Chapter 10: Connection Elements


10.1 Introduction
A steel building structure is essentially a collection of individual members attached to each other to form a stable and serviceable whole, called a frame.
- The behavior of the connection between any two members needs to be understood by the designer since this behavior determines how the structure is analyzed to resist loads.

10.2 Basic Connections
Figures 10.1 and 10.2 (pp. 360 - 361 of the textbook) shows several examples of tension and bracket connections.

The tension connections shown include butt joint, lap joint, hanger, and gusset plate.
- Bolts used in such tension connections are subject to shear and bearing stresses.

Bolts used in bracket connections are subject to shear, bearing, and tensile stresses.
- Similar connections can be accomplished with welds.

10.3 Beam-to-Column Connections
The connection of a beam to a column can be accomplished in a variety of ways (ref. Figure 10.3, p. 361 of the textbook).
• **Simple or shear connections** include the following (Fig. 10.3a through 10.3d).
  - Double-angle connection (a.k.a. framed beam connection).
  - Single-plate connection (a.k.a. shear tab).
  - Unstiffened or stiffened seated connection (a.k.a. beam seat).

• **Fixed or moment connections** include the following (Fig. 10.3e through 10.3h).
  - Shear plate moment connections at the flange or web.
  - Field bolted moment connections.
  - End-plate moment connections.

• For the purposes of design, a connection is assumed to behave as a fixed connection or as a simple connection.

Specification Section B3.6 divides connections into two categories: **simple connections** and **moment connections**.

• The moment connection category includes **fully restrained (FR) moment connections** and **partially restrained (PR) moment connections**.
  - **Fully restrained (FR) connections** transfer moment with a negligible rotation between connected members.
  - **Partially restrained (PR) connections** transfer moment between the two members but the rotation is not negligible.
It is the designer's responsibility to match connection behavior with the appropriate analysis model and to complete the connection design so that the actual connection behavior matches that used in the analysis.

- If the beam analysis is based on moment connections, and the connections are actually simple connections, the beam will not be strong enough to resist the bending moments due to the design loads.

- If the beam analysis is based on simple connections, and the connections are actually moment connections, the moment induced at the end of the beam could cause the connection to fail.

### 10.4 Fully Restrained Connections

**Fully restrained moment connections (Type FR)** are commonly referred to as rigid or fixed connections.

- Examples of beam-to-column connections that are usually considered as fully restrained connections are shown in Figures 10.3e through 10.3h (p. 361 of the textbook).

- **Type FR connections** are assumed to be sufficiently rigid to keep the original angles between members virtually unchanged under load.

**Fully restrained (Type FR)** are those that theoretically allow no rotation at the beam ends and thus transfer 100% of the moment.

- Connections of this type may be used for tall buildings in which resistance to lateral loads (i.e. wind and seismic loads) is developed.
  - The connections provide continuity between the members of the building frame.

- Column web stiffeners (not shown in Figure 10.3) may be required for some of these connections to provide sufficient resistance to rotation.

### 10.5 Simple and Partially Restrained Connections

There are basically two ways in which a simply connected frame can be designed to resist lateral loads and provide stability for gravity loads.

- A positive bracing system, consisting of diagonal steel bracing or shear walls, can be provided.

- Lateral stability may be provided by flexible moment connections.
  - Flexible moment connections are designed for a limited amount of moment resistance accompanied by a significant amount of rotation.
  - The connections are flexible enough to rotate under gravity loads so that no gravity moments are transferred to the columns.
- At the same time, they are assumed to have sufficient strength and stiffness to resist the lateral loads and to provide frame stability.

Examples of beam-to-column connections that are usually considered as simple and partially restrained connections are shown in Figures 10.3a through 10.3d (p. 361 of the textbook).

**Simple connections (Type PR)** are quite flexible and are assumed to allow the beam ends to be free to rotate under load.
- Moment restraint is considered negligible, and the connection is assumed to resist shear only.

**Partially restrained moment connections (Type PR)** have appreciable resistance to end rotation, thus develop appreciable end moments.
- In design practice, it is common for a designer to assume all connections are either simple or rigid, with no consideration given to those situations in between, thus simplifying the analysis.
- Actual semi-rigid connections are often used, but usually no advantage is taken of their moment reducing possibilities in the calculations.
  - One factor that may keep the design professional from taking advantage of the moment reducing capabilities is the statement in the AISC Specification (Section B3.6b) that consideration of a connection as being semi-rigid is permitted only upon presentation of evidence that the connection is capable of providing a certain percentage of end restraint furnished by a completely rigid connection.
    - Such evidence must consist of documentation in the technical literature or must be established by analytical or experimental means.
    - When it becomes possible to accurately predict the percentages of rigidity for various connections, and when better design procedures are available, this type of design will likely become more common.

10.6 **Mechanical Fasteners**
For many years, riveting was the accepted method used for connecting members of steel structures.
- For the last few decades, bolting and welding have been the methods used for making steel connections, and riveting is almost never used.

Bolting of steel structures is a faster construction process that requires less skilled labor than riveting or welding.
• Bolting has an economic advantage over other connection methods in the United States where labor costs are high.

• Although the cost of a high-strength bolt is several times that of a rivet, the overall cost of bolted construction is less because of reduced labor and equipment costs and the smaller number of bolts required to resist the same loads.


• The specification includes bolt types and sizes, steels, preparations needed for bolting, use of washers, tightening procedures, and inspection.

There are several types of bolts that can be used for connecting steel members.

**Common Bolts**

*Common bolts* (a.k.a. machine, unfinished bolts, ordinary) are manufactured according to ASTM A307.

• These bolts are made from carbon steels with a lower strength.

• These bolts are available in diameters from $\frac{1}{2}''$ to $1\frac{1}{2}''$.

• Common bolts are used primarily in light structures subjected to static loads and for connections with secondary members (e.g. purlins, girts, bracing, platforms, and small trusses).

**High strength bolts**

*High strength bolts* are manufactured according to ASTM A325 and A490.

• These bolts are made from medium carbon heated-treated steels with tensile strength two or more times that of common bolts.

• These bolts are available in diameters from $\frac{1}{2}''$ to $1\frac{1}{2}''$.

• High strength bolts are used for all types of structures (from small buildings to skyscrapers and large bridges).

High strength bolts (ASTM A449) or threaded rods (ASTM A354) may be used when diameters greater than $1\frac{1}{2}''$ or lengths exceeding 8” are required.

Threaded rods (ASTM F1554) are preferred when used for anchor rods.

In the fastener tables of the AISC Manual, abbreviations are used when referring to the various types of bolts.
Examples include the following.
A325-SC - slip-critical or fully tensioned A325 bolt
A325-N - snug-tight or bearing A325 bolts with threads included in the shear planes
A325-X - snug-tight or bearing A325 bolts with threads excluded from the shear planes

Bolt Holes
The Specification defines four types of bolt holes that are permitted in steel construction: standard size bolt holes, oversize holes, short-slotted holes, and long-slotted holes.

- Table 10.2 (p. 366 of the textbook) shows the nominal hole dimensions for each of these types and for bolts from $\frac{1}{2}$" diameter and larger.

Standard size bolt holes (STD) are 1/16" larger in diameter than the bolts.
Oversized holes (OVS) help speed up construction and offer larger construction tolerances.
Short-slotted holes (SSL) may be used regardless of the direction of the applied load for slip-critical connections.
- For bearing type connections, the slots must be perpendicular to the direction of loading.
- The use of short-slotted holes provides for some mill and fabrication tolerances.
Long-slotted holes (LSL) may be used in only one of the connected parts of slip-critical or bearing-type connections at any one faying surface.
- For slip-critical joints these holes may be used in any direction.
- For bearing-type connections, the slots must be perpendicular to the direction of loading.
- Long-slotted holes are generally used when connections are being made to existing structures where the exact positions of the members being connected are not known.

In addition to prescribing bolt hole sizes, the Specification gives minimum and maximum hole spacing and edge distances.

The following definitions are given for a group of bolts in a connection.
• **Pitch** is the center-to-center distance in a direction parallel to the axis of the member.

• **Gage** is the center-to-center distance in a direction perpendicular to the axis of the member.

• The **edge distance** is the distance from the center of the bolt to the adjacent edge of a member.

• The **distance between bolts** is the shortest center-to-center distance between fasteners on the same or different gage lines.

**Minimum Spacings**

Bolts should be placed a sufficient distance apart for the following reasons.

• To permit efficient installation.

• To prevent bearing failures of the members between fasteners.

Section J3.3 of the AISC Specification prescribes the minimum center-to-center distances for standard, oversized, or slotted fastener holes.

• **Minimum center-to-center distance:** Not less than 2-2/3 bolt diameters.

• **Preferred center-to-center distance:** Not less than 3 bolt diameters.
  
  - Bearing strengths are directly proportional to the center-to-center spacing up to 3 times the bolt diameter.

  - No additional bearing strength is obtained when spacings exceed 3 times the bolt diameter.

**Minimum Edge Distances**

Bolts should not be placed too near the edges of a member for two major reasons.

