Chapter 11: Simple Connections

11.1 Types of Simple Connections
This chapter addresses two types of connections: simple beam shear connections and simple bracing connections.

- Connection design is a combination of element and connector selection with a check of all appropriate limit states.
- The goal is to select a connection with sufficient strength and appropriate stiffness to carry the load in a manner consistent with the model used in the structural analysis.

The limit states that must be considered for a particular connection depend on the connection elements, the connection geometry, and the load path.

- For bolts, the limit states of tensile rupture, shear rupture, bearing and tear-out, and slip, as appropriate are considered.
- For welds, shear rupture is the only limit state that is considered.
- For connecting elements, the limit states of tension yielding and rupture, compression buckling, flexure, and shear yielding and rupture must be considered.

Under present-day steel specifications, three types of fasteners are permitted for beam-to-beam and beam-to-column connections: welds, unfinished bolts, and high-strength bolts.

Selection of a type of fastener or fasteners to be used for a particular structure involves many factors, including requirements of local building codes, relative economy, preference of the designer, availability of good welders, loading conditions (e.g. static vs. fatigue loadings), preference of the fabricator, and available equipment.

The following information may be helpful in making a decision on the choice of fastener.

1. Unfinished bolts are often economical for light structures subject to small static loads and for secondary members (e.g. purlins, girts, and bracing) in larger structures.

2. Field bolting is very rapid and involves less skilled labor than welding; however, the purchase price of high-strength bolts is rather high.
3. If a structure is later to be disassembled, welding probably is not the best option, and the task is the selection of bolts.

4. For fatigue loadings, slip-critical high-strength bolts and welds are good options.

5. Special care must be taken to install high-strength, slip-critical bolts properly.

6. Welding requires the smallest amounts of steel, probably provides the most attractive-looking joints, and has the widest range of application to different types of connections.

7. When continuous and rigid structure with moment-resisting joints is desired, welding will probably be selected.

8. Welding is almost universally accepted as being satisfactory for shop-work.
   - For field work, welding is popular in most areas of the United States, while in other areas of the United States the use of welding is hindered by the idea that field inspection is rather questionable.

9. Welding of very thick members requires a great deal of extra care.
   - Bolted connections may be a better option.
   - Such bolted connections are far less susceptible to brittle fractures.

### 11.2 Simple Shear Connections

Five of the most commonly used simple shear connections include the following.

1. Double-angle (a.k.a. framed beam connection).
2. Single angle.
4. Unstiffened beam seat.
5. Stiffened beam seat connections.

Standard bolted connections (ref. Figure 11.1, p. 398 of the textbook) are usually designed to resist shear only.

- A **framed connection** between beams, or between a beam and column, consists of a pair of flexible angles.
  - The angles are usually shop-connected to the web of the supported beam and field-connected to the supporting beam, girder, or column.
  - When two beams are connected, it is usually necessary to keep the top flanges at the same elevation, requiring the top flange of the connected beam to be cut back (called **coping**).
    - **Coping** is an expensive process and should be avoided where possible.
A seated connection features an angle under the beam.
- The angle is usually shop-connected to the flange of the column.
- A second angle helps in keeping the top flange of the beam from being twisted out of place during construction.
  - The second angle is usually located at the top of the beam, or at an optional location on the side of the beam.
  - The second angle is field-connected to the beam and column.
- For heavy loads it is necessary to use stiffened beam seats.

Part 10 of the AISC Manual includes many tables that facilitate connection design.
- The AISC Manual has excellent design tables for selecting bolted or welded beam connections of the types shown in Figure 11.1 (p. 398 of the textbook).
- The designer can select most of these connections by simply referring to these standard design tables.

To qualify as a simple end connection, the end of the beam should be as free as possible to rotate.
- To allow such rotation, the angles used for the framed and seated end connections theoretically deform allowing the end of the beam to rotate.
  - The use of thin angles and large gages for the bolt spacing help make the end connections flexible.
  - When the outstanding legs of angles are welded, the vertical welds are placed as far apart as possible to help make the end connections flexible.

Simple connections do have some resistance to moment.
- When the end of the beam begins to rotate, the rotation is resisted to some degree by the tension in the top bolts.
- By neglecting the moment resistance provided by these connections, the result is conservative beam sizes.
- If moments of any significance are to be resisted by the connections, then a more rigid-type joint must be provided than the flexible framed and seated connections.

In Part 10 of the AISC Manual, a series of design tables are presented that may be used to select several different types of standard connections, including the following.
- Bolted or welded double-angle framed connections.
• Bolted or welded unstiffened seated beam connections.
• Bolted or welded stiffened seated beam connections.
• Single-angle framed connections.
• Shear end plates.
• Eccentrically loaded connections.

11.3 Double-Angle Connections: Bolted-Bolted
For small and low-rise buildings (and that means most buildings), simple double-angle connections (a.k.a. framed beam connections) are usually used to connect beams to girders, or beams to columns.
• The angles used are rather thin so that they have the necessary flexibility for the connection to be considered a simple connection.
  - An arbitrary maximum angle thickness of \( \frac{1}{2} '' \) is used in the AISC Manual.

AISC Table 10-1 is a design aid for all-bolted double-angle connections.
• Tabulated bolt and angle available strengths consider the limit states of bolt shear, bolt bearing on the angles, shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles.

Group A and Group B bolts are defined in AISC Specification Section J3.1.
• Group A bolts include ASTM A325, A325M, F1852, A354 Grade BC, and A449.
• Group B bolts include ASTM A490, A490M, F2280, and A354 Grade BD.

Tabulated values of "bolt and angle available strength" consider the limit states of bolt shear, bolt bearing on the angles, shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles.
• Values are tabulated for 2 through 12 rows of 3/4'', 7/8'', and 1'' diameter for Group A and Group B bolts at 3'' spacing.
• For calculation purposes, angle edge distances, \( L_e \) and \( L_{eh} \), are assumed to be 1\( \frac{1}{2} '' \) although the tabulated values are 1\( \frac{1}{2} '' \) and 1\( \frac{3}{4} '' \) to account for possible underrun in beam length.

Tabulated values of "beam web available strength per inch thickness" of web thickness consider the limit state of bolt bearing on the beam web.
  - For beams coped at the top flange only, the limit state of block shear rupture is also considered.
  - For beams coped at both the top and bottom flanges, the tabulated values consider the limit states of shear yielding and shear rupture of the beam web.

11.4
Tabulated values of “supporting available strength per inch thickness” of supporting member available strengths per inch of flange or web thickness consider the limit state of bolt bearing on the support.

**Design Notes for AISC Table 10-1**

- The framing angles extend out from the beam web by \( \frac{1}{2} \)" (known as *setback*) to provide clearance between the end of the beam and the supporting beam web or column flange.

- The minimum length of the framing angles should be at least equal to one-half the distance between the web toes of the beam fillets.
  - The distance between the web toes of the beam fillets is designated as “T” and is listed in the properties table of Part 1 of the AISC Manual.

- Using the design tables of Part 10 of the AISC Manual, double-angle bolted connections can be selected if one or both flanges of a beam are coped.
  - Coping may substantially reduce the design strength of beams and may require the use of larger members or the addition of web reinforcing.
Example Problems - Double-Angle Connections: Bolted-Bolted

Example

Given: Standard bolted-bolted double-angle beam connection shown for the uncoped W30 x 108 beam connected to the flange of a W14 x 61 column.

Service loads: \( R_D = 50 \text{ kips} \)
\( R_L = 70 \text{ kips} \)

Steel (angles): \( F_y = 36 \text{ ksi} \)
\( F_u = 58 \text{ ksi} \)

Steel (beam and column):
\( F_y = 50 \text{ ksi}, F_u = 65 \text{ ksi} \)

Bolts: \( \frac{3}{4}'' \) A325-N in standard size holes

Find: Select the connection using LRFD and ASD.

Solution

\( W30 \times 108 \ (t_w = 0.545'', T = 26\frac{1}{2}'') \)
\( W14 \times 61 \ (t_f = 0.645'') \)

LRFD
\( R_u = 1.2 \ D + 1.6 \ L = 1.2 \ (50) + 1.6 \ (70) = 172 \text{ kips} \)

Determine the number of bolts, the angle thickness, and the angle length using AISC Table 10-1 (Part 10 of the AISC Manual).
- Start with the least number of rows (i.e. 5 rows) that can be used with a W30 section and \( \frac{3}{4}'' \) A325-N bolts (p. 10-20).
  - A325-N bolts are defined as Group A in Specification Section J3.1.
  - Try a connection with an angle thickness of \( 3/8'' \).
    - The "bolt and angle available strength": \( \varphi R_n = 179 \text{ kips} > R_u = 172 \text{ kips} \) OK

Check the bearing strength of the beam web.
- The tabulated LRFD "beam web available strength per inch thickness" for standard bolt holes (STD), an uncoped beam, and a horizontal edge distance \( L_{eh} = 1\frac{3}{4}'' \) is 439 kips/inch.
  - Bearing strength of the beam web:
    \( \varphi R_n = 439 \times 0.545 = 239.2 \text{ kips} > R_u = 172 \text{ kips} \) OK
Check the bearing strength of the supporting column flange.
• The tabulated LRFD "support available strength per inch thickness" for standard bolt holes (STD) is 878 kips/inch.
  Bearing strength of the column flange:
  \[ \varphi R_n = 878 \times (0.645) = 566.3 \text{ kips} > R_u = 172 \text{ kips} \quad \text{OK} \]

Determine the angle dimensions.
• Angle length = 4 spaces \(@ 3”/\text{space} + 2 \text{ (edge distance)} \]
  \[ = 4 \times (3”) + 2 \times (1\frac{1}{4”}) = 14.5” < T = 26.5” \quad \text{OK} \]
• Length of the connected leg (beam web) = gage + 1” edge distance
  \[ = 2.5 + 1 = 3.5” \quad \text{(Say 3\frac{1}{2”})} \]
  Note: A gage of 2.5” is used for the legs bolted to the beam web. Standard gage distances for angles are shown in Table 1-7A on p. 1-48 of the AISC Manual.
• Length of the outstanding leg (column flange)
  \[ = \text{angle thickness} + H_2 + C_1 + 1” \text{ edge distance} \]
  \[ = 3/8 + 1 - 3/8 + 1 - 1/4 + 1 = 4” \quad \text{(Say 4”)} \]
  Note: \( H_2 \) and \( C_1 \) are minimum clearances needed for inserting and tightening the bolts (ref. Table 7-15 of the AISC Manual).

Select 2L’s 4 x 3\frac{1}{2} x 3/8 x 14\frac{1}{2}” long (A36), using five \( \frac{3}{8}” \) A325-N bolts in the connected leg (beam web).

