PDHonline Course S219 (4 PDH)

Steel Connections

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Steel Connections

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COURSE CONTENT

Steel Framing Connections

Introduction

Unless you go to work for a steel fabricator or firm that specializes in the design of connections it is unlikely that you will be involved with the detail design of the framing connections associated with the building that you are the structural engineer for. Never less it is important to have a very good understanding of the design of steel framing connections so that the assumptions that are made as a part of the design and detailing of the structure take into account simplified, practical and subsequently economical solutions to the design and detailing of the connections that will be accomplished by the steel fabricator. In addition, it is important for the EOR to have a clear understanding of the design of steel connections because most major structural failures that have occurred in the recent past were related to some kind of connection deficiency or failure. Please see a discussion of the AISC Code of Standard Practice at the conclusion of this Lecture concerning the delegation of the design of connections to the fabricator by the EOR.

Part 7 (Design Considerations for Bolts), Part 8 (Design Considerations for Welds), Part 9 (Design of Connecting Elements), Part 10 (Design of Simple Shear Connections), Part 11 (Design of Flexible Moment Connections - FMC), Part 12 (Design of Fully Restrained – FR – Moment Connections), Part 13 (Design of Bracing Connections and Truss Connections), Part 14 (Design of Beam Bearing Plates, Column Base Plates, Anchor Rods and Column Splices) and Part 15 (Design of Hanger Connections, Bracket Plates and Crane-Rail Connections) of the AISC 13th Edition all provide guidelines, design requirements, detailing recommendations and connection load tables for the related referenced types of connections. In addition AISC also published a Manual of Steel Construction, Volume II, Connections for ASD (9th Edition) and LRFD (1st Edition) in 1992 that provides more detail information concerning the design of steel connections.

Methods of Joining

Steel connections and the joining of steel members are accomplished using bolts or welding. It is preferable to perform welding of a connection in the fabrication shop where the environment and procedures are more easily controlled than in the field. Bolting requires either punching or drilling of holes through all of the connected plies of materials that are to be joined. Bolt hole sizes may be standard, oversized, short slotted or long slotted depending on the requirements of the connection. It is also common for one ply of material to have a standard hole while another ply of the connection is prepared with a slotted hole. Welding eliminates the need for punching or drilling the piles of connected materials, however, the labor associated with welding requires a greater skill than that required to install bolts, therefore labor costs are greater for welding than bolting.

Bolting

Bolting of structural steel evolved from riveted connections. Riveting became obsolete as the cost of installed high strength structural bolts became competitive with the costs associated with the typical 4 to 5 man skilled riveting crew. The AISC Specification for Structural Steel Buildings and the RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts are the current governing publications for structural steel high strength bolts.

The most commonly used high strength bolts are covered under the ASTM A325, A490, F1852 and F2280 specifications. There are other high strength bolts that are available for unique applications. These bolts include;

ASTM A193: For use in elevated temperatures.

ASTM A320: For use in low temperatures.

ASTM A354 Grade BD: For large diameter bolts with properties similar to A490.

ASTM A449: For large diameter bolts with properties similar to A325.

A325 bolts have a minimum tensile strength of 125 ksi and are permitted to be galvanized. A490 bolts have a minimum tensile strength of 150 ksi but are not permitted to be galvanized because of the potential for hydrogen embrittlement. A325 and A490 high strength bolts are available in sizes from ½ to 1½ inch diameters in 1/8 inch increments and can be ordered in lengths from 1½ to 8 inches in ¼ inch increments.

There are three types of joints that can be used for bolted connections; Snug-Tightened, Pretensioned and Slip-Critical. Table 1 illustrates some general guidelines for these three joint types. A snug-tightened bolted connection means that the joined plies bear directly on the bolts, or in other words the shank of the bolt is how the load transfer from one ply to the next occurs. Bearing connections of this type can be specified with either threads included (N) or excluded (X) from the shear plane. Allowing the threads to be included in the shear plane of a connection results in shear strengths about 25% less than if the threads are excluded from the shear planes. Specifying X bolts means that the length of the bolt has to also be clearly specified correctly in order to assure that the threads are in fact excluded from the shear planes of the connection.

In a pretensioned connection the bolts act like clamps to hold the plies of materials together. The clamping force occurs as a result of the pretensioning force in the bolt, however the actual load transfer occurs through the bearing of the plies against the bolt just as is the case in a snug-tightened connection.

Joint Type		
Snug-Tightened	Pretensioned	Slip-Critical
Applicable except when a pretensioned or slip-critical joint is required	As required by the AISC Specification *(see below)*	Joints that are subject to fatigue load with reversal of loading direction
	Joint subjected to significant load reversal	Joints with oversize holes
	Joint subjected to fatigue with no reversal of loading direction	Joints with slotted holes, except those with applied load approximately normal to the dimension of the long slot
	Joint with A325 or F1852 subjected to tensile fatigue	Joints in which slip of the joint would be detrimental to the performance of the structure
	Joints with A490 (or F2280) bolts with tension (tension or combined shear and tension, with or without fatigue)	
	Seismic connections in lateral force resisting systems when *R* is taken >3	

The AISC Specification requires that joints be pretensioned for the following circumstances:

- ➔ Column Splices in multi-story structures over 125 ft
- ➔ Connections of all girders and beams to columns and any other beams and girders on which the bracing of the columns is dependent in structures over 125 ft
- ➔ Connections in structures that support cranes with a capacity over 5 tons
- ➔ Connections for the support of machinery and other sources of impact or stress reversal
- ➔ End connections for built-up compression members (fabricated with Class A or B faying surfaces)

TABLE 1

For slip-critical joints, the bolts are pretensioned and the faying (i.e. the contact surfaces of the plies of material to be joined together) surfaces are prepared to achieve maximum slip resistance. This reliance on friction between the connected plies for load transfer means that the strength of a slip-critical connection is directly proportional to the slip coefficient (μ). Coatings such as paint or galvanizing tend to reduce the slip coefficient. The RCSC Specification defines three types of faying surfaces with different values of μ which are;

Class A: $\mu = 0.33$
Class B: $\mu = 0.50$
Class C: $\mu = 0.40$

Coatings classified as Class A (except for the case of clean mill scale with no coating) and Class B (except for the case of blast cleaned surfaces with no coating) must have their slip coefficient determined by the *Testing Method to Determine the Slip Coefficient Used in Bolted Joints* provided in Appendix A of the RCSC Specification. Class C surfaces apply to hot dipped galvanized and other similar roughened surfaces and does not require testing.

In general the type of bolt hole selected for a joint should be based on constructability. Standard and short slotted holes can be used for all three of the joint types described above. Long slotted holes are permitted in all three of the joint types also with the approval of the EOR. Oversized holes can be used only in slip-critical joints. OSHA requires that a minimum of two bolts be provided in all connections and that the bolts must remain in place after the member has been released from the crane during erection.
When designing bolted connections it is also good practice to limit the number of different bolt strengths and diameters that are used for the entire project. It is also recommended that different diameters of bolts be specified when different bolt strengths are required. Both of the above practices help to minimize installation errors in the field.

Washers are required for all three joint types that have sloped surfaces or use slotted holes in the outer ply. For pretensioned and slip-critical joints washers are also required for the following types of connections:

- When using A490 bolts and the connection material is less than 40 ksi.
- Under the turned element when using the calibrated wrench pretensioning method.
- Under the nut when the twist-off type of tension control bolt pretension method is used in certain bolt configurations (see Section 6.2.4 and Figure C-8.1 of the RCSC Specification).
- When the direct-tension indicator pretensioning method is used.
- When oversized holes are used in the outer ply.