• Holes punched **too close to the edges** may cause steel opposite the hole to bulge out or crack.

• Holes punched **too close to the ends** of a member may result in the fastener tearing through the metal.

Section J3.4 of the AISC Specification prescribes the minimum edge distance from the center of a standard hole to the edge of a connected part.

• The usual practice is to place the fastener a minimum distance from the edge of the plates equal to 1.5 to 2.0 times the fastener diameter.

• For more specific information, refer to Tables J3.4 and J3.4M of the AISC Specification.
Section J3.4 of the AISC Specification also prescribes the minimum edge distance from the center of an oversized hole or a slotted hole to the edge of a connected part.

• The minimum distance must equal the minimum distance required for a standard hole plus an edge distance increment $C_2$ value.
  - Values of edge distance increment $C_2$ are given in Tables J3.5 and J3.5M of the AISC Specification.

**Maximum Spacing and Edge Distance**

Section J3.5 of the AISC Specification prescribes maximum edge distances for bolted connections.

• The maximum distance from the center of any bolt or rivet to the nearest edge of parts in contact is 12 times the thickness of the connected part under consideration, but not more than 6”.

• The longitudinal spacing of fasteners between elements in continuous contact consisting of a plate and a shape or two plates is prescribed as follows.
  a) For painted members or unpainted members not subject to corrosion, the spacing shall not exceed 24 times the thickness of the thinner plate or 12”.
  b) For unpainted members of weathering steel subject to atmospheric corrosion, the spacing shall not exceed 14 times the thickness of the thinner plate or 7”.

The purpose of such requirements is to reduce the chances of moisture getting between the parts.

• When fasteners are too far from the edges of parts being connected, the edges may separate, permitting the entrance of moisture.

• When moisture gets between parts, failure of the paint may occur and corrosion will develop and accumulate, causing increased separations between parts.

The maximum edge distances and spacings for bolts used for weathering steel are smaller than they are for regular painted steel subject to corrosion, or for unpainted steel not subject to corrosion.

• Since weathering steel is not allowed to be in constant contact with water, the AISC Specification tries to ensure that the parts of the weathering steel member are connected tightly together at frequent intervals to prevent the formation of pockets that might catch and hold water.

Holes cannot be placed very close to the web-flange junction of a beam or the junction of the legs of an angle.
• Such holes can be drilled, but this is a rather expensive practice and should not be followed except for unusual circumstances.
  - Even if holes are drilled in these locations, there may be difficulty in placing and tightening the bolts in the limited available space.

10.7 Bolt Limit States

In bearing-type connections, the loads that are transferred are larger than the frictional resistance caused by tightening the bolts.
• The members slip a little and the bolts must resist both shear and bearing.

Three basic limit states govern the response of bolts in bolted connections: shear through the shank or threads of the bolt, bearing on the elements being connected, and tension in the bolt.

In cases where loaded reversals are expected or where fatigue is a factor, there is an additional limit state to prevent slip in the connection.
• This limit state applies only to connections that are classified as slip-critical.

Failure of a bolted connection can occur in several different ways as shown in figure below.

![Failure modes of bolts](image)

Bolt Shear

Failure of the bolt in a bolted connection can occur in the following ways.
1. Failure of the bolt in a lap joint by shearing of the bolt on the plane between the members (single shear) is shown in Figure 10.9a (p. 368 of the textbook).
2. Failure of the bolt in a lap joint by shearing of the bolt along two planes (double shear) is shown in Figure 10.9b (p. 368 of the textbook).
For the limit state of bolt shear, the nominal strength is based on the shear strength of the bolt and the location of the shear plane(s) with respect to the bolt threads.

Section J3.6 prescribes the nominal shear strength by the following equation.

\[ R_n = F_n A_b \]  

Equation J3-1

where

- \( F_n \) = nominal shearing strength \( F_{nv} \) of the bolt
- \( A_b \) = nominal unthreaded body area of the bolt

- LRFD design strength of a bolt in single shear: \( \phi R_n = \phi F_{nv} A_b \)
- ASD allowable strength of a bolt in single shear: \( R_n/\Omega = (F_{nv}/\Omega) A_b \)

where

- \( \phi = 0.75 \) (LRFD)
- \( \Omega = 2.00 \) (ASD)

The nominal shearing strengths (\( F_{nv} \)) of bolts are given in Table J3.2 of the AISC Specification (shown as Table 10.4, p. 368 of the textbook).

- When a bolt is in double shear, its shearing strength is twice its single shear value simply because the area resisting the load is doubled.

**Bolt Bearing**

Failure of a bolted connection, other than the failure of the bolt itself, can occur in the following ways.

1. Failure of the plate by tearing out of part of the member is shown in Figure 10.9c (p. 368 of the textbook).

2. Failure of the plate by bearing is shown in Figure 10.9d (p. 368 of the textbook).

For the limit state of bearing, the nominal strength of a bolted connection is based upon the strength of the parts being connected and the arrangement of the bolts.

- The strength of the bolted connection is dependent on the following factors.
  - The diameter of the bolts.
  - The spacing between bolts and the distances between the bolts and the edge of the connected parts.
  - The specified strength \( F_u \) of the connected parts.
  - The thickness of the connected parts.

Section J3.10 prescribes the nominal bearing strength for the limit state of bearing at bolt holes using the following equations.
a. For a bolt in a connection with standard, oversized, short-slotted hole (independent of the direction of loading), or long-slotted hole (with the slot parallel to the direction of the bearing force):
   i) When deformation at the bolt hole at service load is a design consideration (i.e. if deformations are to be ≤ 0.25”, according to the Commentary), then
   
   \[ R_n = 1.2 l_c + F_u \leq 2.4 d + F_u \]  
   AISC Equation J3-6a
   
   (Note: For this course, unless otherwise specifically stated otherwise, deformations around bolt holes will be considered important and AISC Equation J3-6a will be used for bearing calculations.)
   
   - Tests of bolted connections have shown that neither the bolts nor the metal in contact with the bolts actually fail in bearing.
   - However, if bearing stresses larger than the values given are permitted, holes seem to elongate more than \( \frac{1}{4}'' \) and impair the strength and serviceability of the connections.
   
   ii) When deformation at the bolt hole at service load is not a design consideration (i.e. if deformations can be > 0.25”), then
   
   \[ R_n = 1.5 l_c + F_u \leq 3.0 d + F_u \]  
   AISC Equation J3-6b

b. For a bolt in a connection with long-slotted holes with the slot perpendicular to the direction of the force:

   \[ R_n = 1.0 l_c + F_u \leq 2.0 d + F_u \]  
   AISC Equation J3-6c

   where
   \[ \varphi = 0.75 \text{ (LRFD)} \] and \[ \Omega = 2.00 \text{ (ASD)} \]
   
   \( l_c \) = the clear distance, in the direction of the force, between the edge of hole and the edge of adjacent hole or edge of material
   
   \( t \) = the thickness of the member bearing against the bolt
   
   \( F_u \) = specified minimum tensile strength of the connected material
   
   \( d \) = the bolt diameter

   • The LRFD design strength of a bolt in bearing: \( \varphi R_n \)
   
   • The ASD allowable strength of a bolt in bearing: \( R_n/\Omega \)

Notes
1. In Chapter 4, the author uses a bolt-hole diameter equal to \( d + 1/8 \). In the examples in this chapter, the author uses a bolt-hole diameter equal to \( d + 1/16 \).
   
   • For consistency, a bolt-hole diameter of \( d + 1/8 \) is used in the examples that follow.
• Using a larger bolt-hole diameter is conservative. Using a larger bolt-hole diameter decreases the clear distance, thus reducing the available bearing strength.

2. In AISC Equation J3-6a, the term "1.2 l_c t F_u" represents the nominal bearing strength based on "tear-out" while the term "2.4 d t F_u" represents the nominal bearing strength based on "bearing deformation."

• Specification Section J3.10 states, "For connections, the bearing resistance shall be taken as the sum of the bearing resistances of the individual bolts."
  - In the determination of the nominal bearing strength \( R_n \) for the connection based on "tear-out," this author uses \( \sum (l_c)_i \), implying that tear-out occurs simultaneously for all the bolts in the connection.
  - In the determination of the nominal bearing strength \( R_n \) for the connection based on "tear-out," another author uses \( n (l_c)_{\text{min}} \), implying that the tear-out strength is based on the "weakest link" - where the clear distance is the smallest.
  - The latter basis is the more conservative approach and will be used in the examples that follow and the approach endorsed for this course.

Bolt Tension
Section J3.6 prescribes the nominal tensile strength for the limit state of tension rupture using the following equation.

\[
R_n = F_n A_b \quad \text{Equation J3-1}
\]
where
\[
F_n = \text{nominal tensile strength } F_{nt}\text{ of the bolt}
\]
\[
A_b = \text{nominal unthreaded body area of the bolt}
\]

• LRFD design strength of a bolt in tension: \( \varphi R_n = \varphi F_{nt} A_b \)
• ASD allowable strength of a bolt in tension: \( R_n/\Omega = (F_{nt}/\Omega) A_b \)

where
\[
\varphi = 0.75 \text{ (LRFD)} \quad \text{and} \quad \Omega = 2.00 \text{ (ASD)}
\]

The nominal tensile strengths \( F_{nt} \) of bolts are given in Table J3.2 of the AISC Specification (shown as Table 10.4, p. 368 of the textbook).
Example Problems - Bearing-Type Connections

Example

Given: Bearing-type bolted connection shown.