**ASD**
\[ R_a = D + L = 50 + 70 = 120 \text{ kips} \]

Determine the number of bolts, the angle thickness, and the angle length using AISC Table 10-1 (Part 10 of the AISC Manual).
• Start with the least number of rows (i.e. 5 rows) that can be used with a W30 section and \( \frac{3}{8}” \) A325-N bolts (p. 10-20).
  - A325-N bolts are defined as Group A in Specification Section J3.1.
    The "bolt and angle available strength" < 120 kips for all 4 of the angle thicknesses listed. Thus, a 5-row connection is not adequate.
• Move to a 6-row connection for a W30 section and \( \frac{3}{8}” \) A325-N bolts (p. 10-19).
  Try a connection with an angle thickness of 5/16”.
  The bolt and angle available strength: \( R_n/\Omega = 124 \text{ kips} > R_a = 120 \text{ kips} \quad \text{OK} \)
Check the bearing strength of the beam web.
• The tabulated ASD "beam web available strength per inch thickness" for standard bolt holes (STD), an uncoped beam, and a horizontal edge distance \( L_{eh} = 1\frac{3}{4}'' \) is 351 kips/inch.
  
  Bearing strength of the beam web:
  \[
  R_n/\Omega = 351 \times (0.545) = 191.3 \text{ kips} > R_a = 120 \text{ kips} \quad \text{OK}
  \]

Check the bearing strength of the supporting column flange.
• The tabulated ASD "support available strength per inch thickness" for standard bolt holes (STD) is 702 kips/inch.
  
  Bearing strength of the column flange:
  \[
  R_n/\Omega = 702 \times (0.645) = 452.8 \text{ kips} > R_a = 120 \text{ kips} \quad \text{OK}
  \]

Determine the angle dimensions.
• Angle length = 5 spaces @ 3''/space + 2 (edge distance)
  \[
  = 5 \times (3'') + 2 \times (1\frac{1}{4}'') = 17.5'' < T = 26.5'' \quad \text{OK}
  \]

• Length of the connected leg (beam web) = gage + 1'' edge distance
  \[
  = 2.5 + 1 = 3.5'' \quad (\text{Say } 3\frac{1}{2}'')
  \]

Note: A gage of 2.5'' is used for the legs bolted to the beam web. Standard gage distances for angles are shown in Table 1-7A on p. 1-48 of the AISC Manual.

• Length of the outstanding leg (column flange)
  \[
  = \text{angle thickness} + H_2 + C_1 + 1'' \text{ edge distance}
  = 5/16 + 1-3/8 + 1-1/4 + 1 = 3-15/16'' \quad (\text{Say } 4'')
  \]

Note: \( H_2 \) and \( C_1 \) are minimum clearances needed for inserting and tightening the bolts (ref. Table 7-15 of the AISC Manual).

Select 2L's 4 x 3\frac{1}{2} x 5/16 x 17\frac{3}{4}'' long (A36), using six \( \frac{3}{4}'' \) A325-N bolts in the connected leg (beam web).
11.4 Double-Angle Connections: Welded-Bolted

AISC Table 10-2 in Part 10 of the AISC Manual is a design aid arranged to permit substitution of welds for bolts in connections designed with AISC Table 10-1.

- Table 10-2 conforms to the bolt spacing and edge distance requirements for the all-bolted double-angle connections of AISC Table 10-1.
- Weld available strengths are tabulated for the limit state of weld shear.
- The weld strength values in Table 10-2 are based on the E70 electrode.
- Table 10-2 is normally used where the angles are welded to the beams in the shop and then field-bolted to the other member (ref. Figure 11.4, p. 409 of the textbook).

Design notes for AISC Table 10-2

For the usual situations, two $4 \times 3\frac{1}{2}$ angles are used.

- The $3\frac{1}{2}''$ connected leg is connected to the beam web by welding and the $4''$ outstanding leg is connected to the other member by bolting.
  - The length of the connected leg to the beam web may be optionally reduced from $3\frac{1}{2}''$ to $3''$ when the web leg is welded and the outstanding leg is bolted to the supporting member (ref. p. 10-12 of the AISC Steel Manual).
- The $4''$ outstanding leg will usually accommodate the standard gages for the bolts used in the supporting members.
  - The length of the outstanding leg to the supporting member may be optionally reduced from $4''$ to $3''$ when the outstanding leg is welded and the connected leg is bolted to the beam web for values of $L$ from $5\frac{1}{2}''$ to $17\frac{1}{2}''$ (ref. p. 10-12 of the AISC Steel Manual).

The minimum angle thickness selected equals the weld size plus 1/16''.

The angle lengths are the same as those used for the non-staggered bolt cases.

- The recommended minimum angle length is taken as one-half of the T-dimension of the beam supported.
- The maximum angle length must be no greater than the T-dimension.

To select a connection of this type, the designer picks a weld size from Table 10-2 for the connection to the beam web and then goes to Table 10-1 to determine the number of bolts required for the connection to the supporting member.
Example Problem – Double-Angle Connections: Welded-Bolted

Example
Given: Double-angle beam connection welded to a W30 x 90 beam and then bolted to a W12 x 58 column.
Service loads: \( R_D = 80 \) kips, \( R_L = 70 \) kips
Steel (angles): \( F_Y = 36 \) ksi, \( F_U = 58 \) ksi
Bolts: \( \frac{3}{4}'' \) A325-N in standard size holes
Electrode: E70

Find: Select the connection using LRFD and ASD.

Solution

\[ W30 \times 90 \ (t_w = 0.470'', \ T = 26\frac{1}{2}'') \quad W12 \times 58 \ (t_f = 0.640'') \]

LRFD
\[ R_u = 1.2 \ D + 1.6 \ L = 1.2 \ (80) + 1.6 \ (70) = 208 \text{ kips} \]

Determine the weld size, angle length, and the number of rows of bolts using AISC Table 10-2 of Part 10 of the AISC Manual (p. 10-46).

- The minimum angle length = \( \frac{1}{2} \ T = 0.5(26.5) = 13.25'' \)

Possible selections for Weld A.

<table>
<thead>
<tr>
<th>Weld size</th>
<th>Angle length</th>
<th>LRFD design strength</th>
<th>Minimum web thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4&quot;</td>
<td>14(\frac{1}{2})&quot; &lt; T = 26(\frac{1}{2})&quot; OK</td>
<td>229 k &gt; 208 k OK</td>
<td>0.381&quot; &lt; 0.470&quot; OK</td>
</tr>
<tr>
<td>1/4&quot;</td>
<td>17(\frac{1}{4})&quot; &lt; T = 26(\frac{1}{2})&quot; OK</td>
<td>267 k &gt; 208 k OK</td>
<td>0.381&quot; &lt; 0.470&quot; OK</td>
</tr>
<tr>
<td>3/16&quot;</td>
<td>20(\frac{1}{2})&quot; &lt; T = 26(\frac{1}{2})&quot; OK</td>
<td>227 k &gt; 208 k OK</td>
<td>0.286&quot; &lt; 0.470&quot; OK</td>
</tr>
<tr>
<td>3/16&quot;</td>
<td>23(\frac{1}{2})&quot; &lt; T = 26(\frac{1}{2})&quot; OK</td>
<td>253 k &gt; 208 k OK</td>
<td>0.286&quot; &lt; 0.470&quot; OK</td>
</tr>
</tbody>
</table>

Select 3/16" weld with an angle length of 20\(\frac{1}{2}\)".

The selected angle length of 20\(\frac{1}{2}\)" corresponds to a 7-row bolted connection (i.e. \( n = 7 \)).

Determine the angle thickness using Table 10-1 (Part 10 of the AISC Manual) with \( \frac{3}{4}'' \) A325-N bolts (p. 10-18).
- A325-N bolts are defined as Group A in Specification Section J3.1.
• The selected angle length of 20½" corresponds to a 7-row bolted connection.
  - If the minimum angle thickness is 1/4" (i.e. weld size + 1/16"),
    the LRFD design strength = 174 kips <  R_u = 208 kips NG
  - If the angle thickness is 5/16",
    the LRFD design strength = 217 kips >  R_u = 208 kips OK

Check the bearing strength of the supporting column flange.
• The tabulated LRFD "support available strength per inch thickness" for standard bolt holes (STD) is 1230 kips/inch.
  Bearing strength of the column flange:
  \[ \varphi R_n = 1230 \times (0.640) = 787.2 \text{ kips} > R_u = 208 \text{ kips} \text{ OK} \]

Select 2L's 4 x 3½ x 5/16" x 20½" long (A36), connected to the beam with 3/16" weld (E70XX) and fitted with holes for seven ¾" A325-N bolts.

\( R_a = D + L = 80 + 70 = 150 \text{ kips} \)

Determine the weld size, angle length, and the number of rows of bolts using AISC Table 10-2 of Part 10 of the AISC Manual (p. 10-46).
• The minimum angle length = \( \frac{1}{2} T = 0.5(26.5) = 13.25" \)

Possible selections for Weld A.

<table>
<thead>
<tr>
<th>Weld size</th>
<th>Angle length</th>
<th>ASD allowable strength</th>
<th>Minimum web thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4&quot;</td>
<td>14½&quot; &lt; T = 26½&quot; OK</td>
<td>153 k &gt; 150 k OK</td>
<td>0.381&quot; &lt; 0.470&quot; OK</td>
</tr>
<tr>
<td>1/4&quot;</td>
<td>17½&quot; &lt; T = 26½&quot; OK</td>
<td>178 k &gt; 150 k OK</td>
<td>0.381&quot; &lt; 0.470&quot; OK</td>
</tr>
<tr>
<td>3/16&quot;</td>
<td>20½&quot; &lt; T = 26½&quot; OK</td>
<td>152 k &gt; 150 k OK</td>
<td>0.286&quot; &lt; 0.470&quot; OK</td>
</tr>
<tr>
<td>3/16&quot;</td>
<td>23½&quot; &lt; T = 26½&quot; OK</td>
<td>169 k &gt; 150 k OK</td>
<td>0.286&quot; &lt; 0.470&quot; OK</td>
</tr>
</tbody>
</table>

Select 3/16" weld with an angle length of 20½" (same as before).

The selected angle length of 20½" corresponds to a 7-row bolted connection (i.e. \( n = 7 \)).