When a pretensioned joint is required there are four different acceptable methods of confirming that the proper pretensioning force has been achieved. These four methods are; Turn-of-Nut, Calibrated Wrench, Twist-Off Bolt (also referred to as Tension Control Bolts) and Direct Tension Indicator. Table 2 provides a description of these four methods as well as the required installation criteria for a snug-tightened bolt.

Joint Type	Installation Methods
Snug-Tightened	A few impacts with an impact wrench or the full effort of an ironworker using an ordinary spud wrench to bring the plies into firm contact.
Pretensioned and Slip-Critical	**Turn-of-Nut**: Installed first as snug-tight, then additional rotation of the nut or head is applied ranging from 1/3 to 1 turn based on grip geometry and bolt aspect ratio.
	Calibrated Wrench: Installed first as snug-tight. Subsequently, a predetermined torque (calibrated daily to provide the required pretension) is applied with a wrench that indicates the torque applied to the nut or head. The nut or head does not need to be rotated greater than specified limits (for turn-of-nut method).
	Twist-Off-Type Tension-Control Bolt: Bolts meeting F1852 or F2280 are used and installed as an assembly. Installed first as snug-tight without severing the spline. Subsequently, all bolts are pretensioned with the twist-off-type tension-control bolt installation wrench.
	Direct Tension Indicator: Direction tension indicators must meet ASTM F959. Bolts (with washers) are first installed as snug-tight, making sure the gap is not less than the job installation gap. Subsequently, the bolts are pretensioned until the direct tension indicator protrusions have been compressed and the gap is less than the job inspection gap.
	If alternative design fastener or alternative washer-type indicating devices are used, install per manufacturer (and as approved by the Engineer of Record).

TABLE 2

The following links are to manufacturers of direct tension indicators:

Applied Bolting Technology: http://www.appliedbolting.com/

Turnasure, LLC: http://www.turnasure.com/

The following links are to manufacturers of twist-off bolts:

LeJuene Bolt Company: http://www.lejeunebolt.com/

Baco Enterprises, Inc.: http://www.bacoent.com/tension.html

See *Banging Bolts – Another Perspective*, Modern Steel Construction, November 1999, for a discussion of the phenomenon of what happens when pretensioned bolted connections slip.

It is also important that the design engineer be familiar with the inspection requirements associated with the RCSC Specifications. For pretensioned and slip-critical joints the RCSC Specifications require the following inspection procedures:

- When using the turn-of-nut method it is necessary to observe that the bolting crew properly rotates the turned element relative to the unturned element the specified amount.
- When the calibrated wrench method is used pre-installation calibration of the wrench must be performed daily. In addition, confirmation that the bolting crew properly applies the calibrated wrench to the turned element must be observed.
- When the twist-off type tension indicator is used it is necessary to observe that the splined ends are properly severed during the installation by the bolting crew.
- When the direct tension indicator method is used the inspector shall observe the pre-installation verification testing. Furthermore, prior to pretensioning confirmation that the appropriate feeler gage is accepted in at least half of the spaces between the protrusions of the direct tension indicator and that the protrusions are properly oriented away from the work must be observed. After pretensioning it is also necessary to verify that the appropriate feeler gage is refused entry into at least half of the spaces between the protrusions of the indicator.

Welding:

Welding is the process of fusing multiple pieces of metal together by heating the metal to a liquid state. Welding can often simplify an over wise complicated bolted connection. Welding should be performed on bare metal, therefore all paint and galvanizing should be removed from the metal that is to be welded. Guidelines for welded construction are published by the American Welding Society (AWS) in the AWS D1.1 Structural Welding Code. The provisions of the AWS Code have been adopted by the AISC Specifications. It is also recommended that every young engineer coming into the industry have his or her own copy of the *Design of Welded Structures* by Omar W. Blodgett.

Several different welding processes are available for joining structural steel. The selection of the appropriate process is typically determined by constructability issues more so than strength requirements. The most common weld processes are Shielded Metal Arc Welding (SMAW), Gas Metal Arc Welding (GMAW), Flux Core Arc Welding (FCAW) and Submerged Arc Welding (SAW). SMAW uses an electrode coated with a material that vaporizes and shields the weld metal to prevent oxidation. The coated electrode is consumable and can be deposited in any position. SMAW is commonly referred to as stick welding.

GMAW and FCAW are similar weld processes that use a wire electrode that is fed by a coil to a gun shaped electrode holder. The main difference between the two processes is the method of shielding the weld. GMAW uses an externally supplied gas mixture while FCAW has a hollow electrode with flux material in the core that generates a gas shield or flux shield when the weld is made. GMAW and FCAW can be deposited in all positions and have a relatively fast deposit rate compared to the other processes.

The SAW process uses a consumable electrode that is submerged below a blanket of granular flux. The flux protects and enhances the resulting weld. SAW tends to produce high quality welds that are both strong and ductile. The major limitation of this process is that the weld can only be deposited in a flat position due to the nature of the granular flux used. The SAW process is most often used in automated, controlled shop welding operations.

From a structural design standpoint it is important to recognize that the effective throat dimension for a weld installed using the SAW process is calculated differently than the other processes. Since the SAW process produces higher quality welds with deeper penetration the effective throat is permitted to be equal to the weld throat size if the weld is less than 3/8 inch. For larger welds the effective throat for SAW welds is the minimum distance from the root to the face of the weld plus 0.11 inch. For the other welding processes the effective throat is taken as the minimum distance from the root to the face of the weld or 0.707 x the leg length for equal leg welds (see Figure 38).

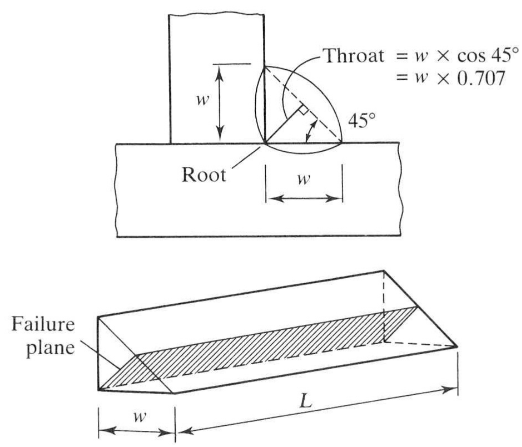


FIGURE 38

There are four types of welds; Fillet, Groove, Plug and Slot. Fillet and groove welds make up the majority of all structural connection welds, therefore only these types of welds will be discussed. There are five types of structural joints that can be made using either fillet or groove welds and include; Butt, Lap, Tee, Corner and Edge (see Figure 39). The welds for these same joints can be executed in any of four positions depending on the configuration and location of the joint and include; Flat, Horizontal, Vertical and Overhead.