Steel (plates): A36 (F_y = 36 ksi, F_u = 58 ksi)
Bolts: 7/8" A325-X
Standard size holes
Deformations at the bolt holes are a design consideration.

Find: The design strength $\phi P_n$ and the allowable strength $P_n/\Omega$ for the connection.

Solution

Tensile yield strength in the gross section of the plates
$P_n = F_y A_g = 36 (1/2)(12) = 216.0$ kips
LRFD ($\phi_t = 0.90$): $\phi_t P_n = 0.90 (216.0) = 194.4$ kips
ASD ($\Omega_t = 1.67$): $P_n/\Omega_t = 216.0/1.67 = 129.3$ kips

Tensile rupture strength in the net section of the plates
$A_n = (1/2)(12 - 2(7/8 + 1/8)) = 5.00$ in$^2$
From AISC Table D3.1, Case 1: $U = 1.00$ (all parts connected)
$A_e = U A_n = 1.0(5.00) = 5.00$ in$^2 \leq 0.85 A_g$ (per AISC Specification J4.1)
$0.85 A_g = 0.85(1/2)(12) = 5.10$ in$^2$ (Use $A_e = 5.00$ in$^2$)

$P_n = F_u A_e = 58.0(5.00) = 290.0$ kips
LRFD ($\phi_t = 0.75$): $\phi_t P_n = 0.75 (290.0) = 217.5$ kips
ASD ($\Omega_t = 2.00$): $P_n/\Omega_t = 290.0/2.00 = 145.0$ kips

Bearing strength of the bolts
Compute $l_c$ (using the smaller value):
Edge-to-edge of holes: $l_c = 3 - 2(1/2)(7/8 + 1/8) = 2.00$" (controls)
Edge of hole to edge of plate: $l_c = 3 - (1/2)(7/8 + 1/8) = 2.50$"
\[ R_n = 1.2 \, l_c \, t \, F_u \leq 2.4 \, d \, t \, F_u \quad \text{AISC Equation J3-6a} \]

\[ R_n = 1.2 \times (2.00)(1/2)(58.0) = 69.6 \text{ kips per bolt} \]

\[ > 2.4 \, d \, t \, F_u = 2.4 \times (7/8)(1/2)(58.0) = 60.9 \text{ kips per bolt} \]

Use \( R_n = 60.9 \text{ kips per bolt} \).

For 4-bolts: Total \( R_n = 4 \times 60.9 = 243.6 \text{ kips} \)

LRFD (\( \phi = 0.75 \)): \( \phi P_n = 0.75 \times 243.6 = 182.7 \text{ kips} \)

ASD (\( \Omega = 2.00 \)): \( P_n/\Omega = 243.6/2.00 = 121.8 \text{ kips} \)

**Shearing strength of the bolts**

\[ A_b = \pi \times \frac{d^2}{4} = \pi \times \frac{(7/8)^2}{4} = 0.601 \text{ in}^2 \]

\[ F_{nv} = 68.0 \text{ ksi} \quad \text{AISC Table J3.2 (Table 10.4, p. 368 of the textbook)} \]

\[ R_n = F_{nv} \times A_b = 68 \times 0.601 = 40.87 \text{ kips per bolt} \]

For 4-bolts: Total \( R_n = 4 \times 40.87 = 163.5 \text{ kips} \)

LRFD (\( \phi = 0.75 \)): \( \phi P_n = 0.75 \times 163.5 = 122.6 \text{ kips} \)

ASD (\( \Omega = 2.00 \)): \( P_n/\Omega = 163.5/2.00 = 81.8 \text{ kips} \)

**Answer:** Shearing strength of the bolt controls

\( \phi P_n = 122.6 \text{ kips (LRFD) and } P_n/\Omega = 81.8 \text{ kips (ASD)} \)

**Bearing strength of the bolts (alternative approach used by this author)**

Compute \( l_c \)

Edge-to-edge of holes: \( l_c = 3 - 2(1/2)(7/8 + 1/8) = 2.00'' \)

Edge of hole to edge of plate: \( l_c = 3 - (1/2)(7/8 + 1/8) = 2.50'' \)

\[ R_n = 1.2 \, l_c \, t \, F_u \leq 2.4 \, d \, t \, F_u \quad \text{AISC Equation J3-6a} \]

Tear-out

\[ R_n = 1.2 \times (2.00)(1/2)(58.0) = 69.6 \text{ kips per bolt (for each interior bolt)} \]

\[ R_n = 1.2 \times (2.50)(1/2)(58.0) = 87.0 \text{ kips per bolt (for each end bolt)} \]

Total bearing strength (tear-out): \( R_n = 2(69.6) + 2(87.0) = 313.2 \text{ kips} \)

Bearing deformation

\[ R_n = 2.4 \, d \, t \, F_u = 2.4 \times (7/8)(1/2)(58.0) = 60.9 \text{ kips per bolt} \]

Total bearing strength (bearing deformation): \( R_n = 4(60.9) = 243.6 \text{ kips} \)

Use \( R_n = 243.6 \text{ kips (same as before)} \)

LRFD (\( \phi = 0.75 \)): \( \phi P_n = 0.75 \times 243.6 = 182.7 \text{ kips} \)

ASD (\( \Omega = 2.00 \)): \( P_n/\Omega = 243.6/2.00 = 121.8 \text{ kips} \)

Note: Tear-out does not often control. So the apparent disagreement among authors may be of little real concern.
Example

Given: Bearing-type bolted connection shown.
Steel (plates):  \( F_u = 58 \text{ ksi} \)
Bolts: 3/4” A325-X
Standard size holes
Edge distance = 2”

Deformations at the bolt holes are a design consideration.

Loads: \( P_u = 300 \text{ kips (LRFD)} \)
\( P_a = 200 \text{ kips (ASD)} \)

Find: Determine the number of bolts required for the connection.

Solution

**Bearing strength of one bolt**

Compute \( l_c \) (using the smaller value):
Edge-to-edge of holes: \( l_c = 3 - 2(1/2)(3/4 + 1/8) = 2.125” \)
Edge of hole to edge of plate: \( l_c = 2 - (1/2)(3/4 + 1/8) = 1.56” \) (controls)

\[
R_n = 1.2 \ l_c \ F_u \leq 2.4 \ d \ F_u
\]
\[
R_n = 1.2 \ (1.56)(3/4)(58.0) = 81.4 \text{ kips per bolt}
\]
\[
> 2.4 \ d \ F_u = 2.4 \ (3/4)(3/4)(58.0) = 78.3 \text{ kips per bolt}
\]
Use \( R_n = 78.3 \text{ kips per bolt} \).

**Shearing strength of one bolt (double shear)**

\( A_b = \pi \ d^2/4 = \pi \ (3/4)^2/4 = 0.442 \text{ in}^2 \)
\( F_{nv} = 68.0 \text{ ksi} \) AISC Table J3.2 (Table 10.4, p. 368 of the textbook)
\[
R_n = F_{nv} \ A_b = 2 \ (68) (0.442) = 60.11 \text{ kips per bolt}
\]

Shear controls: \( R_n = 60.11 \text{ kips per bolt} \)

Determine the number of bolts required.
LRFD (\( \varphi = 0.75 \)): \( \varphi R_n = 0.75 \ (60.11) = 45.08 \text{ kips per bolt} \)
Number of bolts required: \( n = P_u/\varphi R_n = 300/45.08 = 6.65 \) (Use 7 or 8 bolts)
ASD (\( \Omega = 2.00 \)): \( R_n/\Omega = 60.11/2.00 = 30.06 \text{ kips per bolt} \)
Number of bolts required: \( n = P_a/(R_n/\Omega) = 200/30.06 = 6.65 \) (Use 7 or 8 bolts)
Slip
The limit state of slip is associated with connections that are referred to as slip-critical.
- Slip-critical connections are designed to prevent slip at the required strength limit state.

When high-strength bolts are fully tensioned, they clamp the parts being connected tightly together.
- The result is a considerable resistance to slipping on the faying surface.
  - The faying surface is the contact area or shear area between the members.
- If the shearing load is less than the permissible frictional resistance, the connection is referred to as slip-resistant.
  - If the load exceeds the frictional resistance, the members will slip, the bolts will tend to shear off, and the connected parts will bear against the bolts.
  - Special faying surface conditions are used to increase the slip resistance.
    - For example, if faying surfaces are galvanized, then the surfaces may be scored with wire brushes or sand blasted after galvanization to increase slip resistance.

