Connections and Tension Member Design

Connections

Connections must be able to transfer any axial force, shear, or moment from member to member or from beam to column.

Steel construction accomplishes this with bolt and welds. Wood construction uses nails, bolts, shear plates, and split-ring connectors.

Bolted and Welded Connections

The limit state for connections depends on the loads:

1. tension yielding
2. shear yielding
3. bearing yielding
4. bending yielding due to eccentric loads
5. rupture

Welds must resist tension AND shear stress. The design strengths depend on the weld materials.

Bolted Connection Design

Bolt designations signify material and type of connection where:

SC: slip critical
N: bearing-type connection with bolt threads included in shear plane
X: bearing-type connection with bolt threads excluded from shear plane

Bolts rarely fail in ________________. The material with the hole will more likely yield first.

Standard bolt holes are 1/16” larger than the bolt diameter.

ASD

Allowable shear values are given by bolt type, connection type, hole type, diameter, and loading (Single or Double shear) in AISC manual tables.

Allowable bearing force values are given by bolt diameter, ultimate tensile strength, $F_u$, of the connected part, and thickness of the connected part in AISC manual tables.
### TABLE I-D. SHEAR

**Allowable load in kips**

<table>
<thead>
<tr>
<th>ASTM Designation</th>
<th>Conjunction Type</th>
<th>Hole Type</th>
<th>F&lt;sub&gt;c&lt;/sub&gt; ksi</th>
<th>Load in kips</th>
<th>% F&lt;sub&gt;c&lt;/sub&gt;</th>
<th>% F&lt;sub&gt;c&lt;/sub&gt;</th>
<th>% F&lt;sub&gt;c&lt;/sub&gt;</th>
<th>% F&lt;sub&gt;c&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>A307</td>
<td>STD</td>
<td>S</td>
<td>10.0</td>
<td>0.366</td>
<td>0.41</td>
<td>0.45</td>
<td>0.48</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>D</td>
<td>12.0</td>
<td>0.41</td>
<td>0.45</td>
<td>0.48</td>
<td>0.50</td>
<td>0.53</td>
</tr>
<tr>
<td></td>
<td>STD</td>
<td>S</td>
<td>17.0</td>
<td>0.476</td>
<td>0.53</td>
<td>0.58</td>
<td>0.60</td>
<td>0.63</td>
</tr>
<tr>
<td></td>
<td></td>
<td>D</td>
<td>20.0</td>
<td>0.53</td>
<td>0.58</td>
<td>0.60</td>
<td>0.63</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>STD</td>
<td>S</td>
<td>30.0</td>
<td>0.75</td>
<td>0.83</td>
<td>0.88</td>
<td>0.90</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td></td>
<td>D</td>
<td>40.0</td>
<td>0.90</td>
<td>0.92</td>
<td>0.94</td>
<td>0.95</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td>STD</td>
<td>S</td>
<td>50.0</td>
<td>0.96</td>
<td>0.99</td>
<td>1.01</td>
<td>1.02</td>
<td>1.03</td>
</tr>
<tr>
<td></td>
<td></td>
<td>D</td>
<td>60.0</td>
<td>1.01</td>
<td>1.03</td>
<td>1.04</td>
<td>1.05</td>
<td>1.05</td>
</tr>
</tbody>
</table>

### TABLE I-E. BEARING

**Bearing allowable loads in kips**

<table>
<thead>
<tr>
<th>Material</th>
<th>F&lt;sub&gt;c&lt;/sub&gt; (ksi)</th>
<th>Area (Based on Nominal Diameter)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>F&lt;sub&gt;c&lt;/sub&gt; = 58</td>
<td>F&lt;sub&gt;c&lt;/sub&gt; = 65</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Bolt dia.</td>
<td>Bolt dia.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1/8</td>
<td>1/4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6.5</td>
<td>5.5</td>
</tr>
<tr>
<td></td>
<td>9.0</td>
<td>8.0</td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>11.0</td>
</tr>
</tbody>
</table>