Determine the angle thickness using Table 10-1 (Part 10 of the AISC Manual) with ¾" A325-N bolts (p. 10-18).
• A325-N bolts are defined as Group A in Specification Section J3.1.
- The selected angle length of 20\(\frac{1}{2}\)" corresponds to a 7-row bolted connection.
  - If the angle thickness is 1/4" (i.e. weld size + 1/16"),
    the ASD allowable strength = 116 kips < \(R_a = 150\) kips NG
  - If the angle thickness is 5/16",
    the ASD allowable strength = 145 kips < \(R_a = 150\) kips NG
  - If the angle thickness is 3/8",
    the ASD allowable strength = 167 kips > \(R_a = 150\) kips OK

Check the bearing strength of the supporting column flange.
- The tabulated ASD "support available strength per inch thickness" for standard bolt holes (STD) is 819 kips/inch.
  Bearing strength of the column flange:
  \[\frac{R_n}{\Omega} = 819 \times 0.640 = 524.2\text{ kip} > R_a = 150\text{ kips} \quad \text{OK}\]

Select 2L's 4 x 3\(\frac{1}{2}\) x 3/8" x 20\(\frac{1}{2}\)" long (A36), connected to the beam with 3/16" weld (E70XX) and fitted with holes for seven \(\frac{3}{8}\)" A325-N bolts.
11.5 Double-Angle Connections: Bolted-Welded

AISC Table 10-2 in Part 10 of the AISC Manual may also be used for the design of connections where angles are bolted to the beam web and welded to the supporting member.

Additional design notes for AISC Table 10-2

Tabulated values for the strength of Weld B consider the additional limit state of shear rupture strength of the welds on the outstanding legs of the angles.

The tabulated minimum thicknesses of the supported beam web for Welds A and the support for Welds B match the shear rupture strength of these elements with the strength of the weld metal.

• When Welds B line up on opposite sides of the support (e.g. opposite sides of the web of a girder), the minimum thickness is the sum of the thicknesses required for each weld.

• When less than the minimum material thickness is present, the tabulated weld available strength must be reduced by the ratio of the thickness provided to the minimum thickness.

For the usual situations, 4 x 3½ angles are used.

• The 3½" connected leg is connected to the beam web by bolting and the 4" outstanding leg is connected to the other member by welding.
  - The length of the outstanding leg to the supporting member may be optionally reduced from 4” to 3” when the outstanding leg is welded and the connected leg is bolted to the beam web for values of L from 5½” to 17½” (ref. p. 10-12 of the AISC Steel Manual).

The maximum weld size selected equals the angle thickness less 1/16”.

To select a connection of this type, the designer goes to Table 10-1 to determine the number of bolts required for the connection to the beam web and then determines the weld size from Table 10-2 for the connection to the supporting member.
Example Problem – Double-Angle Connections: Bolted-Welded

Example (This is a repeat of the previous example.)

Given: Double-angle beam connection bolted to a W30 x 90 beam and then welded to a W12 x 58 column.
Service loads: $R_D = 80$ kips, $R_L = 70$ kips
Steel (angles): $F_y = 36$ ksi, $F_u = 58$ ksi
Bolts: $\frac{3}{4}''$ A325-N in standard size holes
Electrode: E70

Find: Select the connection using LRFD and ASD.

Solution

$W30 \times 90 \ (t_w = 0.470'', \ T = 26\frac{1}{2}'') \quad W12 \times 58 \ (t_f = 0.640'')$

**LRFD**

$R_u = 1.2 \ D + 1.6 \ L = 1.2 \ (80) + 1.6 \ (70) = 208$ kips

Determine the number of bolts, the angle thickness, and the angle length using AISC Table 10-1 (Part 10 of the AISC Manual).

- Start with the least number of rows (i.e. 5 rows) that can be used with a W30 section and $\frac{3}{4}''$ A325-N bolts (p. 10-20).
  - A325-N bolts are defined as Group A in Specification Section J3.1.
    The “bolt and angle available strength” $< 208$ kips reaction for all 4 of the angle thicknesses listed. Thus, a 5-row connection is not adequate.

- Move to a 6-row connection for a W30 section and $\frac{3}{4}''$ A325-N bolts (p. 10-19).
  Try a connection with an angle thickness of $3/8''$.
  - The bolt and angle available strength: $\varphi R_n = 215$ kips $> R_u = 208$ kips \quad OK

Check the bearing strength of the beam web.

- The tabulated LRFD “beam web available strength per inch thickness” for standard bolt holes (STD), an uncoped beam, and a horizontal edge distance $L_{eh} = 1\frac{3}{4}''$ is 527 kips/inch.
  - Bearing strength of the beam web: $\varphi R_n = 527 \ (0.470) = 247.7$ kips $> R_u = 208$ kips \quad OK
Check the angle length.
- Angle length = 17½” < T = 26.5” OK

Determine the weld size (Weld B) using Table 10-2.
- The maximum weld size is 5/16” (i.e. angle thickness minus 1/16”).
- The selected angle length is 17½”.

Possible selections for Weld B.

<table>
<thead>
<tr>
<th>Weld size</th>
<th>LRFD design strength</th>
<th>Minimum thickness of connecting element</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4” &lt; 5/16” OK</td>
<td>150 k &lt; 208 k NG</td>
<td>0.190” &lt; 0.640” OK</td>
</tr>
<tr>
<td>5/16” OK</td>
<td>188 k &lt; 208 k NG</td>
<td>0.238” &lt; 0.640” OK</td>
</tr>
<tr>
<td>3/8” &gt; 5/16” NG</td>
<td>226 k &gt; 208 k OK</td>
<td>0.286” &lt; 0.640” OK</td>
</tr>
</tbody>
</table>

Angle length or thickness is not adequate.

Select a thicker angle using AISC Table 10-1 (Part 10 of the AISC Manual).
- Try a 6-row connection with an angle thickness of 1/2”.
  The bolt and angle available strength: \( \varphi R_n = 215 \text{ kips} > R_u = 208 \text{ kips} \) OK

Determine the weld size (Weld B) using Table 10-2.
- The maximum weld size is 7/16” (i.e. angle thickness minus 1/16”).
- The selected angle length is 17½”.

Possible selections for Weld B.

<table>
<thead>
<tr>
<th>Weld size</th>
<th>LRFD design strength</th>
<th>Minimum thickness of connecting element</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4” &lt; 7/16” OK</td>
<td>150 k &lt; 208 k NG</td>
<td>0.190” &lt; 0.640” OK</td>
</tr>
<tr>
<td>5/16” &lt; 7/16 OK</td>
<td>188 k &lt; 208 k NG</td>
<td>0.238” &lt; 0.640” OK</td>
</tr>
<tr>
<td>3/8” &lt; 7/16” OK</td>
<td>226 k &gt; 208 k OK</td>
<td>0.286” &lt; 0.640” OK</td>
</tr>
</tbody>
</table>

Angle length and thickness are adequate.

Select 3/8” Weld B with an angle length of 17½”.

Use 4 x 3½ angles (per AISC Manual, p. 10-12).

Select 2L’s 4 x 3½ x ½ x 17½” long (A36), connected to the column with 3/8” weld (E70XX) and connected to the beam web using six ¾” A325-N bolts.
ASD
\[ R_a = D + L = 80 + 70 = 150 \text{ kips} \]

Determine the number of bolts, the angle thickness, and the angle length using AISC Table 10-1 (Part 10 of the AISC Manual).

- Start with the least number of rows (i.e. 5 rows) that can be used with a W30 section and \( \frac{3}{4}" \) A325-N bolts (p. 10-20).
  - A325-N bolts are defined as Group A in Specification Section J3.1. The "bolt and angle available strength" < 150 kips reaction for all 4 of the angle thicknesses listed. Thus, a 5-row connection is not adequate.

- Move to a 6-row connection for a W30 section and \( \frac{3}{4}" \) A325-N bolts (p. 10-19). The "bolt and angle available strength" < 150 kips reaction for all 4 of the angle thicknesses listed. Thus, a 6-row connection is not adequate.

- Move to a 7-row connection for a W30 section and \( \frac{3}{4}" \) A325-N bolts (p. 10-18). Try a connection with an angle thickness of 3/8". The bolt and angle available strength: \( R_a = 150 \text{ kips} \) OK

Check the bearing strength of the beam web.

- The tabulated ASD "beam web available strength per inch thickness" for standard bolt holes (STD), an uncoped beam, and a horizontal edge distance \( L_{eh} = 1\frac{3}{4}" \) is 410 kips/inch.
  
  Bearing strength of the beam web:
  \[ R_n/\Omega = 410 (0.470) = 192.7 \text{ kips} > R_a = 150 \text{ kips} \text{ OK} \]

Check the angle length.

- Angle length = 20\( \frac{1}{2} " \) < T = 26.5" OK

Determine the weld size (Weld B) using Table 10-2.

- The maximum weld size is 5/16" (i.e. angle thickness minus 1/16").

- The selected angle length is 20\( \frac{1}{2} " \).

Possible selections for Weld B.

<table>
<thead>
<tr>
<th>Weld size</th>
<th>ASD allowable strength</th>
<th>Minimum thickness of connecting element</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4&quot; &lt; 5/16&quot; OK</td>
<td>125 k &lt; 150 k NG</td>
<td>0.190&quot; &lt; 0.640&quot; OK</td>
</tr>
<tr>
<td>5/16&quot; OK</td>
<td>156 k &gt; 150 k OK</td>
<td>0.238&quot; &lt; 0.640&quot; OK</td>
</tr>
<tr>
<td>3/8&quot; &gt; 5/16&quot; NG</td>
<td>187 k &gt; 150 k OK</td>
<td>0.286&quot; &lt; 0.640&quot; OK</td>
</tr>
</tbody>
</table>

Angle length and thickness are adequate.
Select 5/16" Weld B with an angle length of 20\(\frac{1}{2}\)^".

Use 4 x 3\(\frac{1}{2}\) angles (per AISC Manual, p. 10-12).

Select 2L's 4 x 3\(\frac{1}{2}\) x 3/8 x 20\(\frac{1}{2}\)^" long (A36), connected to the column with 5/16" weld (E70XX) and connected to the beam web using seven \(\frac{3}{4}\)^" A325-N bolts.
11.6 Double-Angle Connections: Welded-Welded

AISC Table 10-3 in Part 10 of the AISC Manual provides weld values if the double angles are welded to both members.

- The weld used to connect the angles to the beam web is called Weld A.
- The weld used to connect the angles to the other member is called Weld B.
- The weld strength values in Table 10-3 are based on the E70 electrode.

The double-angle welded-welded connection is not a particularly common connection because it requires field welding.

Design notes for AISC Table 10-3

The tabulated minimum thicknesses of the supported beam web for Welds A and the support for Welds B match the shear rupture strength of these elements with the strength of the weld metal.

- When Welds B line up on opposite sides of the support (e.g. opposite sides of the web of a girder), the minimum thickness is the sum of the thicknesses required for each weld.
- When less than the minimum material thickness is present, the tabulated weld available strength must be reduced by the ratio of the thickness provided to the minimum thickness.