Almost all bolted connections with standard size holes are designed as bearing-type connections and not as slip-critical connections.
- High-strength bolted "slip-critical" connections may be designed to prevent slippage if it is felt that slippage should be prevented (e.g. as in the case of frequent stress reversals and fatigue prone connections).

Slip-critical connections should be used only when the designer feels that slipping will adversely affect the serviceability of the structure.
- For such a structure, slipping may cause excessive distortion of the structure or a reduction in strength or stability, even if the strength of the connection is adequate.
- It may be necessary to use a slip-critical connection when oversized holes or when slots parallel to the force are used.

If bolts are tightened to the required tensions for slip-critical connections, there is little chance for slip to occur and for the bolts to bear against the plates that they are connecting.
- The required minimum bolt pretensions are given in AISC Table J3.1 and J3.1M of the AISC Manual.
Tests show that there is little chance of slip to occur between connected members unless there is a shear of at least 50% of the total bolt tension.

- As a result, slip-critical bolts are (theoretically) not stressed in shear.
- However, Section J3.8 of the AISC Specification prescribes the design shear strengths of high-strength bolts in slip-critical connections.
  - Such design shear strengths are actually design friction values on the faying surface.

Likewise, there is (theoretically) little or no bearing stress on the bolts used in slip-critical connections.

- However, Section J3.10 of the AISC Specification states that bearing strength is to be checked for both bearing-type and slip-critical connections.

Bearing strength checks for slip-critical connections may often be considered as not important.

- It is thought that the connections are not going to slip and put the bolts in bearing.
- Furthermore, it is thought that if slip does occur, the calculated bolt strength in bearing will be large compared to the calculated shearing strength.
- Usually these thoughts are correct, but if the connection consists of very thin parts, bearing may control.

Section J3.8 of the AISC Specification states that the nominal slip resistance ($R_n$) of a connection shall be determined using the following equation.

$$R_n = \mu D_u h_f T_b n_s$$  
AISC Equation J3-4

where

- $\mu$ = mean slip coefficient
  - = 0.30 for Class A faying surfaces (i.e. unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel or hot-dipped galvanized and roughened surfaces)
  - = 0.50 for Class B faying surfaces (i.e. unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel)
  (Note: Section 3.2 of Part 16.2 of the AISC Manual provides detailed information pertaining to faying surfaces.)

- $D_u = 1.13$
  (Note: This is a multiplier that gives a ratio of the mean installed pretension to the specified minimum pretension given in Table J3.1 of the AISC Specification.)
\( h_f = \) factor for fillers (coefficient to reflect the reduction in slip due to multiple filler plates), determined as follows:

- 1.00 where there are no fillers or where bolts have been added to distribute the loads in the filler
- 1.00 where bolts have not been added to distribute the load in the filler (for one filler between connected parts)
- 0.85 where bolts have not been added to distribute the load in the filler (for two or more fillers between connected parts)

\( T_b = \) minimum fastener tension, as given by AISC Table J3.1 or Table J3.1M

\( n_s = \) number of slip planes

- LRFD design strength of a bolt in single shear: \( \varphi R_n \)
- ASD allowable strength of a bolt in single shear: \( R_n/\Omega \)

where

For standard size and short-slotted holes perpendicular to the direction of the load

\( \varphi = 1.00 \) for LRFD and \( \Omega = 1.50 \) for ASD.

For oversized and short-slotted holes parallel to the direction of the load

\( \varphi = 0.85 \) for LRFD and \( \Omega = 1.76 \) for ASD

For long-slotted holes

\( \varphi = 0.70 \) for LRFD and \( \Omega = 2.14 \) for ASD

The majority of bolted connections made with standard-size holes can be designed as bearing-type connections.

- If connections are made with three or more bolts in standard-size holes, or are used with slots perpendicular to the force direction, slip probably cannot occur because at least one or more of the bolts will likely be in bearing before the external loads are applied.
  - As the members are assembled their weights often push the bolts against the sides of the holes before they are tightened and put the bolts in some bearing and shear.
Example Problems – Slip-Critical Connections

Example

Given: Lap joint shown (slip-critical connection).
Service loads: \( P_D = 30 \) kips, \( P_L = 50 \) kips

Bolts: \( 1" - A325-X \) in standard size holes with no fillers

Class A faying surface

Edge distance (in the direction of the force) = 1.75"

Bolt spacing (in the direction of the force) = 3" c/c

Steel: \( F_Y = 50 \) ksi, \( F_U = 65 \) ksi

Find: Number of bolts required.

Solution

Calculate the loads to be resisted.

\[
LRFD: \quad P_u = 1.2 \, D + 1.6 \, L = 1.2 \, (30) + 1.6 \, (50) = 116.0 \text{ kips}
\]

\[
ASD: \quad P_a = D + L = 30 + 50 = 80.0 \text{ kips}
\]

Compute the nominal slip-resistance of one bolt.

\[
R_n = \mu \, D_u \, h_f \, T_b \, n_s \quad \text{AISC Equation J3-4}
\]

where

\[
\mu = 0.30 \text{ for Class A faying surfaces}
\]

\[
D_u = 1.13
\]

\[
h_f = 1.00 \text{ (no fillers)}
\]

\[
T_b = 51 \text{ kips for } 1" - A325-SC \text{ bolt (ref. Table J3.1)}
\]

\[
n_s = \text{number of slip planes} = 1.0
\]

\[
R_n = 0.30(1.13)(1.00)(51)(1.0) = 17.29 \text{ kips/bolt}
\]

Slip-critical design.

\[
LRFD (\varphi = 1.00): \quad \varphi R_n = 1.00 \times 17.29 = 17.29 \text{ kips/bolt}
\]

Number of bolts required = 116.0/17.29 = 6.71 \quad \text{(Use 7 or 8 bolts)}

\[
ASD (\Omega = 1.50): \quad \frac{R_n}{\Omega} = 17.29/1.50 = 11.53 \text{ kips/bolt}
\]

Number of bolts required = 80.0/11.53 = 6.94 \quad \text{(Use 7 or 8 bolts)}
Check the bearing strength of 7 bolts.

- Section J3.10 states that bearing strength must be checked for slip-critical connections.

Applicable equation: $R_n = 1.2 \cdot l_c \cdot F_u \leq 2.4 \cdot d \cdot F_u \quad \text{AISC Equation J3-6a}$

Compute $l_c$ (using the smaller value):
- Edge-to-edge of holes: $l_c = 3 - 2(1/2)(1 + 1/8) = 1.875\"$
- Edge of hole to edge of plate: $l_c = 1.75 - (1/2)(1 + 1/8) = 1.187\"$ (controls)

$R_n = 1.2 \cdot l_c \cdot F_u \leq 2.4 \cdot d \cdot F_u \quad \text{AISC Equation J3-6a}$

$R_n = 1.2 \cdot (1.187)(5/8)(65.0) = 57.87 \text{ kips per bolt}$

$< 2.4 \cdot d \cdot F_u = 2.4 \cdot (1)(5/8)(65.0) = 97.5 \text{ kips per bolt}$

Use $R_n = 57.87 \text{ kips per bolt}$.

For 7-bolts: Total $R_n = 7 \cdot (57.87) = 405.1 \text{ kips}$

LRFD ($\phi = 0.75$): $\phi P_n = 0.75 \cdot (405.1) = 303.8 \text{ kips} > 116.0 \text{ kips} \quad \text{OK}$

ASD ($\Omega = 2.00$): $P_n/\Omega = 405.1/2.00 = 202.6 \text{ kips} > 80.0 \text{ kips} \quad \text{OK}$

Check the shear strength of 7 bolts (single shear).

Applicable equation: $R_n = F_{nv} A_b$

$F_{nv} = 68.0 \text{ ksi} \quad \text{AISC Table J3.2 (Table 10.4, p. 368 of the textbook)}$

$A_b = \pi d^2/4 = \pi (1.0)^2/4 = 0.785 \text{ in}^2$

$R_n = F_{nv} A_b = 68(0.785) = 53.38 \text{ kips/bolt}$

For 7-bolts: Total $R_n = 7 \cdot (53.38) = 373.7 \text{ kips}$

LRFD ($\phi = 0.75$): $\phi P_n = 0.75 \cdot (373.7) = 280.3 \text{ kips} > 116.0 \text{ kips} \quad \text{OK}$

ASD ($\Omega = 2.00$): $P_n/\Omega = 373.7/2.00 = 186.9 \text{ kips} > 80.0 \text{ kips} \quad \text{OK}$
Example
This example is a repeat of the previous example using long-slotted holes (instead of standard size holes) in the direction of the load.
• Assume that deformations of the connections will cause an increase in the critical load.
• Design the connection to prevent slip.

Given: Lap joint shown (slip-critical connection).
Service loads: \( P_D = 30 \) kips, \( P_L = 50 \) kips
Bolts: 1" - A325-X in long-slotted holes with no fillers
Class A faying surface
Edge distance (in the direction of the force) = 1.75"
Bolt spacing (in the direction of the force) = 3" c/c
Steel: \( F_y = 50 \) ksi, \( F_u = 65 \) ksi

Find: Number of bolts required.