Notes:
- This table is applicable to all mechanical fasteners in both slip-critical and bearing-type connections utilizing standard holes. Standard holes shall have a diameter nominally 1/8-in. larger than the nominal bolt diameter (d = 1/8-in.).
- Tabulated bearing values are based on F<sub>c</sub> = 1.2 F<sub>b</sub>.
- Connections transmitting axial force whose length between extreme fasteners measured parallel to the line of force exceeds 50 in. tabulated values shall be reduced 20%.
- Connections using high-strength bolts in socket holes with the load applied in a direction other than approximately normal (between 80 and 100 degrees) to the axis of the hole and connections with bolts in oversize holes shall be designed for resistance against slip at working load in accordance with AISC ASD Specification Sect. J3.8.
- Tabulated values apply when the distance I (parallel to the line of force from the center of the bolt to the edge of the connected part) is not less than 1½ times and the distance from the center of a bolt to the center of an adjacent bolt is not less than 3d. See AISC ASD Commentary Sect. J3.8.
- Under certain conditions, values greater than the tabulated values may be justified under Specification Sect. J3.7.
- Values are limited to the double-shear bearing capacity of A490-1 bolts.
- Values for decimal thicknesses may be obtained by multiplying the decimal value of the thickness by the value given for a 1-in. thickness.

* = Slip critical connection.
K = Bearing-type connection with threads included in shear plane.
S = Bearing-type connection with threads excluded from shear plane.
* = Standard round (d = 1/8-in.)
OVS = Oversize round holes
SL = Short-shear holes
Long-shear holes normal to load direction (required in bearing-type connection).
L = Long-shear holes parallel to load direction (required in bearing-type connection).
** = For threaded parts of materials not listed, use F<sub>c</sub> = 1.17 F<sub>b</sub> when threads are included in a shear plane, and F<sub>c</sub> = 0.25 F<sub>b</sub> when threads are excluded from a shear plane.
** = For preloaded bolts, d = 1½-in., dia. and greater, special impact wrenches may be required.
** = Bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 50 in., tabulated values shall be reduced by 20%. See AISC ASD Commentary Sect. J3.4.
Tension Member Design

In steel tension members, there may be bolt holes that reduce the size of the cross section.

Effective Net Area:

The smallest effective area must be determined by subtracting the bolt hole areas. With staggered holes, the shortest length must be evaluated.

A series of bolts can also transfer a portion of the tensile force, and some of the effective net areas see reduced stress.

ASD

For other than pin connected members:

- $F_t^A = 0.60 F_y$ on gross area
- $F_t^A = 0.50 F_u$ on net area

For pin connected members:

- $F_t^A = 0.45 F_y$ on net area

For threaded rods of approved steel:

- $F_t^A = 0.33 F_u$ on major diameter (static loading only)

LRFD

The limit state for tension members are:

- $P_u \leq \phi F_n$

1. yielding

- $\phi_t = 0.9 \quad P_n = F_y A_g$

2. rupture

- $\phi_t = 0.75 \quad P_n = F_u A_e$

where $A_g$ = the gross area of the member (excluding holes)

$A_e$ = the effective net area (with holes, etc.)

$F_u$ = the tensile strength of the steel (ultimate)
Welded Connections

Weld designations include the strength in the name, i.e. E70XX has $F_y = 70$ ksi.

The throat size, $T$, of a fillet weld is determined trigonometry by: $T = 0.707 \times $weld size

ASD

Allowable shear stress of a weld is limited to 30% of the nominal strength.

- $F_v = 18$ ksi for E60XX
- $F_v = 21$ ksi for E70XX

Weld sizes are limited by the size of the parts being put together and are given in AISC manual table J2.4 along with the allowable strength per length of fillet weld, referred to as $S$.

The maximum size of a fillet weld:
- a) can’t be greater than the material thickness if it is ¼” or less
- b) is permitted to be 1/16” less than the thickness of the material if it is over ¼”

The minimum length of a fillet weld is 4 times the nominal size. If it is not, then the weld size used for design is ¼ the length.