Use 2 L's 4 x 3 3/8 when the angle length ≥ 18"; otherwise use 2 L's 3 x 3 1/2 (ref. p. 10-12 of the AISC Manual)

The minimum angle thickness selected equals the weld size plus 1/16".

The angle lengths are those as tabulated in Table 10-3.
Example Problem – Double-Angle Connections: Welded-Welded

Example

Given: All-welded double-angle framed simple end connection for fastening a W30 x 108 beam to the flange of a W14 x 61 column.

Service loads:
\[ R_D = 60 \text{ kips}, \ R_L = 80 \text{ kips} \]

Steel (beam and column):
\[ F_Y = 50 \text{ ksi}, \ F_U = 65 \text{ ksi} \]

Steel (angles):
\[ \text{A36} \ (F_Y = 36 \text{ ksi}, \ F_U = 58 \text{ ksi}) \]

Electrode: E70

Find: Select the connection using LRFD and ASD.

Solution

W30 x 108 \((t_w = 0.545\text{"}, \ T = 26\frac{1}{2}\text{"})\)

W14 x 61 \((t_f = 0.645\text{")}\)

LRFD
\[ R_u = 1.2 \ D + 1.6 \ L = 1.2 (60) + 1.6 (80) = 200 \text{ kips} \]

Determine the weld size and angle length using AISC Table 10-3 of Part 10 of the AISC Manual (pp. 10-46 and 10-47).

Possible selections for Weld A and Weld B.

<table>
<thead>
<tr>
<th>Weld sizes</th>
<th>Angle length</th>
<th>LRFD design strength</th>
<th>Minimum web thickness</th>
</tr>
</thead>
</table>
| A: 1/4"  
B: 3/8" | 16" < T = 26\frac{1}{2}" OK | Weld A: 248 k > 200 k OK 
Weld B: 222 k > 200 k OK | 0.381" < 0.545" OK 
0.286" < 0.645" OK |
| A: 3/16"  
B: 3/8" | 18" < T = 26\frac{1}{2}" OK | Weld A: 205 k > 200 k OK 
Weld B: 235 k > 200 k OK | 0.286" < 0.545" OK 
0.286" < 0.645" OK |
| A: 3/16"  
B: 5/16" | 20" < T = 26\frac{1}{2}" OK | Weld A: 223 k > 200 k OK 
Weld B: 226 k > 200 k OK | 0.286" < 0.545" OK 
0.238" < 0.645" OK |
| A: 3/16"  
B: 1/4" | 22" < T = 26\frac{1}{2}" OK | Weld A: 240 k > 200 k OK 
Weld B: 205 k > 200 k OK | 0.286" < 0.545" OK 
0.190" < 0.645" OK |
A 14" long angle was not considered since the tabulated LRFD design strengths for all sizes of Weld B are less than 200 kips.

Select 3/16" Weld A and 5/16" Weld B with an angle length of 20".

Use 4 x 3 1/2 angles since the angle length is greater than 18".

Minimum angle thickness = larger weld size + 1/16" = 5/16" + 1/16" = 3/8"

Select 2L's 4 x 3 1/2 x 3/8" x 20" long (A36), connected to the beam with 3/16" weld (E70XX) and connected to the column with 5/16" weld (E70XX).

ASD
R_a = D + L = 60 + 80 = 140 kips

Determine the weld size and angle length using AISC Table 10-3 of Part 10 of the AISC Manual (pp. 10-46 and 10-47).

Possible selections for Weld A and Weld B.

<table>
<thead>
<tr>
<th>Weld size</th>
<th>Angle length</th>
<th>ASD allowable strength</th>
<th>Minimum web thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>A: 1/4&quot; B: 3/8&quot;</td>
<td>16&quot; &lt; T = 26 1/2&quot; OK</td>
<td>Weld A: 166 k &gt; 140 k OK Weld B: 148 k &gt; 140 k OK</td>
<td>0.381&quot; &lt; 0.545&quot; OK 0.286&quot; &lt; 0.645&quot; OK</td>
</tr>
<tr>
<td>A: 1/4&quot; B: 3/8&quot;</td>
<td>18&quot; &lt; T = 26 1/2&quot; OK</td>
<td>Weld A: 182 k &gt; 140 k OK Weld B: 157 k &gt; 140 k OK</td>
<td>0.381&quot; &lt; 0.545&quot; OK 0.286&quot; &lt; 0.645&quot; OK</td>
</tr>
<tr>
<td>A: 3/16&quot; B: 5/16&quot;</td>
<td>20&quot; &lt; T = 26 1/2&quot; OK</td>
<td>Weld A: 149 k &gt; 140 k OK Weld B: 151 k &gt; 140 k OK</td>
<td>0.286&quot; &lt; 0.545&quot; OK 0.238&quot; &lt; 0.645&quot; OK</td>
</tr>
<tr>
<td>A: 3/16&quot; B: 5/16&quot;</td>
<td>22&quot; &lt; T = 26 1/2&quot; OK</td>
<td>Weld A: 160 k &gt; 140 k OK Weld B: 171 k &gt; 140 k OK</td>
<td>0.286&quot; &lt; 0.545&quot; OK 0.238&quot; &lt; 0.645&quot; OK</td>
</tr>
</tbody>
</table>

A 14" long angle was not considered since the tabulated ASD allowable strengths for all sizes of Weld B are less than 140 kips.

Select 3/16" Weld A and 5/16" Weld B with an angle length of 20".

Use 4 x 3 1/2 angles since the angle length is greater than 18".

Minimum angle thickness = larger weld size + 1/16" = 5/16" + 1/16" = 3/8"

Select 2L's 4 x 3 1/2 x 3/8" x 20" long (A36), connected to the beam with 3/16" weld (E70XX) and connected to the column with 5/16" weld (E70XX).
11.7 Single-Angle Connections

Single-angle connections may be performed as a bolted-bolted or as a bolted-welded connection (ref. Figure 11.1b, p. 398 of the textbook).

• The single-angle connection is an efficient connection because of the fewer parts than the double-angle connection.

• The disadvantages of the single-angle connection are that the components (i.e. angles, bolts, and welds) are larger than for the double angle connection.
  - For this connection, the bolts in the beam web are in single shear.
    o If the controlling limit state were bolt shear, then the single-angle connection would require twice as many bolts as a double-angle connection.
  - Since the beam force must pass through only one angle, the angle will likely be larger.
  - Greater weld size and weld lengths are also required.

The limit states to be checked for the single-angle connection are the same as those for the double-angle connection, with some modifications and additions.

• The most significant modifications involve the eccentricities induced in the connecting elements.
  - For the connection to the supported beam, as long as there is only one row of bolts, no eccentricities are considered, and this portion of the connection is treated the same as for the double angle connections.
  - For the outstanding leg, the bolts or welds must be designed to account for the connection eccentricity.
  - This eccentricity adds the limit states of flexural yielding and flexural rupture for the outstanding leg of the angle.

11.8 Single-Plate Shear Connections

An economical and simple type of flexible connection for light loads is the single-plate shear connection, also known as a shear tab connection (ref. Figure 11.1c, p. 398 of the textbook).

• Bolt holes are pre-punched in the plate and the web of the beam.
• The plate is shop-welded to the supporting beam or column.
• The beam web is bolted to the single plate in the field.
The behavior of this connection is similar to that of a double-angle connection except that it achieves its rotation capacity through the bending of the tab and deformation of the plate or beam web in bearing at the bolt holes.

Limit states that must be checked for the single-plate shear connection include the following.

- For the bolts: shear rupture.
- For the beam: bearing on the web, and shear yielding of the web.
- For the plate (tab): bearing on the plate, elastic yield moment, shear yield, shear rupture, block shear rupture, buckling, and plastic flexural yielding with shear interaction.
- For the weld: weld rupture with eccentricity.

AISC Table 10-10 is a design aid for single-plate connections welded to the support and bolted to the supported beam.

- Table 10-10a tabulates available strengths for plate material with $F_y = 36$ ksi.
- Table 10-10b tabulates available strengths for plate material with $F_y = 50$ ksi.
Example Problem - Single-Plate Shear Connection

Example

Given: The single-plate shear connection for the W16 x 50 beam to the W14 x 90 column shown.
- Service loads: \( R_D = 15 \) kips, \( R_L = 20 \) kips
- Bolts: \( \frac{3}{4}'' \) A325-N
- Electrode: E70
- Steel (beam and column): A992
  - \((F_Y = 50 \text{ kips, } F_U = 65 \text{ ksi})\)
- Steel (plate): A36
  - \((F_Y = 36 \text{ ksi, } F_U = 58 \text{ ksi})\)

Find: Design the single-plate connection.

Solution

W16 x 50 \((t_w = 0.380'' , T = 13-5/8'')\)

LRFD

\[ R_u = 1.2 \; D + 1.6 \; L = 1.2 \; (15) + 1.6 \; (20) = 50 \text{ kips} \]

Determine the plate length, plate thickness, weld size, and the number of rows of bolts using AISC Table 10-10a of Part 10 of the AISC Manual (pp. 10-108 to 10-110).

- A325-N bolts are defined as Group A in Specification Section J3.1.
- The minimum plate length \( \frac{1}{2} \; T = 0.5 \; (13.625) = 6.81'' \)

Possible selections

<table>
<thead>
<tr>
<th>( n )</th>
<th>Plate length</th>
<th>Plate thickness</th>
<th>Weld size</th>
<th>LRFD design strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>11(\frac{1}{2}'' ) (&lt;\ 13-5/8'' ) OK</td>
<td>1/4''</td>
<td>3/16''</td>
<td>52.2 kips (&gt;\ 50) kips OK</td>
</tr>
<tr>
<td>5</td>
<td>14(\frac{1}{2}'' ) (&gt;\ 13-5/8'' ) NG</td>
<td>1/4''</td>
<td>3/16''</td>
<td>65.2 kips (&gt;\ 50) kips OK</td>
</tr>
</tbody>
</table>

An 8\(\frac{1}{2}'' \) long plate was not considered since the tabulated LRFD allowable strengths for all plate thicknesses and weld sizes are less than 50 kips.

Select plate 1/4'' thick, 11\(\frac{1}{2}'' \) long, with 4 rows of bolts, using 3/16'' weld.
Check the supported beam web available strength using AISC Table 10-1 of Part 10 of the AISC Manual (p. 10-21).

Minimum vertical edge distance: \( L_{ev} = 1'' \) (Use 1\( \frac{1}{4}'' \))  
AISC Table J3.4

Min. horizontal edge distance: \( L_{eh} \geq 2 \times d_b = 1\frac{7}{8}'' \) (ref. AISC Manual, p. 10-103)

Beam web LRFD available strength = 351 kips/inch thickness of beam web
= 351 kips/inch \((0.380'') = 133.4 \text{ kips} > R_u = 50 \text{ kips} \quad \text{OK}

Check the plate or beam web thickness (single vertical row of 4 bolts).