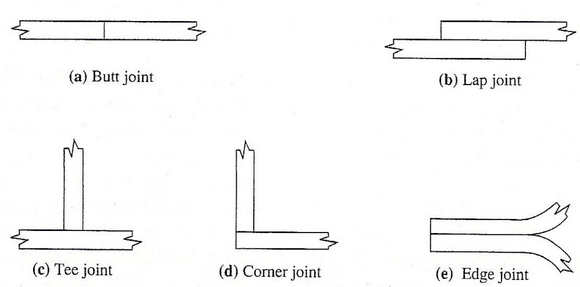


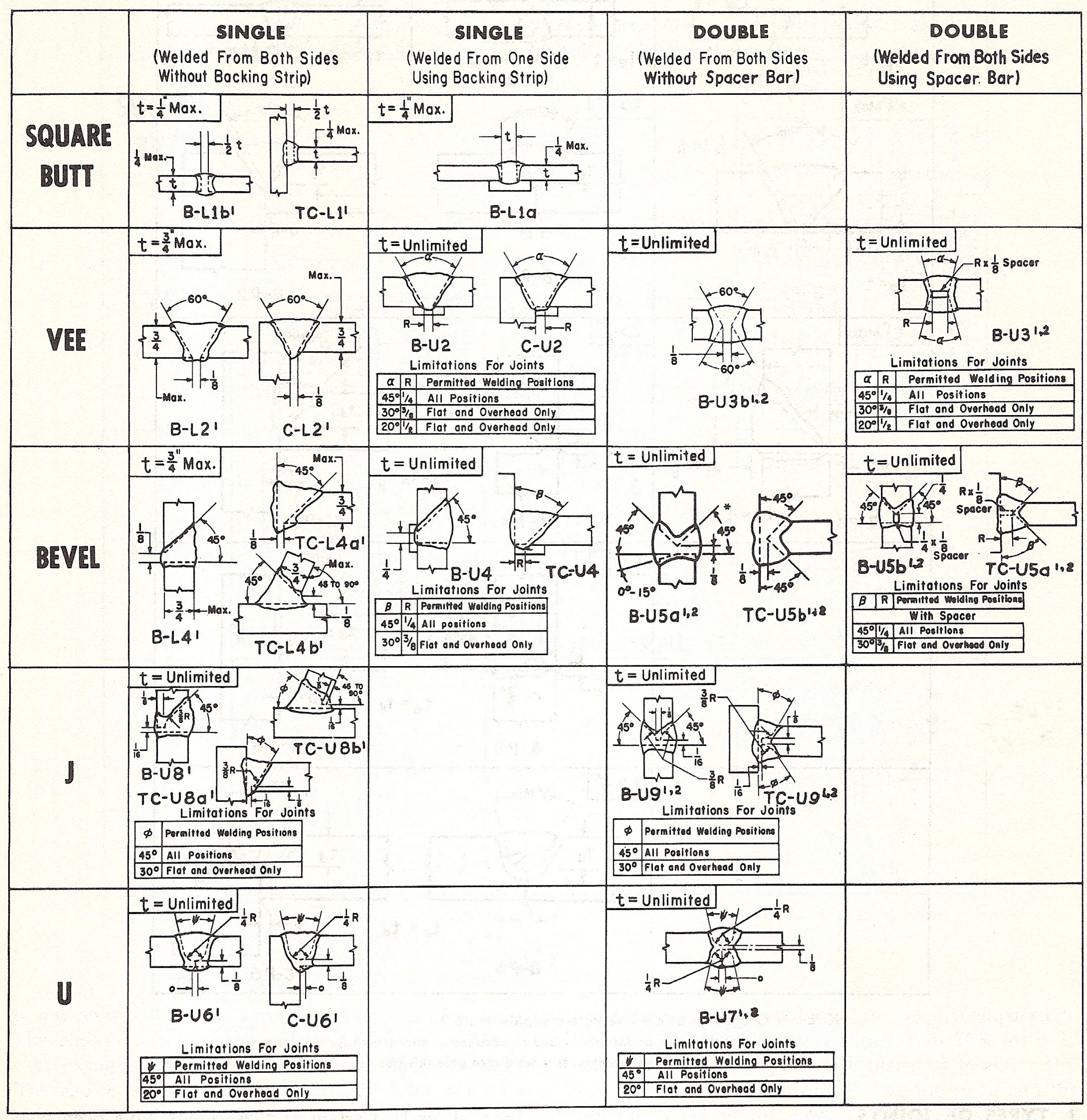
FIGURE 39

Fillet welds are by far the most common type of weld used in construction. This is because fillet welds are easy to fabricate and require very simple preparation of the material to be joined, and as a result are very economical. The strength of a fillet weld is directly proportional to it's length and throat dimension as indicated in Figure 38. Fillet welds that can be completed in one pass provide an additional economical benefit of minimizing primarily labor costs. The maximum size of fillet weld that can be deposited in one pass in the horizontal or flat position is normally 5/16 inch.

Groove welds are typically used when the plies of the materials to be joined are aligned parallel and lie in the same plane, however, it is also common to see groove welds in moment connections where the materials to be joined are perpendicular to each other and the ply that terminates at the intersection must develop the full cross sectional area of the element. Groove welds that extend through the full thickness of the materials joined are called Complete-Joint-Penetration welds or more commonly Full-Penetration welds. Groove welds that do not extend completely through the thickness of the joined materials are called Partial-Penetration welds. Groove weld, particularly full-penetration welds, are expensive for the following reasons:

1. The metal pieces that are being connected must have the joined edges prepared or shaped correctly in order to allow for proper penetration of the weld to occur over the required cross section of the connected materials.
2. It is common for backing and extension bars, and runoff tabs to have to be added around the joint to help contain the weld metal during the welding process.
3. If the joint is to be loaded cyclically all of the backing and extension bars, and runoff tabs must be removed and the joint surfaces must be finished or ground smooth.
4. When beam flanges are connected small weld access holes must be cut in the web just below and above the top and bottom flanges, respectively, to allow access to make the weld and to allow placement of the backing bar.

Examples of prequalified full-penetration and partial-penetration welds are illustrated in Figures 40 and 41, respectively.

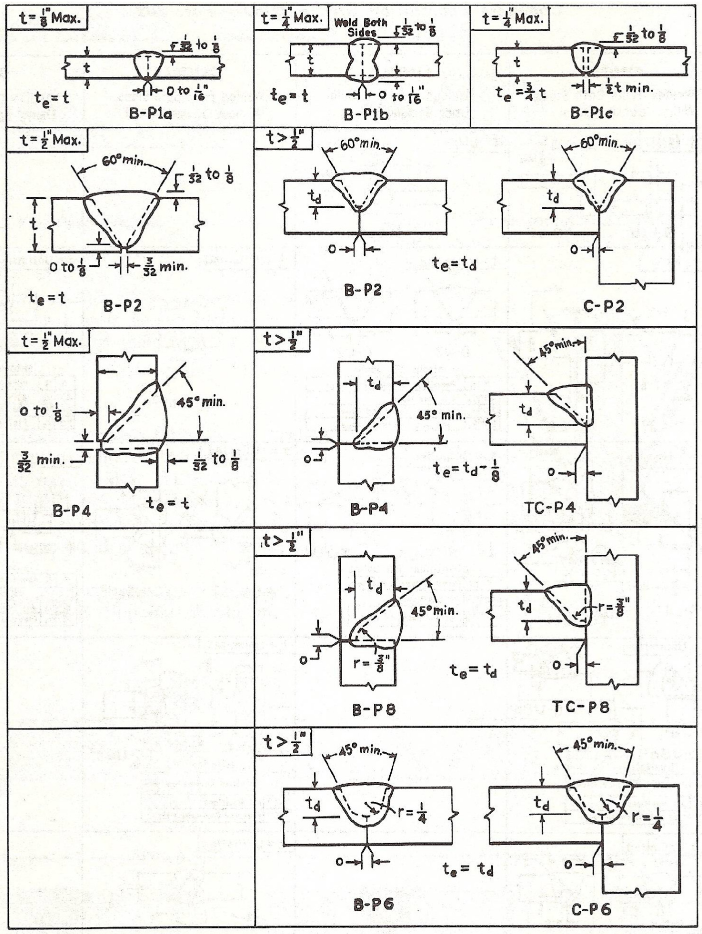


NOTE: The size of the fillet weld reinforcing groove welds in Tee and corner joints shall equal t/4 but shall be 3/8" max.

1. Gouge root before welding second side (Par 505l)
2. Use of this weld preferably limited to base metal thickness of 5/8" or larger.

* When lower plate is bevelled, first weld root pass this side.

FIGURE 40



NOTE: 1. Gouge root before welding second side (Par 505I)
2. Use of this weld preferably limited to base metal thickness of 5/8" or larger.
* When lower plate is bevelled, first weld root pass this side.