Intermittent fillet welds can not be less that four times the weld size, not to be less than 1 ½”.

<table>
<thead>
<tr>
<th>Allowable Strength of Fillet Welds per inch of weld ($S$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weld Size (in.)</td>
</tr>
<tr>
<td>1/16</td>
</tr>
<tr>
<td>1/4</td>
</tr>
<tr>
<td>3/16</td>
</tr>
<tr>
<td>5/32</td>
</tr>
<tr>
<td>3/16</td>
</tr>
<tr>
<td>1/2</td>
</tr>
<tr>
<td>5/32</td>
</tr>
<tr>
<td>3/4</td>
</tr>
</tbody>
</table>

**TABLE J2.4**

Minimum Size of Fillet Welds

<table>
<thead>
<tr>
<th>Material Thickness of Thicker Part Joined (in.)</th>
<th>Minimum Size of Fillet Weld* (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>To 1/4 inclusive</td>
<td>1/16</td>
</tr>
<tr>
<td>Over 1/4 to 1/2</td>
<td>3/32</td>
</tr>
<tr>
<td>Over 1/2 to 5/32</td>
<td>1/4</td>
</tr>
<tr>
<td>Over 5/32</td>
<td>5/32</td>
</tr>
</tbody>
</table>

*Leg dimension of fillet welds. Single-pass welds must be used.

**AMERICAN INSTITUTE OF STEEL CONSTRUCTION**
Framed Beam Connections

Coping is the term for cutting away part of the flange to connect a beam to another beam using welded or bolted angles.

AISC provides tables that give angle sizes knowing bolt type, bolt diameter, angle leg thickness, and number of bolts (determined by shear capacity).

Load and Factor Resistance Design

In addition to resisting shear and tension in bolts and shear in welds, the connected materials may be subjected to shear, bearing, tension, flexure and even prying action. Coping can significantly reduce design strengths and may require web reinforcement. All the following must be considered:

- shear yielding
- shear rupture
- block shear rupture - failure of a block at a beam as a result of shear and tension
- tension yielding
- tension rupture
- local web buckling
- lateral torsional buckling
FRAMED BEAM CONNECTIONS
Bolted

TABLE II Allowable loads in kips

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>A325-N</th>
<th>A490-C</th>
<th>A325-X</th>
<th>A490-X</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_a$, KSI</td>
<td>21.0</td>
<td>28.0</td>
<td>30.0</td>
<td>40.0</td>
</tr>
</tbody>
</table>

Note: For $L = 2 \frac{1}{2}$ use one half the tabular load value shown for $L = 5 \frac{1}{2}$ for the same bolt type, diameter, and thickness.

---

TABLE II-A Bolt Shear

For A307 bolts in standard or slotted holes and for A325 and A490 bolts in slip-critical connections with standard holes and Class A, clean mill scale surface condition.

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>A307</th>
<th>A325-SC</th>
<th>A490-SC</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_n$, KSI</td>
<td>10.0</td>
<td>17.0</td>
<td>21.0</td>
</tr>
</tbody>
</table>

Note: For slip-critical connections with oversize or slotted holes, see Table II-B.

---

Notes:
*Tabulated load values are based on double shear of bolts unless noted. See RCSC Specification for other surface conditions.

*Capacity shown is based on double shear of the bolts; however, for length $L$, net shear on the angle thickness specified is critical. See Table II-C.

---

TABLE II-A Bolt Shear

For bolts in bearing-type connections with standard or slotted holes.

<table>
<thead>
<tr>
<th>Bolt Type</th>
<th>A325-N</th>
<th>A490-C</th>
<th>A325-X</th>
<th>A490-X</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_a$, KSI</td>
<td>21.0</td>
<td>28.0</td>
<td>30.0</td>
<td>40.0</td>
</tr>
</tbody>
</table>

Note: For $L = 2 \frac{1}{2}$ use one half the tabular load value shown for $L = 5 \frac{1}{2}$ for the same bolt type, diameter, and thickness.