Maximum \( t_p \) or \( t_w \leq d_b/2 + 1/16'' \)  
AISC Table 10-9

Maximum \( t_p \) or \( t_w \leq (0.75)/2 + 1/16 = 0.438''

Both the beam web \((t_w = 0.380'')\) and the plate thickness \((1/4'') < 0.438'' \quad \text{OK}

Determine the plate width (ref. Figure 10-11, AISC Manual, p. 10-102).

Plate width = \( a + L_{eh} = 3.5 + 2(0.75) = 5.0'' \)

where

\[
\begin{align*}
    a & \leq 3\frac{1}{2}'' \quad \text{(ref. AISC Manual, p. 10-102)} \\
    L_{eh} & \geq 2d_b \quad \text{(ref. AISC Manual, p. 10-103)}
\end{align*}
\]

Select PL \( \frac{1}{4}'' \times 5'' \times 11\frac{1}{2}'' \) long (A36), connected to the column with 3/16" weld (E70XX) and fitted with holes for four \( \frac{1}{2}'' \) A325-N bolts.

**ASD**

\[ R_a = D + L = 15 + 20 = 35 \text{ kips} \]

Determine the plate length, plate thickness, weld size, and the number of rows of bolts using AISC Table 10-10a of Part 10 of the AISC Manual (pp. 10-108 to 10-110).

- A325-N bolts are defined as Group A in Specification Section J3.1.
- The minimum plate length = \( \frac{1}{2} \times T = 0.5 \times 13.625 = 6.81'' \)

\[
\begin{array}{|c|c|c|c|c|}
\hline
n & Plate length & Plate thickness & Weld size & ASD allowable strength \\
\hline
4 & 11\frac{1}{2}'' \times 13-5/8'' & OK & 5/16'' & 1/4'' & 41.5 \text{kips} > 35 \text{kips} \quad \text{OK} \\
\hline
5 & 14\frac{1}{2}'' > 13-5/8'' & NG & 1/4'' & 3/16'' & 43.5 \text{kips} > 35 \text{kips} \quad \text{OK} \\
\hline
\end{array}
\]

An 8\( \frac{1}{2}'' \) long plate was not considered since the tabulated ASD allowable strengths for all plate thicknesses and weld sizes are less than 35 kips.

Select plate 5/16'' thick, 11\( \frac{1}{2}'' \) long, with 4 rows of bolts, using 1/4'' weld.
Check the supported beam web available strength using AISC Table 10-1 of Part 10 of the AISC Manual (p. 10-21).

Minimum vertical edge distance: \( L_{ev} = 1" \) (Use 1\(\frac{1}{4}"\))

Min. horizontal edge distance: \( L_{eh} \geq 2 \times d_b = 1\frac{1}{2}" \) (ref. AISC Manual, p. 10-103)

Beam web ASD available strength = 234 kips/inch thickness of beam web

\[ = 234 \text{ kips/inch (0.380")} = 88.9 \text{ kips} > R_a = 35 \text{ kips} \quad \text{OK} \]

Check the plate or beam web thickness (single vertical row of 4 bolts).

Maximum \( t_p \) or \( t_w \leq d_b/2 + 1/16" \) (ref. AISC Manual, p. 10-103)

Maximum \( t_p \) or \( t_w \leq (0.75)/2 + 1/16 = 0.438" \)

Both the beam web (\( t_w = 0.380" \)) and the plate thickness (5/16") < 0.438" \quad \text{OK}

Determine the plate width (ref. AISC Manual, pp. 10-102 to 10-103).

Plate width = \( a + L_{eh} = 3.5 + 2(0.75) = 5.0" \)

where

\[ a \leq 3\frac{1}{2}" \quad \text{(ref. AISC Manual, p. 10-102)} \]

\[ L_{eh} \geq 2d_b \quad \text{(ref. AISC Manual, p. 10-103)} \]

Select PL 5/16" x 5" x 11\(\frac{1}{2}" \) long (A36), connected to the column with 1/4" weld (E70XX) and fitted with holes for four \( \frac{3}{4}" \) A325-N bolts.
11.9 Seated Connections

Another type of flexible beam connection is the beam seat connection.

- An unstiffened beam seat is shown in Figure 11.1d (p. 398 of the textbook).
- A stiffened beam seat is shown in Figure 11.1e (p. 398 of the textbook).

These connections are used to attach a beam to the flange of a column.

- Beam seats may be used to add capacity to other types of existing connections in a retrofit situation.
- Beam seats offer an advantage to the worker performing the steel construction by providing immediate support for the beam.
- The seated connection performs well as a simple connection.
  - The beam seat can rotate sufficiently about the bottom of the beam without imposing any significant moment to the supporting member.

The fasteners used with the angles to make these connections may be bolts or welds.

- For a welded type beam seat, the seat angle is usually shop-welded to the column and field welded to the beam.
- The seat angles may be punched for assembly bolts.
  - The holes may be slotted to allow easy alignment of the members.

If the connection is to the web of a girder, the maximum length of the connected leg can be determined as follows.

\[ L_{\text{max}} = d_G - d_B - 2k_G \]

where

- \( d_G \) = the depth of the girder
- \( d_B \) = the depth of the beam
- \( k_G \) = the distance from the outer face of the flange to the web toe of the fillet of the girder
A seated connection may only be used when a top angle is provided.

- The top angle (commonly a 4 x 4 x ¼) provides lateral support for the beam whether placed on top of the beam or at the optional location on the side of the beam.
- Flexible angles are used that bend away from the column or girder to which they are connected when the beam tends to rotate under load.

Unstiffened seated beam connections can support only light loads.

- For such light loads, two vertical end welds on the seat are sufficient.
- The top angle is welded only on its toes, so when the beam tends to rotate, this thin flexible angle will be free to pull away from the column.

The simplicity of this connection results in relatively few limit states to be checked.

- Because the transfer of force between the beam and the seat is through bearing of the beam on the seat, the limit states of web yielding and beam web crippling must be checked.
- The outstanding leg of the seat angle must be checked for the limit states of flexural yielding and shear yielding.
- The fasteners (bolts or welds) used in the connection to the supporting member must be checked for their appropriate limit states.

AISC Tables 10-5 and 10-6 may be used in combination to design unstiffened seated connections that are welded to the supporting member and bolted to the supported beam, or bolted to the supporting member and welded to the supported beam.

- Table 10-5 is a design aid for all-bolted unstiffened seated connections.
- Table 10-6 is a design aid for all-welded unstiffened seated connections.

AISC Tables 10-7 and 10-8 may be used to design stiffened seated connections that are welded to the supporting member and bolted to the supported beam, or bolted to the supporting member and welded to the supported beam.

- Table 10-7 is a design aid for all-bolted stiffened seated connections.
- Table 10-8 is a design aid for bolted/welded stiffened seated connections.

Design notes for AISC Tables 10-5 through 10-8

- The seat design strengths given in the AISC Manual tables were developed for seat angles with either 3½” or 4” outstanding legs.
• The steel used for the angles is A36 ($F_y = 36$ ksi and $F_u = 58$ ksi).

• The values in the tables were calculated based on a 3/4" setback rather than the nominal 1/2" setback used for web framing angles (ref. p. 10-85 of the AISC Manual).
Example Problem - All-welded Unstiffened Seated Connection

Example

Given: An all-welded unstiffened seated connection for a W24 x 55 beam connected to a W14 x 68 column.

Electrode: E70

Service loads: \( R_D = 20 \text{ kips}, R_L = 30 \text{ kips} \)

Steel (angles): A36 (\( F_Y = 36 \text{ ksi}, F_U = 58 \text{ ksi} \))

Steel (beam and column): \( F_Y = 50 \text{ ksi}, F_U = 65 \text{ ksi} \)

Find: Design the seat angle and welds.

Solution

\[
W24 \times 55 \text{ (} d = 23.6", t_w = 0.395", b_f = 7.01", t_f = 0.505", k = 1.01") \\
W14 \times 68 \text{ (} b_f = 10.0", t_f = 0.720"\text{)}
\]

**LRFD**

\( R_u = 1.2 \ D + 1.6 \ L = 1.2 \ (20) + 1.6 \ (30) = 72.0 \text{ kips} \)

**Check local web yielding**

Assume \( l_b = 3 \frac{1}{2} " \) for the outstanding leg.

Determine the required length of bearing \( l_b \) based on AISC Equation J10-3 and simplified using the following equation (ref. AISC Manual, p. 9-20).

\[
\varphi R_n = \varphi R_1 + l_b (\varphi R_2) \quad \text{Equation 9-45a}
\]

From AISC Table 9-4: \( \varphi R_1 = 49.9 \) and \( \varphi R_2 = 19.8 \) for W24 x 55

\[
\varphi R_n = 72.0 = 49.9 + l_b (19.8) \\
\text{Required } l_b = 1.12" < l_b = 3 \frac{1}{2} " \quad \text{OK}
\]

**Check web crippling**

\[ l_b/d = 3.5/23.6 = 0.148 < 0.20 \]

Determine the required length of bearing \( l_b \) based on AISC Equation J10-5a and simplified using the following equation (AISC Manual, p. 9-20).

\[
\varphi R_n = \varphi R_3 + l_b (\varphi R_4) \quad \text{Equation 9-47a}
\]

From AISC Table 9-4: \( \varphi R_3 = 63.7 \) and \( \varphi R_4 = 5.60 \) for W24 x 55
\[ \varphi R_n = 72.0 = 63.7 + l_b \times (5.60) \]

Required \( l_b = 1.48" \) (controls) < \( l_b = 3\frac{1}{2}" \) \( \text{OK} \)

Determine the seat angle size, angle thickness, and weld size.

- Using the lower portion of AISC Table 10-6 (pp. 10-91 to 10-92), the possible selections for "weld available strength" are as follows.

<table>
<thead>
<tr>
<th>Weld size</th>
<th>Seat angle size</th>
<th>LRFD weld available strength</th>
<th>Available angle thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
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<tr>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Select the 8 x 4 angle, 6" or 8" long.

- Then, using the upper portion of AISC Table 10-6, with a required bearing length of \( l_b = 1.48" \) (say 1.5"):

  The required angle thickness is 3/4" for an angle length of 6".
  LRFD outstanding angle leg length strength = 72.9 kips > \( R_u = 72.0 \) kips \( \text{OK} \)

  The required angle thickness is 3/4" for an angle length of 8".
  LRFD outstanding angle leg length strength = 97.2 kips > \( R_u = 72.0 \) kips \( \text{OK} \)

Select 8 x 4 x 3/4 angle, 8" long, connected to the column with 3/8" weld (E70XX).