FIGURE 41

Regardless of the process or type of weld, shear is always the controlling limit state for the design of a weld. The strength of a weld is based on either the shear strength of the weld or the shear strength of the base metal (based on its entire thickness). E70 is the most common electrode used in welding and the basis of the weld load tables in the AISC 13th Edition.

Simple Shear Connections

Simple shear connections are assumed to have little or no rotational resistance and are designed to carry only the shear component of the load as an idealized pin or roller, therefore no moment forces are assumed to be transmitted by this type of connection. However, it has been shown experimentally that shear connectors do posses some amount of rotational restraint. For design purposes, ignoring the rotational restraint produces a conservative result. The majority of the rotational flexibility of most types of shear connections is achieved via the deformation of the connection elements (i.e. plates, angles, tees, etc.) or through slotted or oversized holes. To assure this rotational ductility the shear connection elements are typically designed using thin and or mild yield strength materials (i.e. A36). Typical types of shear connections include;

- Double-Angle Connections: Double-angle connections are made by attaching the legs of two angles to the webs of the supporting and supported beams and can be bolted, welded or bolted-welded as shown in Figure 42.

- Shear End-Plate Connection: This connection involves welding a plate perpendicular to the end of the web of the supported beam and bolting or welding the plate to the supporting beam (see Figure 43).

- Unstiffened and Stiffened Seated Connection: For this type of connection the vertical leg of an angle is attached to the supporting member while the outstanding horizontal leg provides a "seat" upon which the supported beam rests (see Figure 44). The angle seat can be bolted or welded to either the supported or supporting beam or both. A stiffened seated connection involves placing a vertical stiffener plate centered under the horizontal leg of the angle in order to assist with the load transfer.

- Single-Plate (Shear Tab) Connection: A shear-tab consists of a plate welded to the supporting member and bolted to the web of the supported beam (see Figure 45). This type of connection can make erection of the supported beam quicker because it can simply be swung into place.

- Single Angle Connection: A single angle connection is similar to a double angle connection and can be bolted, welded or bolted-welded. This type of connection has similar erection efficiencies as was described for the shear-tab connection above.

- Tee Shear Connection: This type of connection is similar to a shear-tab in that the web of a WT or split T section is used to connect to the supported beam. The flange of the WT in turn is attached to the supporting member. As with all of the other connections, this type of joint can be bolted, welded or bolted-welded.

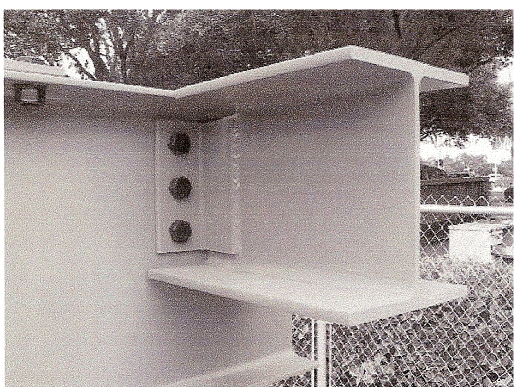


FIGURE 42

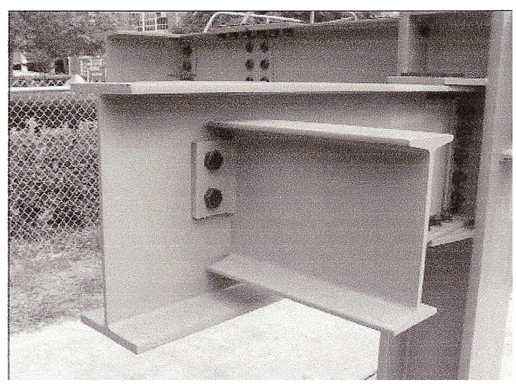


FIGURE 43

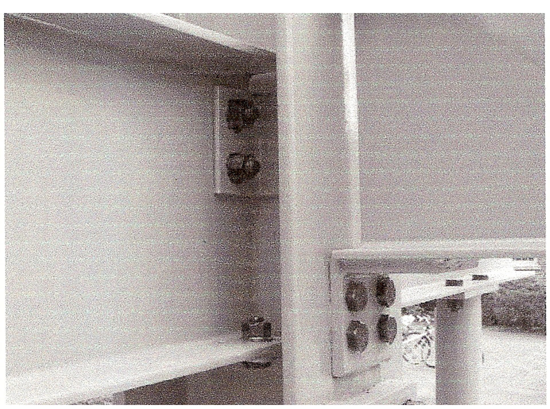


FIGURE 44

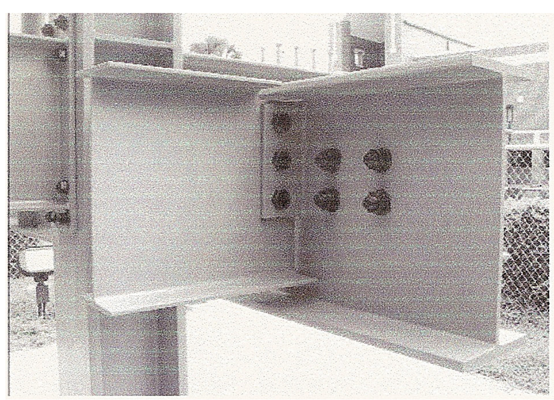


FIGURE 45

Moment Connections

Moment connections typically are used to transfer the moment carried by the flanges of the supported beam to the supporting member. Moment connections (or continuous or rigid frame connections) are assumed to exhibit little or no relative rotation between the supporting and supported members. A fully restrained (FR) connection assumes that the intersecting angles between the connected member are maintained (i.e. no relative rotation) and that there is full transfer of moment from one member to the other. Infinite rigidity can never be realistically attained, however, therefore even FR moment connections do exhibit some minimal amount of rotational flexibility. This flexibility is typically ignored, however, and the connection is idealized as having full fixity between the members. Partially restrained (PR) connections assume that there will be some relative rotational movement that occurs between the intersecting members, however, transfer of the moment between the connecting parts still occurs. Moment connections also normally include a simple shear connection at the webs of the supported member to enable transfer of the shear component to the supporting member. Typical types of moment connections include;

- Flange-Plated Connections (Beam-to-Column): These types of connections are made with top and bottom flange-plates that connect the flanges of the supported beam to a supporting column. Flange plates can be bolted or welded to the supported beam and can either be fillet or groove welded to the supporting column. The plates are typically shop attached to the column and field attached to the beam (see Figure 46).

- Directly Welded Flange Connections: These types of connections are typically made with full-penetration groove welds that directly connect the beam flanges to the supporting member (see Figure 47).

- Extended End-Plate Connections: Extended end-plates are similar in appearance to shear end-plates except the plates are typically extended beyond the top and bottom of the beam in order to facilitate the bolted connection required to transfer the flange forces. The plates are typically welded to the ends of the beam (similar to direct welded flange connections) and bolted to the supporting member (see Figure 48).

- Moment Splice Connections: This type of connection is designed to transfer the flange forces across two beams in order for the entire section to behave as if it was one continuous member. Moment splices can be configured similar to flange-plated, direct welded flange and extended end-plate connections (see Figure 49).