---

Tabulated load values are based on double shear of bolts.

Shaded values are based on double shear of the bolts; however, for length $L$, net shear on the angle thickness specified is critical. See Table II-C.

For shaded cells without values, shear rupture is critical for lengths $L$ and $L'$ on angle thickness specified. See Table II-C.

---

American Institute of Steel Construction
Example 1

10.2. The butt splice shown in Figure 10.22 uses two 8 × \(\frac{3}{8}\)" plates to “sandwich” in the 8 × \(\frac{1}{2}\)" plates being joined. Four \(\frac{3}{8}\)" φ A325-SC bolts are used on both sides of the splice. Assuming A36 steel and standard round holes, determine the allowable capacity of the connection.

Solution:

Shear, bearing, and net tension will be checked to determine the critical condition that governs the capacity of the connection.

Shear: Using the AISC allowable shear in Table 10.1:

\[ P_s = 20.4 \text{k/bolt} \times 4 \text{ bolts} = 81.6 \text{k} \] (double shear) *(Table I-D)*

Bearing: Using the AISC bearing in Table 10.2:

The thinner material with the largest proportional load governs, therefore, the \(\frac{1}{2}\)" center plate governs. Assume the bolts are at a 3d spacing, center to center.

\[ P_b = 30.5 \text{k/bolt} \times 4 \text{ bolts} = 122 \text{k} \] *(Table I-D)*

Tension: The center plate is critical since its thickness is less than the combined thickness of the two outer plates.

Hole diameter = (bolt diameter) + \(\frac{1}{16}\)" = \(7\frac{1}{8}\)" + \(\frac{1}{16}\)" = \(7\frac{1}{4}\)".

\[ A_{net} = (8" - 2 \times \frac{1}{16}\") \times \left(\frac{1}{2}\"\right) = 3.06 \text{in.}^2 \]

\[ P_t = F_t \times A_{net} \]

where:

\[ F_t = 0.5F_y = 0.5(38 \text{ksi}) = 19 \text{ksi} \]

\[ P_t = 29 \text{k/in.}^2 \times 3.06 \text{in.}^2 = 88.7 \text{k} \]

For yielding in the cross section without holes:

\[ A_{gross} = (8") \times \left(\frac{1}{2}\"\right) = 4.0 \text{in.}^2 \]

\[ P_t = F_t \times A_{gross} \]

where:

\[ F_t = 0.6F_y = 0.6(36 \text{ksi}) = 21.6 \text{ksi} \]

\[ P_t = 21.6 \text{k/in.}^2 \times 4.0 \text{in.}^2 = 86.4 \text{k} \]

The maximum connection capacity is governed by shear.

\[ P_{allow} = 81.6 \text{k} \]
Example 2

10.7 Determine the capacity of the connection in Figure 10.44 assuming A36 steel with E70XX electrodes.

Solution:
Capacity of weld:
For a $\frac{3}{8}''$ fillet weld, $S = 4.64$ k/in
Weld length = 22''
Weld capacity = $22'' \times 4.64$ k/in = 102.1 k

Capacity of plate:

$$F_{p} = 0.6F_{y} = 22\text{ ksi}$$

Plate capacity = $\frac{3}{8}'' \times 6'' \times 22$ k/in.$^2 = 49.5$ k

$\therefore$ Plate capacity governs, $P_{allow} = 49.5$ k

The weld size used is obviously too strong. What size, then, can the weld be reduced to so that the weld strength is more compatible to the plate capacity? To make the weld capacity = plate capacity:

$22'' \times \text{(weld capacity per in.)} = 49.5$ k

Weld capacity per inch = $\frac{49.5\text{k}}{22\text{ in.}}$

From Table 10.5, use $\frac{3}{8}''$ weld ($S = 2.78$ k/in.).
Minimum size fillet = $\frac{3}{8}''$ based on a $\frac{3}{8}''$ thick plate.
Example 3

The steel used in the connection and beams is A992 with $F_y = 50$ ksi, and $F_u = 65$ ksi. Using A490-N bolt material, determine the maximum capacity of the connection based on shear in the bolts, bearing in all materials and pick the number of bolts and angle length (not staggered). Use A36 steel for the angles.

