
t	thickness of connected part, in
t_f	flange thickness, in
t_f	flange thickness of channel shear connector, in
t_w	web thickness of channel shear connector, in

t_w	web thickness, in
w	plate width; distance between welds, in
w	unit weight of concrete, lb/ft^3
w_r	average width of concrete rib or haunch, in
x	subscript relating symbol to strong-axis bending
x_o, y_o	coordinates of the shear center with respect to the centroid, inh
$\bar{x}$	connection eccentricity, in
y	subscript relating symbol to weak-axis bending
z	distance from the smaller end of tapered member used, for the variation in depth, in
α	separation ratio for built-up compression members $= h/2r_{ib}$
Δ_{oh}	translation deflection of the story under consideration, in
γ	depth tapering ratio; subscript for tapered members
ζ	exponent for alternate beam-column interaction equation
η	exponent for alternate beam-column interaction equation
λ_c	column slenderness parameters
λ_e	equivalent slenderness parameter
λ_{eff}	effective slenderness ratio
λ_p	limiting slenderness parameter for compact element
λ_r	limiting slenderness parameter for noncompact element
ϕ	resistance factor
ϕ_b	resistance factor for flexure
ϕ_c	resistance factor for compression
ϕ_c	resistance factor for axially loaded composite columns
ϕ_{sf}	resistance factor for shear on the failure path
ϕ_t	resistance factor for tension
ϕ_v	resistance factor for shear

INDEX

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