  Note: Because the flange width \( (b_f) \) of the W24 x 55 is 7.01", the 8" long angle is selected since it provides full support for the beam. The 8" angle also fits within the flange width of the W14 x 68 column \( (b_f = 10.0") \).

\[ \text{ASD} \]
\[ R_a = D + L = 20 + 30 = 50.0 \text{ kips} \]

**Check local web yielding**

Assume \( l_b = 3\frac{1}{2}" \) for the outstanding leg.

Determine the required length of bearing \( l_b \) based on AISC Equation J10-3 and simplified using the following equation (AISC Manual, p. 9-20).

\[ R_n/\Omega = R_1/\Omega + l_b \times (R_2/\Omega) \]

Equation 9-45b

From AISC Table 9-4: \( R_1/\Omega = 33.2 \) and \( R_2/\Omega = 13.2 \) for W24 x 55

\[ R_n/\Omega = 50.0 = 33.2 + l_b \times (13.2) \]

11.30
Required \( l_b = 1.27" < l_b = 3\frac{1}{2}" \quad OK

Check web crippling
\( l_b/d = 3.5/23.6 = 0.148 < 0.20 \)

Determine the required length of bearing \( l_b \) based on AISC Equation J10-5a and simplified using the following equation (AISC Manual, p. 9-20).

\[
\frac{R_n}{\Omega} = \frac{R_3}{\Omega} + l_b \left( \frac{R_4}{\Omega} \right) \quad \text{Equation 9-47b}
\]

From AISC Table 9-4: \( R_3/\Omega = 42.5 \) and \( R_4/\Omega = 3.74 \) for \( W24 \times 55 \)

\[
\frac{R_n}{\Omega} = 50.0 = 42.5 + l_b (3.74)
\]

Required \( N = 2.01" \) (controls) \( l_b = 3\frac{1}{2}" \quad OK

Determine the seat angle size, angle thickness, and weld size.

- Using the lower portion of AISC Table 10-6 (pp. 10-91 to 10-92), the possible selections for “weld available strength” are as follows.

<table>
<thead>
<tr>
<th>Weld size</th>
<th>Seat angle size</th>
<th>ASD weld available strength</th>
<th>Available angle thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 x 3\frac{1}{2}</td>
<td>All &lt; 50 kips</td>
<td>NG</td>
<td></td>
</tr>
<tr>
<td>5 x 3\frac{1}{2}</td>
<td>All &lt; 50 kips</td>
<td>NG</td>
<td></td>
</tr>
<tr>
<td>5/8&quot;</td>
<td>6 x 4</td>
<td>54.5 kips &gt; 50 kips</td>
<td>3/8&quot; - 3/4&quot;</td>
</tr>
<tr>
<td>1/2&quot;</td>
<td>7 x 4</td>
<td>57.0 kips &gt; 50 kips</td>
<td>3/8&quot; - 3/4&quot;</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>8 x 4</td>
<td>53.4 kips &gt; 50 kips</td>
<td>1/2&quot; - 1&quot;</td>
</tr>
</tbody>
</table>

Select the 8 x 4 angle, 6" or 8" long.

- Then, using the upper portion of AISC Table 10-6, with a required bearing length of \( l_b = 2.01" \) (say 2.0"):

The required angle thickness is 1" for an angle length of 6".
ASD outstanding angle leg length strength = 86.2 kips > \( R_a = 50.0 \) kips \quad OK

The required angle thickness is 1" for an angle length of 8".
ASD outstanding angle leg length strength = 115.0 kips > \( R_a = 50.0 \) kips \quad OK

Select 8 x 4 x 1 angle, 8" long, connected to the column with 3/8" weld (E70XX).

Note: Because the flange width (b_f) of the W24 x 55 is 7.01", the 8" long angle is selected since it provides full support for the beam. The 8" angle also fits within the flange width of the W14 x 68 column (b_f = 10.0").
Example Problem - All-bolted Unstiffened Seated Connections

Given: An all-bolted unstiffened seated connection for a W24 x 55 beam connected to a W14 x 68 column.
Bolts: 3/4" - A325 N in standard size bolt holes
Service loads: \( R_D = 12 \) kips, \( R_L = 30 \) kips
Steel (angles): A36 (\( F_Y = 36 \) ksi, \( F_U = 58 \) ksi)
Steel (beam and column): \( F_Y = 50 \) ksi, \( F_U = 65 \) ksi

Find: Design the bolted seat angle connection.

Solution

\[
W24 \times 55 \ (d = 23.6\", \ t_w = 0.395\", \ b_f = 7.01\", \ t_f = 0.505\", \ k = 1.01")
W14 \times 68 \ (b_f = 10.0\", \ t_f = 0.720")
\]

**LRFD**

\[ R_u = 1.2 \ D + 1.6 \ L = 1.2 \ (12) + 1.6 \ (30) = 62.4 \text{ kips} \]

**Check local web yielding**

Assume \( l_b = 3 \frac{1}{2}" \) for the outstanding leg.

Determine the required length of bearing \( l_b \) based on AISC Equation J10-3 and simplified using the following equation (ref. AISC Manual, p. 9-20).

\[
\phi_R^n = \phi_R^1 + l_b \ (\phi_R^2)
\]

Equation 9-45a

From AISC Table 9-4: \( \phi_R^1 = 49.9 \) and \( \phi_R^2 = 19.8 \) for W24 x 55

\[
\phi_R^n = 62.4 = 49.9 + l_b \ (19.8)
\]

Required \( l_b = 0.63" \) (controls) < \( l_b = 3 \frac{1}{2}" \) OK

**Check web crippling**

\[
l_b/d = 3.5/23.6 = 0.148 < 0.20
\]

Determine the required length of bearing \( l_b \) based on AISC Equation J10-5a and simplified using the following equation (AISC Manual, p. 9-20).

\[
\phi_R^n = \phi_R^3 + l_b \ (\phi_R^4)
\]

Equation 9-47a

From AISC Table 9-4: \( \phi_R^3 = 63.7 \) and \( \phi_R^4 = 5.60 \) for W24 x 55

\[
\phi_R^n = 62.4 = 63.7 + l_b \ (5.60)
\]

Required \( l_b = -0.23" \) < \( l_b = 3 \frac{1}{2}" \) OK
Determine the seat angle type, seat angle size, and angle thickness.

- Using the lower portion of AISC Table 10-5 (pp. 10-89 to 10-90), the possible selections for "bolt available strength" for the given bolt diameter (3/4") and bolt type (A325-N) are as follows:
  - A325-N bolts are defined as Group A in Specification Section J3.1.

<table>
<thead>
<tr>
<th>Seat type</th>
<th>Seat angle size</th>
<th>LRFD bolt available strength</th>
<th>Available angle thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type A</td>
<td>6 x 4</td>
<td>35.8 kips &lt; 62.4 kips</td>
<td>NG</td>
</tr>
<tr>
<td></td>
<td>7 x 4</td>
<td>71.6 kips &gt; 62.4 kips</td>
<td>OK</td>
</tr>
<tr>
<td></td>
<td>8 x 4</td>
<td>107 kips &gt; 62.4 kips</td>
<td>OK</td>
</tr>
<tr>
<td>Type B</td>
<td>6 x 4</td>
<td>71.6 kips &gt; 62.4 kips</td>
<td>OK</td>
</tr>
<tr>
<td></td>
<td>3/8&quot; - 3/4&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3/8&quot; - 3/4&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1/2&quot; - 1&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type C</td>
<td>8 x 4</td>
<td>107 kips &gt; 62.4 kips</td>
<td>OK</td>
</tr>
<tr>
<td></td>
<td>1/2&quot; - 1&quot;</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: Type D, Type E, and Type F seated connections are not suitable for connections made with column flanges since the center row of bolts would conflict with the column web.

Select the Type B, 6 x 4 angle, 6" or 8" long.

Angles with a longer leg (such as the 7 x 4 or 8 x 4) may be selected, if necessary, if the required angle thickness exceeds 3/4", or to increase the bearing strength of the supporting member since the gage and thus the edge to edge bolt hole distance (i.e. l_c) would be greater.

- Then, using the upper portion of AISC Table 10-5, with a required bearing length of l_b = 0.63" (say 5/8"):
  - The maximum angle thickness is 1/2" for an angle length of 6".
    LRFD outstanding angle leg length strength = 64.8 kips > R_u = 62.4 kips OK
  - The required angle thickness is 1/2" with an angle length of 8".
    LRFD outstanding angle leg length strength = 86.4 kips > R_u = 62.4 kips OK

Finally, check the bearing strength of the supporting member (i.e. column flange).

- Applicable equation (assuming that deformation at the bolt hole at service load is a design consideration): R_n = 1.2 l_c t F_u ≤ 2.4 d t F_u  AISC Equation J3-6a

- Determine the value for l_c.

  Edge of hole to edge of plate:
  - The distance l_c for the vertical distance from the edge of the bolt hole to the end of the column is presumably large and can be disregarded.

  Edge-to-edge of holes:
  \[ l_c = g_2 - 2 \left( \frac{1}{2} \right) (\text{diameter of the bolt hole}) = 2.5 - 2 \left( \frac{1}{2} \right) (3/4 + 1/8) = 1.625" \]
where
\[ g_2 = \text{the gage distance between two horizontal lines of bolts in the vertical leg of the Type B angle seat (ref. AISC Manual, p. 1-48).} \]

\[ R_n = 1.2 \left( t \right) \left( F_u \right) = 1.2 \left( 1.625 \right) \left( 0.720 \right) \left( 65 \right) = 91.3 \text{kips/bolt} \]
\[ \leq 2.4 \left( d \right) \left( F_u \right) = 2.4 \left( 3/4 \right) \left( 0.720 \right) \left( 65 \right) = 84.2 \text{kips/bolt} \]

For 4 bolts: \( R_n = 4 \left( 84.2 \right) = 336.8 \text{kips} \)

LRFD (\( \phi = 0.75 \)): \( \phi R_n = 0.75 \left( 336.8 \right) = 252.6 \text{kips} > R_u = 62.4 \text{kips} \quad \text{OK} \)

Select 6 x 4 x 1/2 beam seat angle, 8" long, Type B connection, connected to the column with four 3/4" A325-N bolts.

Note: Because the flange width (b\text{f}) of the W24 x 55 is 7.01", the 8" long angle is selected since it provides full support for the beam. The 8" angle also fits within the flange width of the W14 x 68 column (b\text{f} = 10.0"").

\textbf{ASD}

\[ R_a = D + L = 12 + 30 = 42.0 \text{kips} \]

\textbf{Check local web yielding}

Assume \( l_b = 3\frac{1}{2}" \) for the outstanding leg.