W21x93: $d = 21.62$ in, $t_w = 0.58$ in, $t_f = 0.93$ in
W10x54: $t_f = 0.615$ in

SOLUTION:

The maximum length the angles can be depends on how it fits between the top and bottom flange with some clearance allowed for the fillet to the flange, and getting an air wrench in to tighten the bolts. This example uses 1” of clearance:

$$\text{Available length} = \text{beam depth} - \text{both flange thicknesses} - 1” \text{ clearance at top} \& 1” \text{ at bottom}$$

$$= 21.62 \text{ in} - 2(0.93 \text{ in}) - 2(1 \text{ in}) = 17.76 \text{ in}.$$

The standard lengths for non-staggered holes (L) and staggered holes (L’) are shown in Table II-A. The closest size within the available length is 17 ½ in. This will fit 6 bolts (n) with a standard spacing.

We have a choice of bolt diameters of ¾”, 7/8” and 1” in Table II-A. These have allowable loads for shear (double) of 148 kips, 202 kips, and 264 kips. But the last two values are shaded and the note says that “net shear on the angle thickness specified is critical” and to see Table II-C. The angle thickness (t) is listed below the bolt diameter.

Table II-C gives a value of 207 kips for a 7/8” bolt diameter, ½” angle thickness, and 17.5” length. It gives a value of 242 kips for a 1” bolt diameter, 5/8” angle thickness, and 17.5” length. Therefore, 242 kips is the maximum value limited by shear in the angle.

$$P_p = 264 \text{ kips} \quad \text{for double shear of 1” bolts} \quad \text{(Table I-D: 6 bolts ⋅ (44 k/bolt) = 264 kips)}$$

$$P_v = 242 \text{ kips} \quad \text{for net shear in angle}$$

We also need to evaluate bearing of bolts on the angles, beam web, and column flange where there are bolt holes. Table I-E provides allowable bearing load for the material type, bolt diameter and some material thicknesses. The last note states that “Values for decimal thicknesses may be obtained by multiplying the decimal value of the unlisted thickness by the value given for a 1-in. thickness”. This comes from the definition for bearing stress:

$$f_p = \frac{P}{td} \leq F_p, \quad \text{where} \quad P_p = t \cdot d \cdot F_p \quad \text{at the allowable bearing stress}$$

For a constant diameter and allowable stress, the allowable load depends only on the thickness.

a) Bearing for 5/8” thick angle: There are 12 bolt holes through two angle legs to the column, and 12 bolt holes through two angle legs either side of the beam. The material is A36 ($F_u = 58$ ksi), with 1” bolt diameters.

$$P_p = 12 \text{ bolts} \cdot (43.5 \text{ k/bolt}) = 522 \text{ kips}$$

b) Bearing for column flange: There are 12 bolt holes through two angle legs to the column. The material is A992 ($F_u = 65$ ksi), 0.615” thick, with 1” bolt diameters.

$$P_p = 12 \text{ bolts} \cdot (78 \text{ k/bolt} \cdot 1”) \cdot (0.615 \text{ in}) = 576 \text{ kips}.$$

c) Bearing for beam web: There are 6 bolt holes through two angle legs either side of the beam. The material is A992 ($F_u = 65$ ksi), 0.58” thick, with 1” bolt diameters

$$P_p = 6 \text{ bolts} \cdot (78 \text{ k/bolt} \cdot 1”) \cdot (0.58 \text{ in}) = 271 \text{ kips}.$$

Although, the bearing in the beam web is the smallest at 271 kips, with the shear on the bolts even smaller at 264 kips, the maximum capacity for the simple-shear connector is 242 kips limited by net shear in the angles.