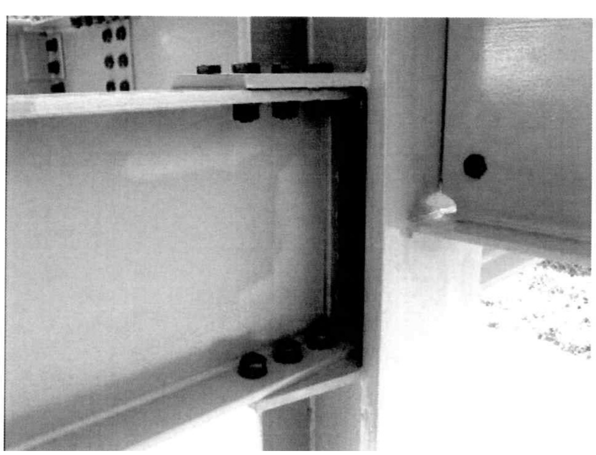


FIGURE 46

FIGURE 47

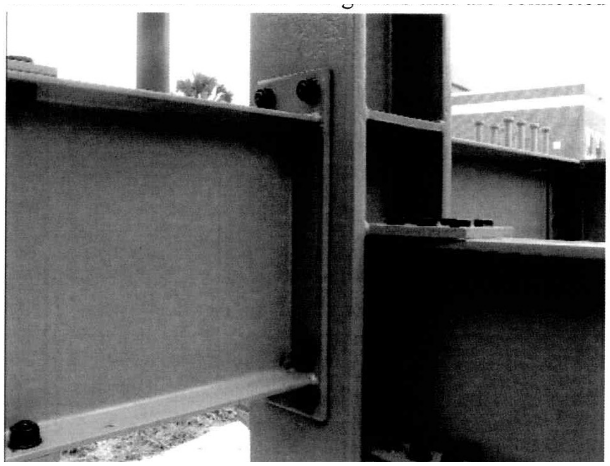


FIGURE 48

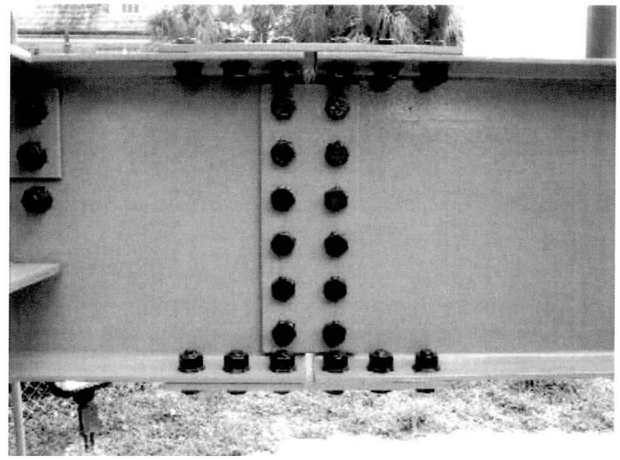


FIGURE 49

Column Connections

The design of column connections (beam-to-column and base plates) has already been discussed in Lecture 3. An additional type of column connection that has not been discussed, however, includes a column splice. Column splices are used when it is economical to change column sizes up through the building or the overall height of the structure exceeds the available fabricated column length. Column splices at both interior and exterior locations are typically located approximately 4 feet above the finished floor to facilitate ease of erection and for use in securing temporary perimeter safety cables, respectively.

Most column splices are end bearing and can be attached via bolted side plates or directly welded flange and web connections. Some columns splices must also transfer moment and or shear forces. In this case the design of the splice is similar to the moment connections described above.

Miscellaneous Connections

Miscellaneous connections are attachments that cannot be categorized with any of the above connection types. Examples of these types of connections include; Clevis (Figure 50), Skewed or Bent Plates (Figure 51) and truss panel points (Figure 52).

FIGURE 50

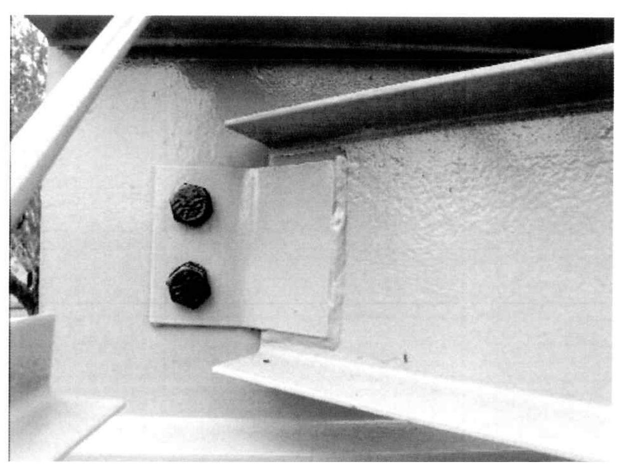


FIGURE 51

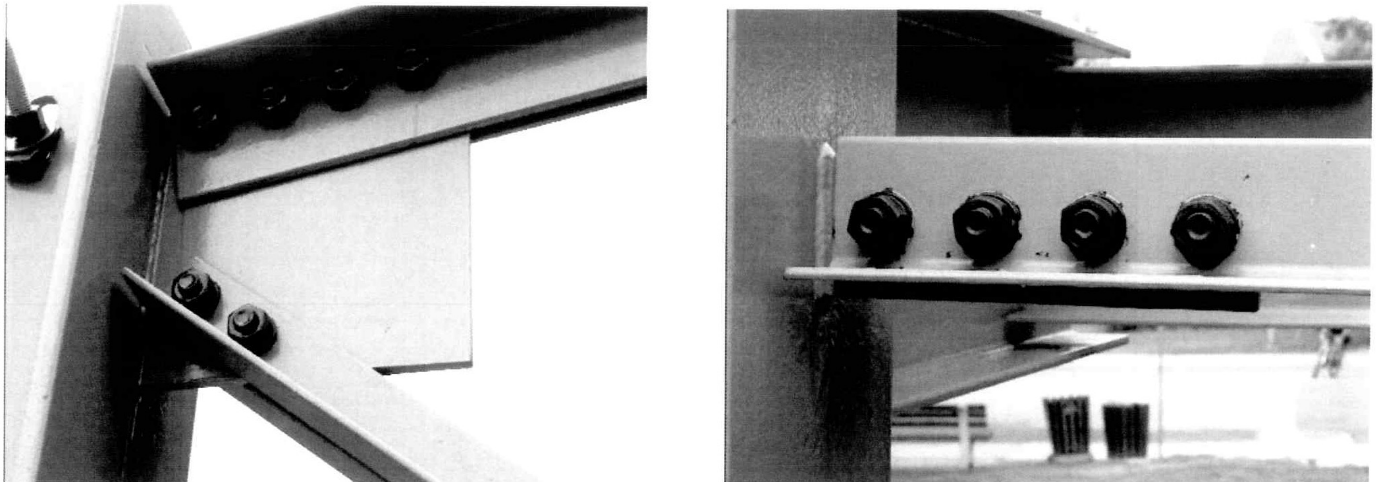


FIGURE 52

Limit States

The strength based design of steel connections is based on the limit state of either the connected members or the connectors (bolts and or welds). Each component of a connection has it own limit state failure path based on the material yield strength. At the same time each element also has a serviceability limit state that must be considered in order to provide the appropriate amount of stiffness or ductility at the connection. Common connection limit states include;

> Block Shear Rupture (BSR): BSR is a failure path that includes an area of the connection that is subject to both shear and tension. The name of this limit state is derived from the associated failure path that results in a block of the material being torn out (see Figure 53).