Solution

Calculate the loads to be resisted.
LRFD: \( P_u = 1.2 \ D + 1.6 \ L = 1.2 \ (30) + 1.6 \ (50) = 116.0 \) kips
ASD: \( P_a = D + L = 30 + 50 = 80.0 \) kips

Compute the nominal slip-resistance of one bolt.
\( R_n = \mu \ D_u \ h_f T_b \ n_s \)  \( \text{AISC Equation J3-4} \)
where
\( \mu = 0.30 \) for Class A faying surfaces
\( D_u = 1.13 \)
\( h_f = \) filler factor = 1.0 (no fillers)
\( T_b = 51 \) kips for 1" - A325 bolt (ref. Table 12.1, p. 370 of the textbook)
\( n_s = \) number of slip planes = 1.0
\( R_n = 0.30(1.13)(1.00)(51)(1.0) = 17.29 \) kips/bolt (same as before)
**Slip-critical design.**

LRFD ($\phi = 0.70$): $\phi R_n = 0.70 \times 17.29 = 12.10$ kips/bolt

Number of bolts required = $116.0/12.10 = 9.59$ (Use 10 bolts)

ASD ($\Omega = 2.14$): $R_n/\Omega = 17.29/2.14 = 8.08$ kips/bolt

Number of bolts required = $80.0/8.08 = 9.90$ (Use 10 bolts)

Shear and bearing were checked in the previous example and are obviously OK here since a larger number of bolts are required.
Combined Tension and Shear in Bearing-Type Connections

According to Specification Section J3.7, the available tensile strength of a bolt subjected to combined tension and shear is determined according to the limit states of tension and shear rupture using the following equation.

\[ R_n = F'_{nt} A_b \tag{Equation J3-2} \]

where

\[ F'_{nt} = \text{the nominal tensile stress modified to include the effects of shear stress} \]

\[ F'_{nt} = 1.3 F_{nt} - \left( F_{nt}/\phi F_{nv} \right) f_{rv} \leq F_{nt} \quad \text{for LRFD} \tag{Equation J3-3a} \]

\[ F'_{nt} = 1.3 F_{nt} - \left( \Omega F_{nt}/F_{nv} \right) f_{rv} \leq F_{nt} \quad \text{for ASD} \tag{Equation J3-3b} \]

\[ F_{nt} = \text{nominal tensile stress from AISC Table J3.2} \]

\[ F_{nv} = \text{nominal shear stress from AISC Table J3.2} \]

\[ f_{rv} = \text{required shear stress using LRFD or ASD load combinations} \]

- LRFD design strength of a bolt in single shear: \( \phi R_n \)
- ASD allowable strength of a bolt in single shear: \( R_n/\Omega \)

where

\[ \phi = 0.75 \quad \text{(LRFD)} \quad \text{and} \quad \Omega = 2.00 \quad \text{(ASD)} \]

The available shear stress of the fastener shall equal or exceed the required shear stress, \( f_{rv} \).

When the required stress in either shear or tension is less than or equal to 30% of the corresponding available stress, the effects of the combined stresses can be ignored.
10.8 Welds

Welding is a process by which metallic parts are connected by heating their surfaces to a plastic or fluid state and allowing the parts to flow together and join (with or without the addition of other molten metal).

The adoption of structural welding was slow for several decades because of two perceived major disadvantages.
1. Welds had reduced fatigue strength compared with riveted and bolted connections.
2. It was impossible to ensure a high quality of welding without extensive and costly inspection.

These attitudes persisted for years although tests began to show that neither reason was valid.
• Today most engineers agree that welded joints have considerable fatigue strength.
• They also admit that the rules governing the qualification of welders, the better techniques applied, and the excellent workmanship requirements of the AWS (American Welding Society) specifications make the inspection of welding much less of a problem.
• Furthermore, the chemistry of steels manufactured today is especially formulated to improve their weldability.

Advantages of Welding
Following are several of the advantages that welding offers.
1. The first advantage is economic.
   • Welding permits large savings in pounds of steel used.
     - Welded structures allow the elimination of a large percentage of the gusset and splice plates necessary for bolted structures.
2. Welding has a wider range of application than bolting.
   • For example, consider a steel pipe column and the difficulty of connecting it to other steel members by bolting.
3. Welded structures are more rigid, because the members are welded directly to each other.
   • Conversely, the connections for bolted structures are made through intermediate connection angles or plates that deform due to load transfer, making the entire structure more flexible.
4. The process of fusing pieces together creates the most truly continuous structures.
   - This continuity advantage has permitted the construction of countless slender and graceful statically indeterminate steel frames throughout the world.
   - Outspoken proponents of welding have referred to bolted structures, with their heavy plates and abundance of bolts, looking like armored cars compared with the clean, smooth lines of welded structures.

5. It is easier (and less expensive) to make changes in design and to correct errors during construction if welding is used.

6. Welding is relatively silent.

7. Fewer pieces are used, and as a result, time is saved in detailing, fabrication, and field construction.

American Welding Society (AWS)
The American Welding Society's *Structural Welding Code* is the generally recognized standard for welding in the United States.
   - The AISC Specification clearly states that the provisions of the AWS Code apply under the AISC Specification, with only a few minor exceptions.
     - Exceptions are listed in Section J2 of the AISC Specification.

Welding Processes
Although both gas welding and arc welding are available, almost all structural welding is arc welding.
   - In electric-arc welding, the metallic rod which is used as the electrode melts off into the joint as the weld is being made.
   - When gas welding is used, it is necessary to introduce a metal rod known as a filler or welding rod.
     - Gas welding is a slow process compared with other means of welding.
     - Gas welding is normally used for repair and maintenance work and not for fabrication and assembly of large steel structures.

The American Welding Society's *Structural Welding Code* accepts four welding processes as being prequalified.
   - Prequalified means that the processes are acceptable without the necessity of further proof of their suitability by procedure qualification tests.
The prequalified processes that are listed in the AWS Specification 13.1 include the following.
1. Shielded metal arc welding (SMAW)
   - The SMAW process is the usual process applied for hand welding.
2. Submerged arc welding (SAW) - automatic or semi-automatic.
3. Gas shielded metal arc welding (GMAW) - automatic or semi-automatic.
4. Flux-cored arc welding (FCAW) - automatic or semi-automatic.

Further discussion for each of these methods may be found in the textbook (pp. 378-379).

Welding Inspection
Three steps must be taken to ensure good welding for a particular job.
1. Good welding procedures.
2. Prequalified workers.
3. Competent inspectors in the shop and in the field.

Visual Inspection: Visual inspection by a competent person usually gives a good indication of the quality of welds.
- A competent inspector should be able to recognize good welds in regard to shape, size, and general appearance.
  - The metal in a good weld should return to its original color after it has cooled.
    - For example, if the metal is overheated, the weld may have a rusty and reddish-looking color.
- An inspector may use various scales and gages to check the sizes and the shapes of the welds.
- Visual inspection is the most economical inspection method and is particularly useful for single-pass welds.

Visual inspection cannot provide information regarding the subsurface condition of the weld.
- There are several methods for determining the internal soundness of a weld, including the use of penetrating dyes and magnetic particles, ultrasonic testing, and radiographic procedures.
  - These methods can be used to detect internal defects such as porosity, weld penetration, and the presence of slag.
**Liquid Penetrants:** For this method, dyes are spread over weld surfaces and penetrate into the surface cracks of the weld.
- Like a visual inspection, this method enables the visual detection of cracks that are open to the surface.

**Magnetic Particles:** In this method, the weld being inspected is magnetized electrically.
- Cracks that are at or near the surface of the weld cause north/south poles to form along each side of the cracks.
- Dry iron powdered filings or a liquid suspension of particles is placed on the weld.
  - The particles form patterns when many of them cling to the cracks.
  - The particles show the locations of the cracks and indicate the size and shape of the cracks.

**Ultrasonic Testing:** Sound waves are sent through the material being tested and are reflected from the opposite side of the metal.
- Defects in the weld affect the time of the sound transmission.
  - The operator can read the picture, locate the flaws, and learn how severe the flaws are.
- Ultrasonic testing is expensive and does not work too well for some stainless steels or for extremely coarse-grained steels.

**Radiographic Procedures:** The more expensive radiographic methods can be used to check occasional welds in important structures (e.g. welding of stainless steel piping at chemical or nuclear projects).
- These methods are satisfactory for butt welds, but not for fillet welds.
- These methods can also pose radioactive danger.
  - Careful procedures are used to protect the technicians as well as nearby workers.

**Types of Weld**

**Classifications:** Welds are classified based on the types of welds made, the positions of the welds, and the types of joints used.

**Weld Types:** Four basic types of welds are used in steel construction: fillet welds, groove welds, plug welds, and slot welds (ref. Figure 10.13, p. 380 of the textbook).
- The fillet and groove welds are common in structural work; the plug and slot welds are not as common in structural work.
• Fillet welds are the most economical weld.
  - Most structural connections are made by fillet welds.

• Groove welds are generally more expensive than fillet welds because of the cost of preparation and the difficulty of fitting pieces together in the field.