Determine the required length of bearing \( l_b \) based on AISC Equation J10-3 and simplified using the following equation (AISC Manual, p. 9-20).

\[ R_n / \Omega = R_1 / \Omega + l_b \left( R_2 / \Omega \right) \quad \text{Equation 9-45b} \]

From AISC Table 9-4: \( R_1 / \Omega = 33.2 \) and \( R_2 / \Omega = 13.2 \) for W24 x 55

\[ R_n / \Omega = 42.0 = 33.2 + l_b \left( 13.2 \right) \]

Required \( l_b = 0.67" \) (controls) \( < l_b = 3\frac{1}{2}" \quad \text{OK} \)

\textbf{Check web crippling}

\[ l_b / d = 3.5/23.6 = 0.148 < 0.20 \]

Determine the required length of bearing \( l_b \) based on AISC Equation J10-5a and simplified using the following equation (AISC Manual, p. 9-19).

\[ R_n / \Omega = R_3 / \Omega + l_b \left( R_4 / \Omega \right) \quad \text{Equation 9-47b} \]

From AISC Table 9-4: \( R_3 / \Omega = 42.5 \) and \( R_4 / \Omega = 3.74 \) for W24 x 55

\[ R_n / \Omega = 42.0 = 42.5 + l_b \left( 3.74 \right) \]

Required \( l_b = -0.13" \) \( < l_b = 3\frac{1}{2}" \quad \text{OK} \)
Determine the seat angle type, seat angle size, and angle thickness.

- Using the lower portion of AISC Table 10-5 (pp. 10-89 to 10-90), the possible selections for "bolt available strength" for the given bolt diameter (3/4") and bolt type (A325-N) are as follows.

  - A325-N bolts are defined as Group A in Specification Section J3.1.

<table>
<thead>
<tr>
<th>Seat type</th>
<th>Seat angle size</th>
<th>ASD bolt available strength</th>
<th>Available angle thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type A</td>
<td>6 x 4</td>
<td>23.9 kips &lt; 42.0 kips</td>
<td>NG</td>
</tr>
<tr>
<td></td>
<td>7 x 4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8 x 4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type B</td>
<td>6 x 4</td>
<td>47.7 kips &gt; 42.0 kips</td>
<td>OK</td>
</tr>
<tr>
<td></td>
<td>7 x 4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8 x 4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type C</td>
<td>8 x 4</td>
<td>71.6 kips &gt; 42.0 kips</td>
<td>OK</td>
</tr>
</tbody>
</table>

Note: Type D, Type E, and Type F seated connections are not suitable for connections made with column flanges since the center row of bolts would conflict with the column web.

Select the Type B, 6 x 4 angle, 6" or 8" long.

Angles with a longer leg (such as the 7 x 4 or 8 x 4) may be selected, if necessary, if the required angle thickness exceeds 3/4", or to increase the bearing strength of the supporting member since the gage and thus the edge to edge bolt hole distance (i.e. l_c) would be greater.

- Then, using the upper portion of AISC Table 10-5, with a required bearing length of l_b = 0.67" (say 11/16"):

  The maximum angle thickness is 1/2" for an angle length of 6".
  ASD outstanding angle leg length strength = 37.0 kips < R_a = 42.0 kips NG
  The required angle thickness is 1/2" for an angle length of 8".
  ASD outstanding angle leg length strength = 49.3 kips > R_a = 42.0 kips OK

Finally, check the bearing strength of the supporting member (i.e. column flange).

- Applicable equation (assuming that deformation at the bolt hole at service load is a design consideration): R_n = 1.2 l_c * F_u ≤ 2.4 d * F_u AISC Equation J3-6a

- Determine the value for l_c.

  Edge of hole to edge of plate:
  - The distance l_c for the vertical distance from the edge of the bolt hole to the end of the column is presumably large and can be disregarded.

  Edge-to-edge of holes:
  l_c = g_2 - 2 (1/2)(diameter of the bolt hole) = 2.5 - 2 (1/2) (3/4 + 1/8) = 1.625"
where
\[ g_2 = \text{the gage distance between two horizontal lines of bolts in the vertical leg of the Type B angle seat } \text{(ref. AISC Manual, p.1-48).} \]

\[ R_n = 1.2 \ l_c \ t \ F_u = 1.2 \ (1.625) \ (0.720) \ (65) = 91.3 \text{ kips/bolt} \]
\[ \leq 2.4 \ d \ t \ F_u = 2.4 \ (3/4) \ (0.720) \ (65) = 84.2 \text{ kips/bolt} \]

For 4 bolts: \( R_n = 4(84.2) = 336.8 \text{ kips} \)

\[ \text{ASD (} \Omega = 2.00\text{): } R_n/\Omega = 336.8/2.00 = 168.4 \text{ kips} \ > R_a = 42.0 \text{ kips } \text{OK} \]

Select 6 x 4 x 1/2 beam seat angle, 8” long, Type B connection, connected to the column with four 3/4” A325-N bolts.

Note: Because the flange width (b_f) of the W24 x 55 is 7.01”, the 8” long angle is selected since it provides full support for the beam. The 8” angle also fits within the flange width of the W14 x 68 column (b_f = 10.0”).
11.10 Light Bracing Connections

Bracing connections have many possible variations.

- Figure 11.13a (p. 432 of the textbook) shows a tension member bolted to a bracket (tee), while the bracket is welded to a column member.
- Figure 11.13b (p. 432 of the textbook) shows a tension member that is welded to a bracket (tee), while the bracket is bolted to a column member.
- Such a connection could be a bolted-bolted connection or a welded-welded connection.

Although they appear to be a simple connection, light bracing connections require checking for several limit states.

Limits states for light bracing bolted-welded connections include the following.

- For angles, tension yielding, tension rupture, bolt bearing and tear-out, and block shear rupture.
- For bolts, shear rupture.
- For the gusset plate, tension yielding, tension rupture, and bolt bearing and tear-out.
- For the welds, weld rupture for combined tension and shear.

Limits states for light bracing welded-bolted connections include the following.
- For angles, tension yielding, and tension rupture.
- For welds, weld rupture.
- For the stem tee, tension yielding, tension rupture, block shear, and shear yielding.
- For the tee flange, flange bending, shear yielding, shear rupture, bolt bearing and tear-out, and block shear.
- For the bolts, combined shear and tension.
- For the column flange, flange bending, bolt bearing and tear-out.
- For the column web, web yielding.

A further consideration that should be given to tensile connections is the possibility of prying action.
Consider the connection shown in the figure below.

![Figure](image)

- The connection shown above in figure (a) is subjected to prying action as illustrated in figure (b).
  - If the flanges are thick and stiff or have stiffener plates as illustrated in figure (c), the prying action will be negligible.
  - If the flanges are thin and flexible and have no stiffeners, the prying action may be significant.

The additional tensile force in the bolts resulting from prying action should be added to the tensile force resulting from the applied loads.

Hanger and other tension connections should be designed to prevent significant deformations.
- To achieve this goal, the distance $b$ should be as small as possible.
  - The minimum value $b$ is equal to the space needed to use a wrench for tightening the bolts.

Information regarding wrench clearance dimensions is presented in Part 7 of the AISC Manual (ref. Tables 7-15 and 7-16 entitled “Entering and Tightening Clearance”).

Prying action, which is only present in bolted connections, is caused by the deformation of the connecting elements when tensile forces are applied.
• The result is an increase in the forces in some of the bolts caused directly by the tensile forces.

• If the thicknesses of the connected parts are greater than minimum thicknesses prescribed by Part 9 of the AISC Manual (p. 9-10), then prying action is considered negligible.
  
  For LRFD: \[ t_{\text{min}} = \left[\frac{(4 \ T \ b')}{(\varphi \ p \ F_u)}\right]^{1/2} \]  
  Equation 9-20a

  For ASD: \[ t_{\text{min}} = \left[\frac{(\Omega \ 4 \ T \ b')}{(p \ F_u)}\right]^{1/2} \]  
  Equation 9-20b

where

\[ T = \text{the required strength of each bolt} \]
\[ = r_{ut} = T_u/n \text{ for LRFD} \]
\[ = r_{at} = T_a/n \text{ for ASD} \]

\[ b' = (b - d_{b}/2) \]

\[ b = \text{the distance from the centerline of the bolt to the face of tee} \]

  Note: For an angle, the distance “b” is the distance from the centerline of the bolt to the centerline of the angle leg.

\[ d_{b} = \text{the bolt diameter} \]
\[ p = \text{tributary length; maximum = 2b, but} \leq s, \text{unless tests indicate larger lengths can be used} \]
\[ s = \text{bolt spacing} \]
\[ F_u = \text{the specified minimum tensile strength of the connecting element} \]

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

• If the thicknesses of the connected parts are less than or equal to the minimum thicknesses prescribed by Part 9 of the AISC Manual (p. 9-10), then prying action is considered significant.
Example Problem – Prying Action

Example

Given: The connection shown.
Bolts: Four 7/8” - A325 spaced at 5.5” c/c (i.e. the gage distance for the W36 x 150)
Steel (tee): A992 (F_u = 65 ksi)
Service loads: P_D = 30 kips, P_L = 40 kips

Find: Is the flange of the tee sufficiently thick if prying action is considered? Evaluate using both LRFD and ASD.

Solution

WT8 x 22.5 (t_f = 0.565”, t_w = 0.345”, b_f = 7.04”, g = 3.5”)
The gage distance for the WT8 x 22.5 listed in the AISC Manual is 3½” (not 4” as shown in the figure above). A gage distance of 3½” is used in the following calculations.

LRFD
Determine the factored load.

\[ T_u = 1.2 \times D + 1.6 \times L = 1.2 \times 30 + 1.6 \times 40 = 100 \text{ kips} \]

Compute the required strength for each bolt.

\[ r_{ut} = T_u / n = 100 / 4 = 25.00 \text{ kips} \]

Determine the minimum thickness for the flange of the tee.

Applicable equation: \( t_{min} = \left[ \frac{(4 \times T b')}{(\phi \ p \ F_u)} \right]^{1/2} \)  
\( T = r_{ut} = 25.00 \text{ kips} \)

\( b = (g - t_w)/2 = (3.5 - 0.345)/2 = 1.578" \)

\( b' = b - d_b/2 = 1.578 - (7/8)/2 = 1.141" \)

\( p = 2b = 2 \times 1.578 = 3.16" < s = 5.5" \) (Use p = 3.16”)

\( t_{min} = \left[ \frac{(4 \times T b')}{(\phi \ p \ F_u)} \right]^{1/2} \)

\[ = \left[ \frac{(4 \times (25.00) \times 1.141)}{(0.9 \times (3.16) \times 65)} \right]^{1/2} = 0.786" > t_f = 0.565" \]

Prying action must be considered.
Determine the load combination.

\[ T_a = D + L = 30 + 40 = 70.0 \text{ kips} \]

Compute the required strength for each bolt.

\[ r_{at} = T_a/n = 70.0/4 = 17.50 \text{ kips} \]

Determine the minimum thickness for the flange of the tee.