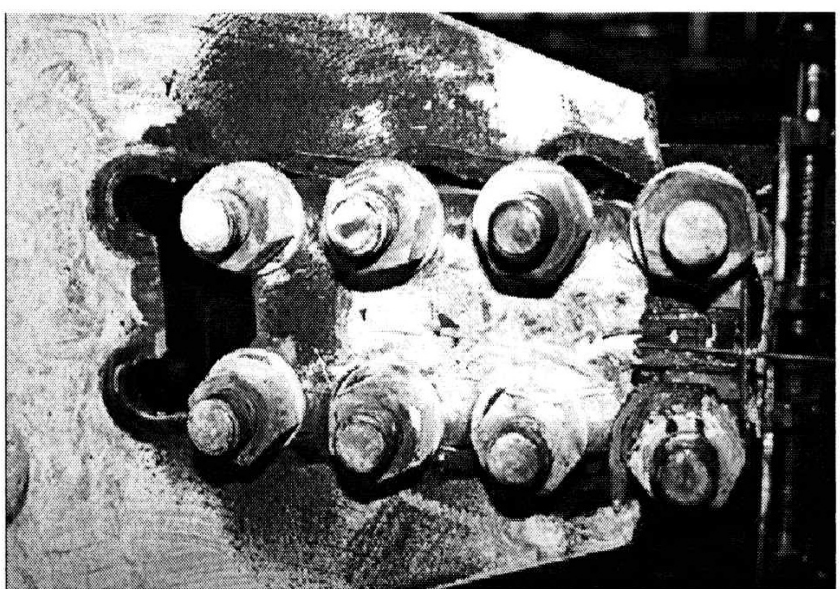


FIGURE 53

Bolt Bearing (BB): BB relates to the deformation of the material at the loaded edge of the bolt holes (see Figure 54).

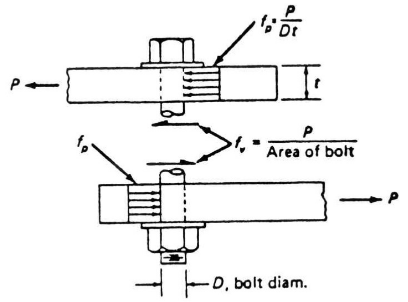


Fig. 4.6.6. Forces assumed acting on a bearing-type connection.

FIGURE 54

Bolt Shear (BS): BS is applicable to each bolted ply of a connection that is subjected to shear. Single shear (Figure 55a) occurs when the shear force is transmitted through a bolt that connects two plies. Like wise double shear (Figure 55b) occurs when the shear force is transmitted through a bolt via three plies.

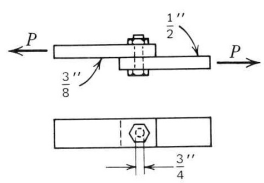


FIGURE 55a

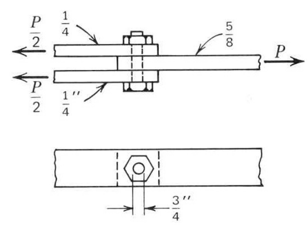


FIGURE 55b

Bolt Tension Fracture (BTF): Bolts subject to tension force along their longitudinal axis will fail in tension within the threaded portion of the bolt because this area of the bolt corresponds to the part of the bolt with the least cross sectional area.

Concentrated Forces; including Flange Local Buckling (FLB), Web Compression Buckling (WCB), Web Crippling (WC), Web Local Buckling (WLB) and Web Local Yielding (WLY): The forces that are transferred from one member to another can result in localized deformation (yielding) or buckling of the connected parts. A discussion of the checks for these localized member limit states was provided in Lecture 1. However, when the cross sectional area of a member is modified as a part of a connection it is necessary to analyze the remaining section for these same conditions. An example of this type of condition for WLB would include a situation in which the web and flange of a supported beam is coped (see Figure 56).

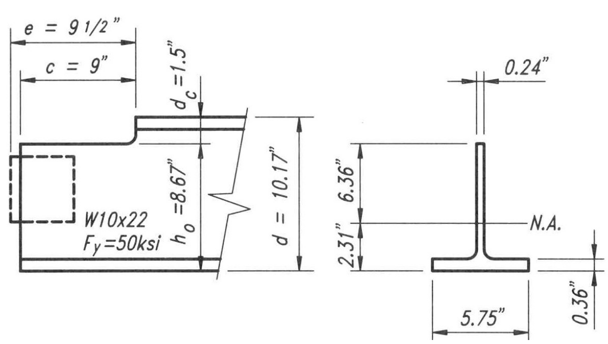


FIGURE 56

Flexural Yielding (FY): When a beam is coped the section properties (see Figure 56 above) of the remaining beam can be significantly reduced. The modified section must therefore be checked for flexural bending strength.

Prying Action (PA): PA is a phenomenon in which additional tension forces are induced in the bolts due to deformation of the connected parts near the bolts (see Figure 57).

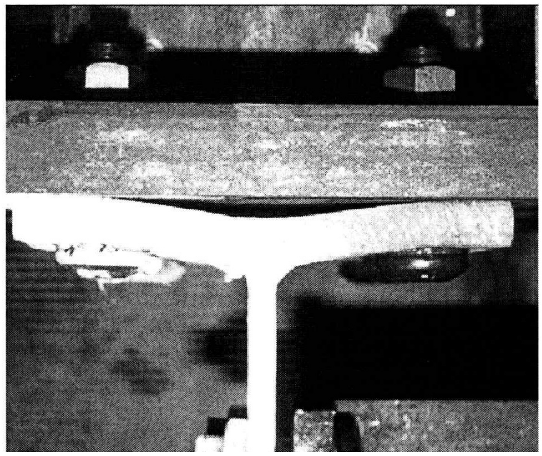


FIGURE 57

Shear Yielding and Shear Rupture (SY & SR): Most connections, including moment connections, are subject to some type of shear component. SY and SR limit state checks are therefore applicable to most connections and apply to both welded and bolted connections. For welded plies (without bolt holes) SY will typically control over SR. If the ratio of the yield strength to ultimate tensile strength is less than 1.2 then SR will generally control.

Tensile Yielding and Tension Rupture (TY & TR): TY is a limit state that is a function of the gross cross sectional area of the member that is subjected to a tension load. TR (see Figure 58) is a limit state that is a function of the effective net area (gross area - bolt holes and notches). The net area is also effectively reduced to account for shear lag. Shear lag occurs when the tension force is not evenly distributed through out the cross sectional area of the member.

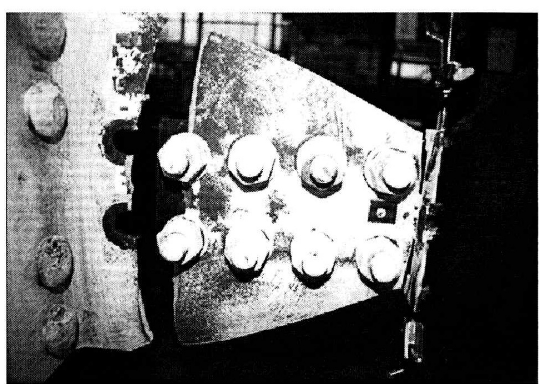


FIGURE 58

Weld Shear (WS): Weld shear is applicable to each welded ply of a connection. For example the failure mode for fillet welds is always assumed to be a shear failure of the effective throat of the weld.

Whitmore Section Yielding/Buckling (WSY): WSY is a limit state that applies to bolted and welded gusset plates and other similar connections that are much wider than the pattern of bolts of welds that fall within the plate. As a result the stress distribution through the ends of the members that are attached to the gusset is complex. WSY involves either yielding or buckling of the connecting plate near the ends of the attached members. The Whitmore Section Method (WSM) of analysis assumes that the member force is uniformly distributed over an effective area. The effective area is determined by multiplying the gusset plate thickness by an effective length that is determined for the projection of 30 degree lines on each side of the "strut" member that is connected to the gusset plate. The 30 degree lines form a trapezoid with the effective area assumed as the base dimension of the trapezoid (see Figure 59).