Position: Welds are referred to as **flat, horizontal, vertical, or overhead** - listed in order of their economy.
• Flat welds are the most economical.
• Overhead welds are the most expensive.

**Type of Joint:** Welds may be further classified according to the type of joint used: **butt, lap, tee, edge, or corner.**

**Welding Symbols:** The American Welding Society has developed various welding symbols.
• With this standardized shorthand system, a great deal of information can be presented in a small space on construction drawings.

**Weld Sizes**
Specification Section J2 addresses effective areas and sizes for welds.
• The effective dimensions of groove welds are given in Tables J2.1 and J2.2.
• The effective areas of fillet welds are given in Specification Section J2.2.

The minimum permissible size fillet weld is dependent on the thinner of the two parts being joined and may not exceed the thickness of the thinner part (ref. Table J2.4 of the AISC Manual).

<table>
<thead>
<tr>
<th>Part thickness</th>
<th>Minimum size of fillet weld</th>
</tr>
</thead>
<tbody>
<tr>
<td>¼&quot; or less</td>
<td>1/8”</td>
</tr>
<tr>
<td>Over ¼&quot; and up to ½&quot;</td>
<td>3/16”</td>
</tr>
<tr>
<td>Over ½&quot; and up to ¾&quot;</td>
<td>1/4”</td>
</tr>
<tr>
<td>Over ¾&quot;</td>
<td>5/16”</td>
</tr>
</tbody>
</table>

• The smallest practical weld size is 1/8”.
• The most economical weld size is probably 1/4” or 5/16”.
• The 5/16” weld is the largest weld that can be made in one pass with the shielded metal arc welded process (SMAW).
• The 1/2” weld is the largest weld that can be made in one pass with the submerged arc process (SAW).
The maximum size of a fillet weld of connected parts is based on the following.

<table>
<thead>
<tr>
<th>Part thickness</th>
<th>Maximum size of fillet weld</th>
</tr>
</thead>
<tbody>
<tr>
<td>Less than $\frac{1}{4}$&quot;</td>
<td>Maximum weld size equals part thickness</td>
</tr>
<tr>
<td>$\frac{1}{4}$&quot; or greater</td>
<td>1/16&quot; less than part thickness</td>
</tr>
</tbody>
</table>

10.9 Weld Limit States

The only limit state to be considered for a weld is shear rupture through the throat.

- Yielding of the weld metal will occur, but it occurs over such a short distance that it is not a factor in connection behavior.

The American Welding Society classifies electrodes according to the tensile strength of the weld metal and indicates electrode strength as $F_{EXX}$.

- E represents electrode strength.
- XX represents the tensile strength.

A typical electrode used to weld A992 steel has a strength of 70 ksi and is designated as an E70 electrode.

Table J2.5 of the AISC Specification provides nominal strengths for various types of welds.

Fillet Weld Strength

The design strength ($\varphi R_n$) and the allowable strength ($R_n/\Omega$) of a weld equal the lower value of the base metal strength and the weld metal strength and may be found using the following equations.

For the base metal, the nominal strength is

$$R_n = F_{nBM} A_{BM}$$

AISC Equation J2-2

For the weld metal, the nominal strength is

$$R_n = F_{nw} A_{we}$$

AISC Equation J2-3

where

- $F_{nBM} = \text{the nominal stress of the base metal}$
- $F_{nw} = \text{the nominal stress of the weld metal} = 0.6 F_{EXX}$
- $A_{BM} = \text{the cross-sectional area of the base metal}$
- $A_{we} = \text{the effective area of the weld} = \text{throat x weld length} = 0.707 \ w \ l$
- $w = \text{weld size}$
$l = \text{weld length}$

- LRFD design strength of a bolt in shear rupture: $\varphi R_n$
- ASD allowable strength of a bolt in shear rupture: $R_n/\Omega$

where

$$\varphi = 0.75 \text{ (LRFD)} \quad \text{and} \quad \Omega = 2.00 \text{ (ASD)}$$

For the most commonly used electrode, $F_{EXX} = 70 \text{ ksi}$:

- LRFD design strength = $\varphi R_n = 0.75(0.6)70(0.707wl) = 22.27wl$
- ASD allowable strength = $R_n/\Omega = 0.6(70)(0.707wl)/2.00 = 14.85wl$

When a load is applied to a fillet weld at an angle other than along the length of the weld, Section J2.4 of the AISC Specification states that the strength of the fillet welds may be determined by the following equation.

$$F_{nw} = (0.6 F_{EXX})(1.0 + 0.50 \sin^{1.5}\theta) \quad \text{AISC Equation J2-5}$$

where

- $F_{EXX} = \text{the electrode classification number, ksi}$
- $\theta = \text{the angle between the line of action of the load and the longitudinal axis of the weld}$

As the angle $\theta$ increases, the strength of the weld increases.

- If the load is perpendicular to the longitudinal axis of the weld, the result is a 50% increase in the computed weld strength.
  $$F_{nw} = (0.6 F_{EXX})(1.0 + 0.50 \sin^{1.5}\theta) = (0.6 F_{EXX})(1.0 + 0.50) = 1.5 (0.6 F_{EXX})$$

- If the load is parallel to the longitudinal axis of the weld, the strength of the fillet weld is simply
  $$F_{nw} = (0.6 F_{EXX})(1.0 + 0.50 \sin^{1.5}\theta) = (0.6 F_{EXX})(1.0 + 0) = 0.6 F_{EXX}$$

Other important provisions related to welding are given in Section J2.2b of the AISC Specification.

1. The minimum length of a fillet weld may not be less than four times the nominal leg size of the weld (or else the size of the weld shall be considered not to exceed $\frac{1}{4}$ of its length).

2. The use of end returns, or boxing, at the end of fillet welds is recommended to provide better fatigue resistance and to make sure that weld thicknesses are maintained over their full lengths.
3. When longitudinal welds are used for the connection of plates and bars, the length may not be less than the perpendicular distance between the welds, because of shear lag.

4. For lap joints, the minimum amount of lap permitted is equal to five times the thickness of the thinner part joined, but may not be less than 1", in order to keep the joint from rotating excessively.

5. If the actual length (l) of an end-loaded fillet weld is greater than 100 times its leg size (w), because of stress variations along the weld, it is necessary to determine a smaller effective length for determining the strength of the weld.
   • The effective length is determined by multiplying the actual length by the reduction factor β, where
     \[
     \beta = 1.2 - 0.002 \left( \frac{l}{w} \right) \leq 1.0 \quad \text{AISC Equation J2-1}
     \]
   • If the actual weld length (l) is greater than 300w, the effective length shall be taken as 180w.
Example Problems - Design of Simple Fillet Welds

Example

Given: 1" length of 1/4” fillet weld formed by the shielded metal arc process
Electrode: E70 (minimum tensile strength \( F_{\text{EXX}} = 70 \text{ksi} \))
Load is parallel to the weld length.

Find:

a. Strength \((\varphi R_n \text{ and } R_n/\Omega)\) of the 1" weld.
b. Strength \([\varphi R_n \beta L \text{ and } (R_n/\Omega) \beta L]\) if the weld is 20" long.
c. Strength \([\varphi R_n \beta L \text{ and } (R_n/\Omega) \beta L]\) if the weld is 30” long.

Solution

a. Strength \((\varphi R_n \text{ and } R_n/\Omega)\) of the 1" weld.

Nominal weld strength: \( R_n = F_w A_w \)
\[ F_{nw} = \text{nominal strength of the weld metal} = 0.60 F_{\text{EXX}} = 0.60 (70) = 42.0 \text{ksi} \]
\[ A_{we} = \text{effective area of the weld} = \text{throat \times weld length} \]
\[ A_{we} = (1/4) (0.707) (1.0) = 0.1767 \text{in}^2 \]
\[ R_n = 42.0 (0.1767) = 7.42 \text{kips/inch} \]

Compute the weld strength.
LRFD design strength \((\varphi = 0.75)\): \( \varphi R_n = 0.75 (7.42) = 5.56 \text{kips/inch} \)
ASD allowable strength \((\Omega = 2.00)\): \( R_n/\Omega = 7.42/2.00 = 3.71 \text{kips/inch} \)

b. Strength \([\varphi R_n \beta L \text{ and } (R_n/\Omega) \beta L]\) if the weld is 20” long.

Check the length to weld size ratio: \( l/w = 20/(1/4) = 80 < 100 \), thus \( \beta = 1.0 \)

Compute the weld strength.
LRFD design strength: \( \varphi R_n \beta L = 5.56 \text{kips/inch} (1.0) (20) = 111.2 \text{kips} \)
ASD allowable strength: \( (R_n/\Omega) \beta L = 3.71 \text{kips/inch} (1.0) (20) = 74.2 \text{kips} \)

c. Strength \([\varphi R_n \beta L \text{ and } (R_n/\Omega) \beta L]\) if the weld is 30” long.