Applicable equation: \( t_{\text{min}} = \left[\frac{\Omega 4 T b'}{(p F_u)}\right]^{1/2} \) Equation 9-20b

\[ T = r_{at} = 17.50 \text{ kips} \]
\[ b = (g - t_w)/2 = (3.5 - 0.345)/2 = 1.578" \]
\[ b' = b - d_b/2 = 1.578 - (7/8)/2 = 1.141" \]
\[ p = 2b = 2 (1.578) = 3.16" < s = 5.5" \] (Use \( p = 3.16" \))

\[ t_{\text{min}} = \left[\frac{\Omega 4 T b'}{(p F_u)}\right]^{1/2} \]
\[ = \left[\frac{1.67 (4) (17.50) 1.141}{(3.16) 65}\right]^{1/2} = 0.806" > t_f = 0.565" \]

Prying action must be considered.
11.11 Beam Bearing Plates and Column Base Plates

When the ends of beams are supported by direct bearing on concrete or other masonry construction, it is often necessary to distribute the beam reactions over the masonry by means of beam-bearing plates.

- The reaction $R_u$ (LRFD) or $R_a$ (ASD) is assumed to spread uniformly through the bearing plate to the masonry.
- The masonry is assumed to push up against the plate with a uniform pressure.
  - This pressure tends to curl up the plate and bottom flange of the beam.
- The AISC Manual recommends that the bearing plate be designed to resist the entire bending moment produced by the upward pressure on the plate.
  - The flange of the beam is considered to have no contribution in resisting this moment.
  - The critical section for calculating the moment is taken at a distance $k$ from the centerline of the beam.

On some occasions, the beam flanges alone may provide sufficient bearing area.
- Bearing plates are still recommended.
  - Bearing plates are useful during construction to ensure an even bearing surface for the beam.
    - Bearing plates can be placed separately from the steel and carefully leveled to the proper elevations.

According to Section J8 of the AISC Specification, the design strength for bearing on concrete is taken equal to $\varphi_c P_p$ (LRFD) or $P_p/\Omega_c$ (ASD).
- If the bearing plate extends for the full area of a concrete support, then the bearing strength of the concrete is determined from the following equation.
  \[
  P_p = 0.85 f_c' A_1
  \]  
  AISC Equation J8-1
- If the area of the bearing plate is less than the full area of the concrete support, then $P_p$ is determined from the following equation.
  \[
  P_p = 0.85 f_c' A_1 (A_2/A_1)^{1/2} \leq 1.7 f_c' A_1
  \]  
  AISC Equation J8-2

where

- $f_c' = \text{compression strength of the concrete}$
- $A_2 = \text{maximum area of the supporting surface that is geometrically similar to and concentric with the loaded area and } (A_2/A_1)^{1/2} \leq 2.0$
The following equations are used to determine the required area $A_1$ of the beam bearing plate.

LRFD ($\varphi_c = 0.65$): $A_1 = \frac{R_u}{(\varphi_c 0.85f_c')}$

ASD ($\Omega_c = 2.31$): $A_1 = \frac{\Omega_c R_a}{0.85f_c'}$

After $A_1$ is determined, its length $C$ (parallel to the beam) and its width $B$ are selected.

- The length and width are usually taken to the nearest full inch.
- The length $C$ may not be less than the "$l_b$" required to prevent web yielding or web crippling of the beam.
- The length $C$ should not be less than $3\frac{1}{2}"$ or 4" for practical construction reasons.
- The length $C$ may not be greater than the thickness of the wall or other support.
  - It should actually be less than that thickness, particularly at exterior walls, to prevent the steel from being exposed.

The required plate thickness is determined as follows.

- The moment $M_u$ (LRFD) or $M_a$ (ASD) is computed at the critical section (similar to the computation of moment at the support for a cantilever beam).
  - The critical section for calculating the moment is taken at a distance $k$ from the centerline of the beam web.

LRFD: $M_u = (R_u/A_1) n (n/2) = R_u n^2/2A_1$

ASD: $M_a = (R_a/A_1) n (n/2) = R_a n^2/2A_1$

- The moment $M_u$ (LRFD) or $M_a$ (ASD) and is then equated to $\varphi_b F_y Z$ and $F_y Z/\Omega_b$, respectively.
  - For a typical 1" wide section of plate, $Z = bh^2/4 = 1"(t^2)/4 = t^2/4$

- The following equations are then used to find the required plate thickness.

LRFD ($\varphi_b = 0.90$): $R_u n^2/2A_1 = \varphi_b F_y (t^2/4)$

and $t_{min} = [(2 R_u n^2)/(\varphi_b A_1 F_y)]^{1/2} \quad \text{ref. Equation 14-7a}$

ASD ($\Omega_b = 1.67$): $R_a n^2/2A_1 = F_y (t^2/4)/\Omega_b$

and $t_{min} = [(2 R_a n^2) \Omega_b/(A_1 F_y)]^{1/2} \quad \text{ref. Equation 14-7b}$

where

$n = (B - 2k)/2$
Example Problem – Beam Bearing Plate

Example

Given: A W18 x 71 beam; the end is supported by a reinforced concrete wall.
Steel (bearing plate): A36 (F_y = 36 ksi)
Concrete compression strength: f_c' = 3 ksi
Service load reactions: R_D = 30 kips, R_L = 50 kips
Maximum length of end bearing perpendicular to the 8” wall is the full wall thickness: C ≤ 8.0”

Find: Design a beam bearing plate.

Solution

W18 x 71 (d = 18.5”, t_w = 0.495”, b_f = 7.64”, t_f = 0.810”, k = 1.21”)

LRFD
R_u = 1.2 D + 1.6 L = 1.2 (30) + 1.6 (50) = 116.0 kips

Compute the plate area and plate dimensions.

LRFD (φ_c = 0.65): A_1 = R_u/(φ_c 0.85 f_c') = 116.0/[0.65(0.85)(3.0)] = 70.0 in²

Let C = 8”: B = A_1/C = 70.0/8.0 = 8.75” (Use 9”)

Try PL 8” x 9” (A_1 = 72.0 in²)

Check web local yielding (l_b = C = 8”) using Equation J10-3.
R_n = F_yw t_w (2.5 k + l_b) = 50 (0.495) [2.5 (1.21) + 8] = 272.9 kips
LRFD (φ = 1.00): φ R_n = 1.00 (272.9) = 272.9 kips > 116.0 kips OK

Check web crippling.
l_b/d = 8.0/18.5 = 0.432 > 0.2 Use AISC Equation J10-5b
R_n = 0.40 t_w^2 [1 + (4 l_b/d - 0.2)(t_w/t_f)^1.5][E F_yw t_f/t_w]^{1/2}
= 0.40(0.495)^2[1 + (4(8.0/18.5) - 0.2)(0.495/0.810)^1.5][29,000(50)(0.810/0.495)]^{1/2}
= 0.0980 (1.7308) (1540.4) = 261.3 kips

LRFD (φ = 0.75): φ R_n = 0.75 (261.3) = 196.0 kips > 116.0 kips OK
Determine the plate thickness.

\[ B = 2n + 2k \] thus,

\[ n = \frac{(B \ - \ 2k)}{2} \ = \frac{(9 \ - \ 2(1.21))}{2} = 3.29" \]

LRFD \( (\varphi_b = 0.90) \):

\[ t_{\text{min}} = \left[ \frac{(2 \ R_u \ n^2)}{\varphi_b \ A_1 \ F_y} \right]^{1/2} \]

\[ = \left[ 2 \ (116.0) \ (3.29)^2/0.90 \ (72.0) \ 36 \right]^{1/2} = 1.038" \quad (\text{Say} \ 1\frac{1}{8}"") \]

Use PL 8" x 9" x 1\frac{1}{8}" 

**ASD**

\[ R_a = D + L = 30 + 50 = 80.0 \text{ kips} \]

Compute the plate area and plate dimensions.

**ASD \( (\Omega_c = 2.31) \):**

\[ A_1 = \frac{\Omega_c \ R_a}{0.85 \ f_c'} = 2.31 \ (80.0)/(0.85 \ (3.0)) = 72.5 \text{ in}^2 \]

Let \( C = 8" \): \( B = A_1/C = 72.5/8.0 = 9.06" \quad (\text{Use} \ 10") \)

Try PL 8" x 10" \( (A_1 = 80 \text{ in}^2) \)

Check web local yielding \( (l_b = C = 8") \) using Equation J10-3.

\[ R_n = F_{yw} \ t_w (2.5 \ k + l_b) = 50 \ (0.495) \ [2.5(1.21) + 8] = 272.9 \text{ kips} \]

**ASD \( (\Omega = 1.50) \):** \( R_n/\Omega = 272.9/1.50 = 181.9 \text{ kips} > 80.0 \text{ kips} \quad \text{OK} \)

Check web crippling.

\[ \frac{l_b}{d} = 8.0/18.5 = 0.432 > 0.2 \quad \text{Use AISC Equation J10-5b} \]

\[ R_n = 0.40 \ t_w^2 \left[ 1 + (4 \ l_b/d - 0.2)(t_w/t_f)^{1.5}\right] \left(E \ F_{yw} \ t_f/t_w\right)^{1/2} \]

\[ = 0.40(0.495)^2 \left[ 1 + (4(8.0/18.5) - 0.2)(0.495/0.810)^{1.5}\right] \left[29,000(50)(0.810/0.495)\right]^{1/2} \]

\[ = 0.0980(1.7308)(1540.4) = 261.3 \text{ kips} \]

**ASD \( (\Omega = 2.00) \):** \( R_n/\Omega = 261.3/2.00 = 130.6 \text{ kips} > 80.0 \text{ kips} \quad \text{OK} \)

Determine the plate thickness.

\[ B = 2n + 2k \] thus,

\[ n = \frac{(B \ - \ 2k)}{2} \ = \frac{(10 \ - \ 2(1.21))}{2} = 3.79" \]

**ASD \( (\Omega_b = 1.67) \):**

\[ t_{\text{min}} = \left[ \frac{(2 \ R_u \ n^2)}{\Omega_b/(A_1 \ F_y)} \right]^{1/2} \]

\[ = \left[ 2 \ (80.0) \ (3.79)^2/(1.67)/(80.0 \ (36)) \right]^{1/2} = 1.15" \quad (\text{Say} \ 1\frac{1}{4}"") \]

Use PL 8" x 10" x 1\frac{1}{4}"