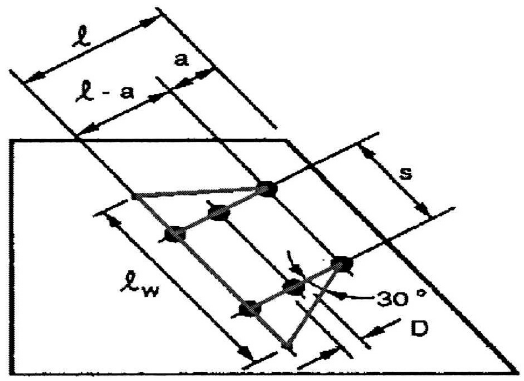


FIGURE 59

There are two different methods used to analyze a braced connection that utilizes gusset plates; The WSM (as described above) and the Uniform Force Method (UFM). The UFM involves selecting a connection geometry that eliminates (or are at least minimizes) moments on the three connection interfaces between the gusset plate, beam and column. This is accomplished by basically aligning the longitudinal axis of the connected members to a common work point (see Figure 60). This method then allows for the connection to be designed for only shear and tension forces only. It should be noted that these two methods of analysis are used to solve different problems and a direct comparison between the two is really not appropriate. For example, for a gusset plate associated with a compression brace connection the UFM would be used to design the connection of the plate to the beam and column whereas the WSM would be used to analyze the plate itself for buckling. Both checks can influence the thickness of the plate but each method addresses different types of limit states.

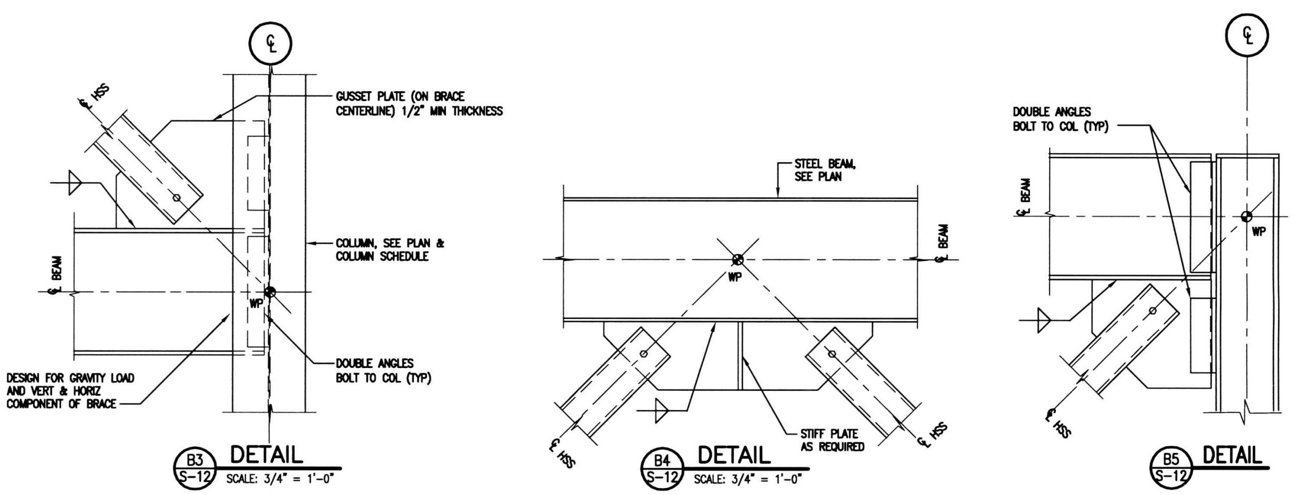


FIGURE 60

Design

Combined Shear and Tension in Bolts:

As with all other cases of combined loading. An interaction formula approach can be used for the design of bolts subjected to both shear and tension forces. The interaction formula (which is based on an elliptical interaction curve) can be expressed as indicated below where strengths can be expressed as forces or stresses with either LRFD or ASD approaches;

$$(f_t/F_t)^2 + (f_v/F_v)^2 \leq 1.0$$

Where: f_t = required tensile strength (stress)

F_t = available tensile strength (stress)

f_v = required shear strength (stress)

F_v = available shear strength (stress)

However, the AISC Specification approximates the curve using three straight lines which results in an interaction formula of;

$$(f_t/F_t) + (f_v/F_v) \leq 1.3$$

This interaction relationship is the basis for the nominal tension stress in bolts in the presence of shear found in Equations J3-3a (LRFD) and J3-3b (ASD) of the 13th Edition on page 16.1-109. The effects of combined tension and shear in slip-critical bolted connections are provided in Equations J3-5a and J3-5b (page 16.1-110).

Eccentric Bolted and Welded Connections:

An eccentric connection is one in which the resultant of the applied load does not pass through the center of gravity of the fasteners or welds. There are two types of eccentric connections; those causing shear only in the fasteners or welds (Figure 61) and those causing both shear and tension (Figure 62).

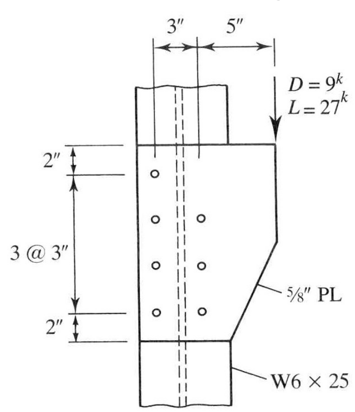


FIGURE 61

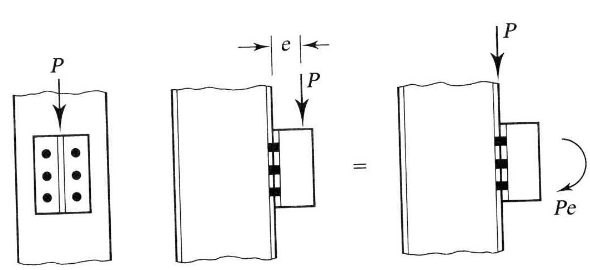


FIGURE 62

Bolted End-Plate Connection:

LRFD and ASD procedures for the design of a bolted extended end-plate connection are provided below. Additional guidelines can also be found in AISC Design Guide 4; *Extended End-Plate Moment Connections Seismic and Wind Applications* and Design Guide 16; *Flush and Extended Multiple-Row Moment End-Plate Connections*.

Typical Details

The following are examples of typical details provided on a recent project which indicate to the fabricator and erector the general intent for column splices (Figures 63 and 64), beam-to-beam moment connections (Figure 65) and beam-to-column moment connections (Figures 66 and 67).