Check the length to weld size ratio: \( l/w = 30/(1/4) = 120 > 100 \)
\[ \beta = 1.2 - 0.002 (l/w) = 1.2 - 0.002 (120) = 0.96 \]

Compute the weld strength.
LRFD design strength: \( \varphi R_n \beta L = 5.56 \text{kips/inch} (0.96) (30) = 160.1 \text{kips} \)
ASD allowable strength: \( (R_n/\Omega) \beta L = 3.71 \text{kips/inch} (0.96)(30) = 106.8 \text{kips} \)
Example

Given: The welded connection shown.
   Electrode: E70 (minimum tensile strength $F_{\text{EXX}} = 70$ ksi)
   Steel: A572 Grade 50 ($F_y = 50$ ksi, $F_u = 65$ ksi)

Note: The plate is listed as PL $\frac{3}{4} \times 10$ but dimensioned as 8" wide. A width of 10" is used in the following calculations.

Find: Design strength of the connection.

Solution

**Determine the weld strength.**

Compute the nominal weld strength: $R_n = F_{nw} A_{we}$

- $F_{nw} =$ nominal strength of the weld metal = 0.60 $F_{\text{EXX}} = 0.60 (70) = 42.0$ ksi
- $A_{we} =$ effective area of the weld = throat x weld length
  - = (7/16) (0.707) (1.0) = 0.3093 in$^2$ per inch of weld

- $R_n = 42.0 (0.3093) = 12.99$ kips per inch of weld

Check the length to weld size ratio: $l/w = 10/(7/16) = 22.86 < 100$, thus $\beta = 1.0$

Compute the LRFD weld strength.

- $\varphi R_n \beta L = 0.75(12.99)(1.0)(2)(10.0) = 194.9$ kips

- $\text{ASD allowable strength } (\Omega = 2.00):$
  - $(R_n/\Omega) \beta L = (12.99/2.00)(1.0)(2)(10.0) = 129.9$ kips

**Compute the tensile yield strength of the gross section for PL $\frac{3}{4} \times 10$.**

Compute the nominal strength of the plate: $R_n = F_y A_g$

- $R_n = F_y A_g = 50(3/4)(10) = 375.0$ kips

Compute the LRFD (\varphi = 0.90): $\varphi R_n = 0.90 (375.0) = 337.5$ kips
- $\text{ASD } (\Omega = 1.67): R_n/\Omega = 375.0/1.67 = 224.6$ kips
Compute tensile rupture strength of the net section for PL \( \frac{3}{4} \times 10 \).

Compute the nominal strength: \( R_n = F_u A_e \)

From AISC Table D3.1, Case 4:

\[
1.5w = 1.5(10) = 15'' > l = 10'' \geq w = 10'', \text{ thus } U = 0.75
\]

\[
A_n = (3/4)(10) = 7.5 \text{ in}^2
\]

\[
A_e = U A_n = 0.75 (7.5) = 5.625 \text{ in}^2
\]

\[
R_n = F_u A_e = 65 (5.625) = 365.6 \text{ kips}
\]

Compute the tensile rupture strength of the net section.

LRFD (\( \varphi = 0.75 \)): \( \varphi R_n = 0.75 (365.6) = 274.2 \text{ kips} \)

ASD (\( \Omega = 2.00 \)): \( R_n/\Omega = 365.6/2.00 = 182.8 \text{ kips} \)

Answer: Weld strength controls - 194.9 kips (LRFD) and 129.9 kips (ASD)
Example

Given: The welded connection shown.
   Electrode: E70 (minimum tensile strength $F_{Exx} = 70$ ksi)
   Steel: $F_y = 50$ ksi, $F_u = 65$ ksi

Find: Design the shielded metal arc weld (SMAW) to resist a full-capacity load on the 3/8” x 6” member.

Solution

Compute the tensile yield strength of gross section for PL 3/8” x 6”.
Compute the nominal strength: $R_n = F_y A_g$
   $R_n = F_y A_g = 50(3/8)(6) = 112.5$ kips

Compute the design strength.
   LRFD ($\varphi = 0.90$): $\varphi R_n = 0.90 (112.5) = 101.2$ kips
   ASD ($\Omega = 1.67$): $R_n/\Omega = 112.5/1.67 = 67.4$ kips

Compute the tensile rupture strength of the net section for PL 3/8” x 6”.
Compute the nominal strength: $R_n = F_u A_e$
   From AISC Table D3.1, Case 4: Assume $U = 0.75$
   $A_n = (3/8)(6) = 2.25$ in$^2$
   $A_e = U A_n = 0.75 (2.25) = 1.6875$ in$^2$
   $R_n = F_u A_e = 65 (1.6875) = 109.7$ kips

Compute the design strength.
   LRFD ($\varphi = 0.75$): $\varphi R_n = 0.75 (109.7) = 82.3$ kips
   ASD ($\Omega = 2.00$): $R_n/\Omega = 109.7/2.00 = 54.8$ kips

Tensile rupture strength of the net section controls:
   $\varphi R_n = 82.3$ kips (LRFD) and $R_n/\Omega = 54.8$ kips (ASD)

Design the weld.
Select weld size (both plates are 3/8”).
   Maximum weld size = 3/8 – 1/16 = 5/16”
   Minimum weld size = 3/16” (AISC Table J2.4)
   Use 5/16” weld (maximum size for one pass)
Compute the nominal weld strength per inch of weld:

\[ R_n = F_{nw} A_{we} \]

- \( F_{nw} = \) nominal strength of the weld metal = 0.60 \( F_{EXX} = 0.60 \times 70 = 42.0 \text{ ksi} \)
- \( A_{we} = \) effective area of the weld per inch = throat \( \times \) 1" weld length
  - \( = (5/16) \times (0.707) \times (1.0) = 0.221 \text{ in}^2 \) per inch of weld
- \( R_n = 42.0 \times 0.221 = 9.28 \text{ kips per inch of weld} \)

Compute the weld strength per inch of weld.

- LRFD design strength (\( \varphi = 0.75 \)):
  - \( \varphi R_n = 0.75 \times 9.28 = 6.96 \text{ kips per inch of weld} \)
- ASD allowable strength (\( \Omega = 2.00 \)):
  - \( R_n/\Omega = 9.28/2.00 = 4.64 \text{ kips per inch of weld} \)

Determine the required weld length.

- Minimum weld length = 6"
  - LRFD: \( 82.3/6.96 = 11.82" \) (Say 12", 6" on each side)
  - ASD: \( 54.8/4.64 = 11.81" \) (Say 12", 6" on each side)

Check the length to weld size ratio (LRFD and ASD).

- \( l/w = 6.0/(5/16) = 19.2 < 100 \), thus \( \beta = 1.0 \)

Check the shear lag factor \( U \).

- From AISC Table D3.1, Case 4:
  - \( 1.5w = 1.5(6) = 9" > l = 6" \geq w = 6", \) thus \( U = 0.75 \) (as assumed)
  - No changes required.

Use 5/16" SMAW weld, 6" on each side.
Longitudinal and Transverse Fillet Welds

Section J2.4(c) of the AISC Specification states that the total nominal strength of a connection with side and transverse fillet welds is equal to the larger of the values obtained from the following two equations.

\[
R_n = R_{nw\ell} + R_{nwt} \quad \text{AISC Equation J2-10a}
\]

\[
R_n = 0.85 R_{nw\ell} + 1.5 R_{nwt} \quad \text{AISC Equation J2-10b}
\]

where

\[
R_{nw\ell} = \text{the total nominal strength of the longitudinal or side fillet welds} = F_{nw\ell} A_{we}
\]

\[
R_{nwt} = \text{total nominal strength of the transversely loaded fillet welds} = F_{nwt} A_{we}
\]

The total nominal strength of the transversely loaded fillet welds, \( R_{nwt} \), is calculated with \( F_{nw} = 0.60 F_{EXX} \) and not with \( F_{nw} = (0.60 F_{EXX})(1.0 + 0.50 \sin^{1.5}\theta) \).
Example Problem – Longitudinal and Transverse Fillet Welds

Example

Given: The welded connection shown.
   Electrode: E70 (i.e. the minimum tensile strength \( F_{EXX} = 70 \) ksi)
   Weld size: 5/16”

Find: The total LRFD design strength and the total ASD allowable strength of the weld.

Solution

Compute the nominal weld strength of the longitudinal weld: \( R_{wl} = F_{nw} A_{we} \)
   \( F_{nw} = \) nominal strength of the weld metal = 0.60 \( F_{EXX} = 0.60 (70) = 42.0 \) ksi
   \( A_{we} = \) effective area of the weld = throat \( \times \) weld length
   \( = (5/16) (0.707) 2 (8.0) = 3.535 \) in\(^2\)
   \( R_{nwl} = 42.0 (3.535) = 148.5 \) kips

Compute the nominal weld strength of the transverse weld: \( R_{wt} = F_{nw} A_{we} \)
   \( F_{nw} = \) nominal strength of the weld metal = 0.60 \( F_{EXX} = 0.60 (70) = 42.0 \) ksi
   \( A_{we} = \) effective area of the weld = throat \( \times \) weld length
   \( = (5/16) (0.707) (10.0) = 2.209 \) in\(^2\)
   \( R_{nwt} = 42.0 (2.209) = 92.8 \) kips

Determine the total nominal weld strength of the connection with both transverse and longitudinal welds. (Use the larger value of AISC Equations J2-10a and J2-10b.)

\[
R_n = R_{nwl} + R_{nwt} \quad \text{AISC Equation J2-10a}
\]
\[
= 148.5 + 92.8 = 241.3 \text{ kips}
\]

\[
R_n = 0.85 R_{nwl} + 1.5 R_{nwt} \quad \text{AISC Equation J2-10b}
\]
\[
= 0.85 (148.5) + 1.5 (92.8) = 265.4 \text{ kips (controls)}
\]

Compute the strength of the weld.
   LRFD design strength (\( \varphi = 0.75 \)): \( \varphi R_n = 0.75(265.4) = 199.1 \) kips
   ASD allowable strength (\( \Omega = 2.00 \)): \( R_n/\Omega = 265.4/2.00 = 132.7 \) kips
**Groove Weld Strength**

Three types of groove welds are commonly used (ref. Figure 10.13, p. 380 of the textbook): *square groove joint*, *single-vee joint*, and *double-vee joint*.

- The square groove joint is used to connect relatively thin material up to 5/16”.
- As the material becomes thicker, it is necessary to use the single-vee groove weld and the double-vee groove weld.

Groove welds are said to have **reinforcement** (i.e. added weld metal that causes the throat dimension to be greater than the thickness of the welded material).

- The reinforcement gives a little extra strength because the extra material takes care of pits and other defects.
- For vibrating loads, stress concentrations develop in the reinforcement and may contribute to earlier failure.
  - It is common practice to provide reinforcement and then grind it off flush with the material being connected (AASHTO Section 10.34.2.1).

**Full-Penetration Groove Welds**

- If plates with different **thicknesses** are joined, the strength of a full-penetration groove weld is based on the strength of the **thinner** plate.
- If plates with different **strengths** are joined, the strength of a full-penetration groove weld is based on the strength of the **weaker** plate.

Full penetration welds are the best type of weld for resisting fatigue failures.

- Some specifications only permit full-penetration groove welds if fatigue is possible.

**Partial-Penetration Groove Welds**

Groove welds that do not extend completely through the full thickness of the parts being joined are referred to as **partial-penetration groove welds**.
• Such welds can be made from one or two sides, with or without preparation of the edges (such as bevels).

Partial-penetration welds often are economical in cases in which the welds are not required to develop large forces in the connected materials (e.g. column splices and connecting parts of various built-up members).

In Table J2.5 of the AISC Specification, the design stresses for partial-penetration welds are the same as for full-penetration welds when compression or tension is parallel to the axis of the weld.
• If tension is transverse to the weld axis, there is substantial strength reduction because of the possibility of high stress concentrations.
Example Problem – Full-Penetration and Partial Penetration Groove Welds

Example

Given: The plates shown.

Electrode: E70 (minimum tensile strength $F_{\text{EXX}} = 70$ ksi)

Steel: $F_y = 50$ ksi

Find:

a. The LRFD design strength and the ASD allowable strength of a shielded metal arc weld (SMAW) full-penetration groove weld.

b. The LRFD design strength and the ASD allowable strength of a shielded metal arc weld (SMAW) partial-penetration groove weld with 45° bevel and a depth of $\frac{1}{2}$".

Solution

Full-penetration groove weld
From AISC Table J2.5: Strength of the joint is controlled by the base metal.

Determine the nominal strength of the base metal.

$R_n = \text{strength of base material} = F_y A_g = 50 \times (3/4)(6.0) = 225.0$ kips

Compute the LRFD design strength and the ASD allowable strength.

LRFD ($\phi_t = 0.90$): $\phi R_n = 0.90(225.0) = 202.5$ kips

ASD ($\Omega_t = 1.67$): $R_n/\Omega = 225.0/1.67 = 134.7$ kips

Partial-penetration weld
From AISC Table J2.5: Strength of the joint is controlled by the weaker of the base metal or the weld.

Determine the LRFD design strength and the ASD allowable strength of the base metal.

Base metal values are the same as above: LRFD = 202.5 kips, ASD = 134.7 kips

Determine the nominal strength of the weld.

Effective throat = depth of groove minus $1/8”$  
$= 1/2” - 1/8” = 3/8”$  

AISC Table J2.1
The nominal strength of the weld: \( R_n = F_{nw} A_{we} \)

From AISC Table J2.5: \( F_{nw} = 0.60 F_{EXX} \)

\( A_{we} = \text{effective throat of weld} \times \text{weld length} = (3/8)(6.0) = 2.25 \text{ in}^2 \)

\( R_n = 0.60 \times 70 \times 2.25 = 94.5 \text{ kips} \)

**Compute the LRFD design strength and the ASD allowable strength.**

**LRFD (\( \varphi = 0.80 \)):** \( \varphi R_n = 0.80 \times 94.5 = 75.6 \text{ kips} \)

**ASD (\( \Omega = 1.88 \)):** \( R_n / \Omega = 94.5 / 1.88 = 50.3 \text{ kips} \)

**Strength of the joint is controlled by the weld:** 75.6 kips (LRFD), 50.3 kips (ASD)
10.10 Connecting Elements
The plates, gussets, angles, brackets, and other elements that form a connection are called connecting elements.

- Specification Section J4 prescribes the strength for such elements loaded in tension, compression, flexure, shear, and block shear.

Connecting Elements in Tension
Although the Specification addresses tension in connecting elements in Section J4.1, the basic tension provisions found in Specification Chapter D are not altered.

- Two limit states are considered: the limit state of yielding and the limit state of rupture.

For tensile yielding of connecting elements,
\[ R_n = F_y A_g \quad \text{Equation J4-1} \]
\[ \varphi = 0.90 \text{ for LRFD} \quad \text{and} \quad \Omega = 1.67 \text{ for ASD} \]

For tensile rupture of connecting elements,
\[ R_n = F_u A_e \quad \text{Equation J4-2} \]
\[ \varphi = 0.75 \text{ for LRFD} \quad \text{and} \quad \Omega = 2.00 \text{ for ASD} \]

where
\[ A_e = \text{effective net area as defined in Section D3.} \]
For bolted splice plates, \( A_e = A_n \leq 0.85 A_g \).

Connecting Elements in Compression
Most connecting elements in compression are relatively short and have a low slenderness ratio (i.e. \( KL/r \)).

- The determination of the appropriate effective length factor requires significant engineering judgment, usually amounting to making an educated guess for such a factor.

- To simplify the connection design, Section J4.4 of the Specification provides a simple approach for the compressive strength of connecting elements if the slenderness ratio is no greater than 25.

- Two limit states are considered: the limit state of yielding and the limit state of buckling.

When \( KL/r \leq 25 \),
\[ R_n = F_y A_g \quad \text{Equation J4-6} \]
\[ \varphi = 0.90 \text{ for LRFD} \quad \text{and} \quad \Omega = 1.67 \text{ for ASD} \]

When \( KL/r > 25 \), the provisions of Chapter E (Design of Members for Compression) apply.
Connecting Elements in Flexure
According to Specification Section J4.5, the available flexural strength of connected members shall be the lower value obtained according to the limit states of flexure yielding, local buckling, flexural lateral-torsional buckling, and flexural rupture.

- These are the same limit states considered for flexural members in Specification Chapter F, Design of Members for Flexure.

Connecting Elements in Shear
According to Specification Section J4.2, the available shear strength of connecting elements in shear shall be the lower value obtained according to the limit states of shear yielding and shear rupture.

For shear yielding of the element:

\[ R_n = 0.6 F_y A_{gv} \]  
Equation J4-3

\[ \phi = 0.90 \text{ for LRFD and } \Omega = 1.67 \text{ for ASD} \]

where

\[ A_{gv} = \text{gross area subject to shear} \]

For shear rupture of the element:

\[ R_n = 0.6 F_u A_{nv} \]  
Equation J4-4

\[ \phi = 0.75 \text{ for LRFD and } \Omega = 2.00 \text{ for ASD} \]

where

\[ A_{nv} = \text{net area subject to shear} \]

Block Shear Strength
The available strength for the limit state of block shear rupture along a shear failure path or paths and a perpendicular tension failure path is determined by the following equation.

\[ R_n = 0.60 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.60 F_y A_{gv} + U_{bs} F_u A_{nt} \]  
Equation J4-5

\[ \phi = 0.75 \text{ for LRFD and } \Omega = 2.00 \text{ for ASD} \]

where

\[ A_{gv} = \text{gross area subject to shear} \]
\[ A_{nt} = \text{net area subject to tension} \]
\[ A_{nv} = \text{net area subject to shear} \]
\[ U_{bs} = 1 \text{ for a uniform tensile stress distribution} \]
\[ = 0.5 \text{ for a nonuniform tensile stress distribution} \]

The only case identified by the Commentary where the tensile stress distribution is not uniform is that of a coped beam with two rows of bolts.