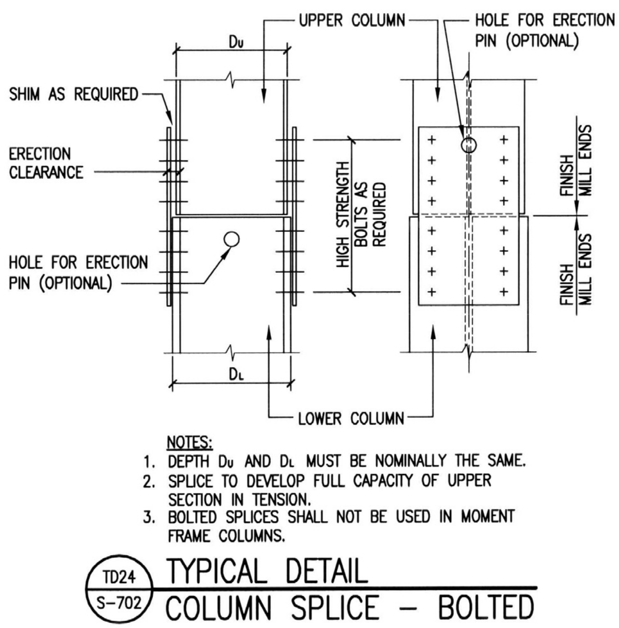


FIGURE 63

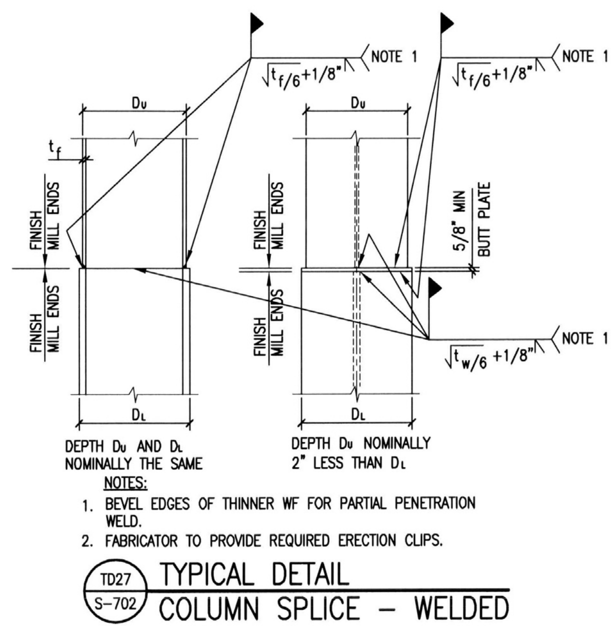


FIGURE 64

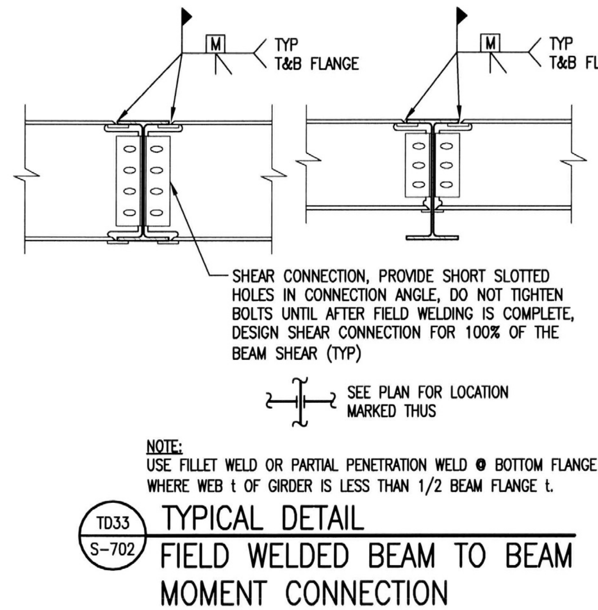


FIGURE 65

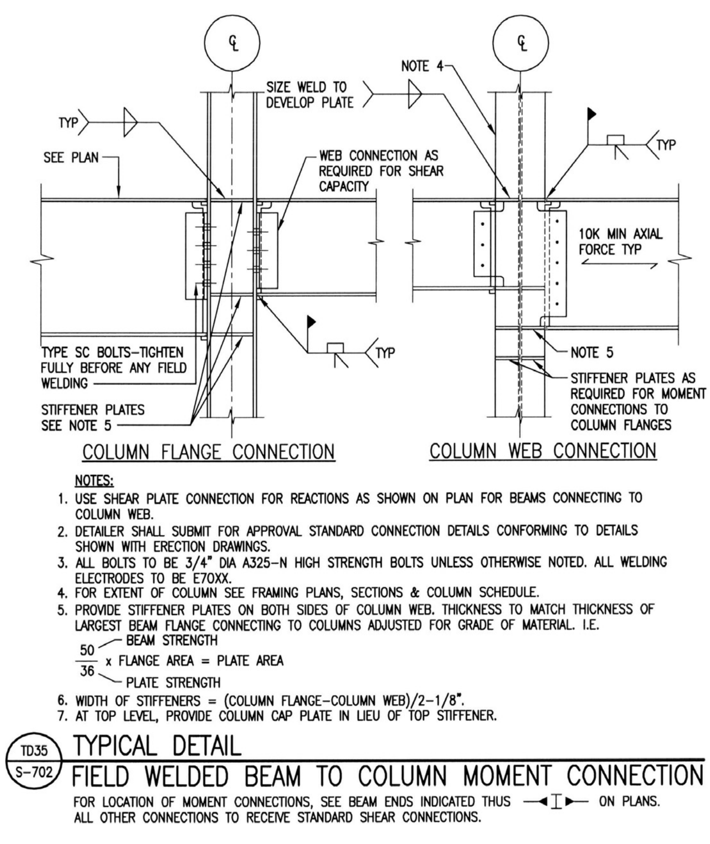


FIGURE 66

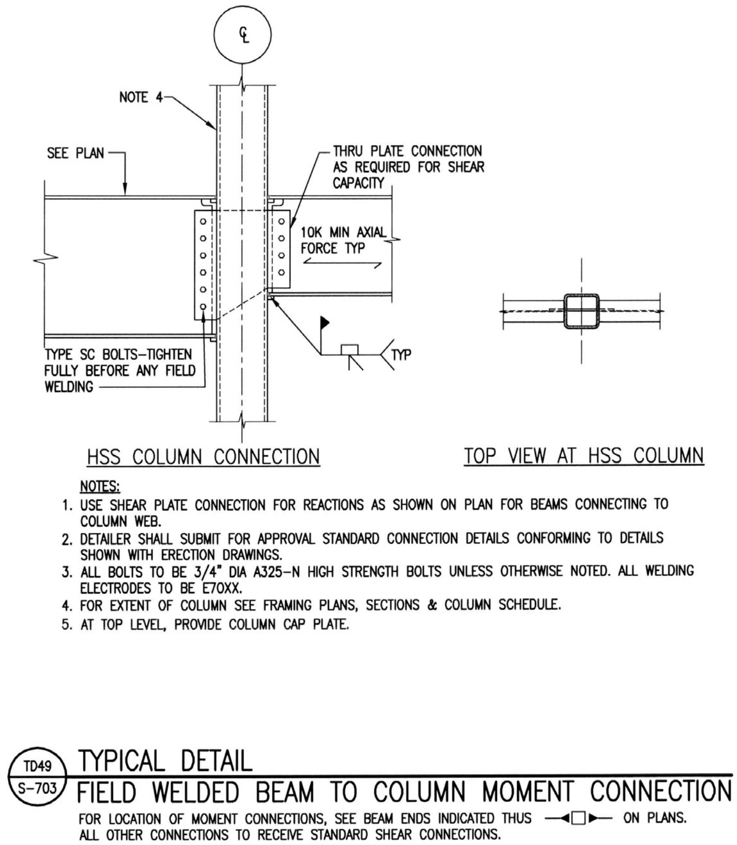


FIGURE 67
Code of Standard Practice

The current AISC Code of Standard Practice for Steel Buildings and Bridges, per Section 3.1.2, allows for the EOR to provide a complete design of all steel connections in the Contract Drawings or allow the fabricator to complete the connection details while preparing the shop and erection drawings. Because the typical fee (0.5%± of the estimated construction cost) that most structural engineers are able to obtain from the client (typically an Architect) barely allows for the structural design and detailing of the building, much less the design of all of the connections, it is more common to delegate the final design and detailing of the connections to the fabricator and his or her detailer and licensed engineer.

The only problem with this approach has been that previous editions of the Code of Standard Practice indicated that any approval of the shop drawings constituted acceptance of all responsibility for the design adequacy of any of the connections designed by the fabricator. This situation has been typically dealt with by most practicing engineers by specifically omitting this language from the referenced Code and requiring that the fabricator's engineer submit signed and sealed calculations for the connection design.

The current Code still infers this same liability by the EOR even though it is no longer explicitly stated in Sections 3.1.2 or 4.4. The resolution of this issue is currently under review by AISC and CASE. A discussion of this pending situation can be found in an article entitled *Making the Connection: Proposed Modification to AISC's Code of Standard Practice Section 3.1.2*, in the July 2008 issue of Structural Engineer magazine.