### Connection and Tension Member Design

**Notation:**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>area (net = with holes, bearing = in contact, etc...)</td>
</tr>
<tr>
<td>$A_e$</td>
<td>effective net area found from the product of the net area $A_n$ by the shear lag factor $U$</td>
</tr>
<tr>
<td>$A_b$</td>
<td>area of a bolt</td>
</tr>
<tr>
<td>$A_g$</td>
<td>gross area, equal to the total area ignoring any holes</td>
</tr>
<tr>
<td>$A_{gv}$</td>
<td>gross area subjected to shear for block shear rupture</td>
</tr>
<tr>
<td>$A_n$</td>
<td>net area, equal to the gross area subtracting any holes, as is $A_{net}$</td>
</tr>
<tr>
<td>$A_{nt}$</td>
<td>net area subjected to tension for block shear rupture</td>
</tr>
<tr>
<td>$A_{nv}$</td>
<td>net area subjected to shear for block shear rupture</td>
</tr>
<tr>
<td>$ASD$</td>
<td>allowable stress design</td>
</tr>
<tr>
<td>$d$</td>
<td>diameter of a hole</td>
</tr>
<tr>
<td>$f_p$</td>
<td>bearing stress (see P)</td>
</tr>
<tr>
<td>$f_t$</td>
<td>tensile stress</td>
</tr>
<tr>
<td>$f_v$</td>
<td>shear stress</td>
</tr>
<tr>
<td>$F_{\text{connector}}$</td>
<td>shear force capacity per connector</td>
</tr>
<tr>
<td>$F_n$</td>
<td>nominal tension or shear strength of a bolt</td>
</tr>
<tr>
<td>$F_u$</td>
<td>ultimate stress prior to failure</td>
</tr>
<tr>
<td>$F_{\text{EXX}}$</td>
<td>yield strength of weld material</td>
</tr>
<tr>
<td>$F_y$</td>
<td>yield strength</td>
</tr>
<tr>
<td>$F_{yw}$</td>
<td>yield strength of web material</td>
</tr>
<tr>
<td>$g$</td>
<td>gage spacing of staggered bolt holes</td>
</tr>
<tr>
<td>$I$</td>
<td>moment of inertia with respect to neutral axis bending</td>
</tr>
<tr>
<td>$k$</td>
<td>distance from outer face of W flange to the web toe of fillet</td>
</tr>
<tr>
<td>$l$</td>
<td>name for length</td>
</tr>
<tr>
<td>$L$</td>
<td>name for length</td>
</tr>
<tr>
<td>$L_c$</td>
<td>clear distance between the edge of a hole and edge of next hole or edge of the connected steel plate in the direction of the load</td>
</tr>
<tr>
<td>$L'$</td>
<td>length of an angle in a connector with staggered holes</td>
</tr>
</tbody>
</table>

- $LRFD = \text{load and resistance factor design}$
- $n = \text{number of connectors across a joint}$
- $N = \text{bearing length on a wide flange steel section}$
- $p = \text{pitch of connector spacing}$
- $R = \text{generic load quantity (force, shear, moment, etc.) for LRFD design}$
- $R_a = \text{required strength (ASD)}$
- $R_n = \text{nominal value (capacity) to be multiplied by } \phi$
- $R_u = \text{factored design value for LRFD design}$
- $s = \text{longitudinal center-to-center spacing of any two consecutive holes}$
- $S = \text{allowable strength per length of a weld for a given size}$
- $SC = \text{slip critical bolted connection}$
- $t = \text{thickness of a hole or member}$
- $t_w = \text{thickness of web of wide flange}$
- $T = \text{throat size of a weld}$
- $V = \text{internal shear force}$
- $V_{\text{longitudinal}} = \text{longitudinal shear force}$
- $U = \text{shear lag factor for steel tension member design}$
- $U_{bs} = \text{reduction coefficient for block shear rupture}$
- $X = \text{bearing type connection with threads excluded from the shear plane}$
- $y = \text{vertical distance}$
- $\pi = \text{pi (3.1415 radians or 180°)}$
- $\phi = \text{resistance factor}$
- $\gamma = \text{diameter symbol}$
- $\Omega = \text{load factor in LRFD design}$
- $\Sigma = \text{safety factor for ASD}$
- $\sum = \text{summation symbol}$
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.

**Single Shear** - forces cause only one shear “drop” across the bolt.

\[ f_v = \frac{P}{2A} = \frac{P}{\pi r^2} \]

**(two shear planes)**

**Double Shear** - forces cause two shear changes across the bolt.

\[ f_v = \frac{P}{A} = \frac{P}{\pi r^2} \]

**Free-body diagram of middle section of the bolt in shear.**

**Figure 5.12 A bolted connection in double shear.**
**Bearing of a Bolt on a Bolt Hole** – The bearing surface can be represented by projecting the cross section of the bolt hole on a plane (into a rectangle).

\[ f_p = \frac{P}{A} = \frac{P}{td} \]

*Figure: Bearing stress on plate.*

**Horizontal Shear in Composite Beams**

Typical connections needing to resist shear are plates with nails or rivets or bolts in composite sections or splices.

The pitch (spacing) can be determined by the capacity in shear of the connector(s) to the shear flow over the spacing interval, \( p \).

\[ \frac{V_{\text{longitudinal}}}{p} = \frac{V Q}{I} \]

\[ V_{\text{longitudinal}} = \frac{V Q}{I} \cdot p \]

where

\[ n F_{\text{connector}} \geq \frac{V Q_{\text{connected area}}}{I} \cdot p \]

\( p = \) pitch length

\( n = \) number of connectors connecting the connected area to the rest of the cross section

\( F = \) force capacity in one connector

\( Q_{\text{connected area}} = A_{\text{connected area}} \times y_{\text{connected area}} \)

\( y_{\text{connected area}} = \) distance from the centroid of the connected area to the neutral axis

**Connectors to Resist Horizontal Shear in Composite Beams**

Even vertical connectors have shear flow across them.

The spacing can be determined by the capacity in shear of the connector(s) to the shear flow over the spacing interval, \( p \).

\[ p \leq \frac{n F_{\text{connector}} J}{V Q_{\text{connected area}}} \]
Tension Member Design

In 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.

\[ f_t = \frac{P}{A_e} \text{ or } \frac{T}{A_e} \]

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

Connections in Wood

Connections for wood are typically mechanical fasteners. Shear plates and split ring connectors are common in trusses. Bolts of metal bear on holes in wood, and nails rely on shear resistance transverse and parallel to the nail shaft.

Bolted Joints

Stress must be evaluated in the member being connected using the load being transferred and the reduced cross section area called net area. Bolt capacities are usually provided in tables and take into account the allowable shearing stress across the diameter for single and double shear, and the allowable bearing stress of the connected material based on the direction of the load with respect to the grain (parallel or perpendicular). Problems, such as ripping of the bolt hole at the end of the member, are avoided by following code guidelines on minimum edge distances and spacing.
Nailed Joints

Because nails rely on shear resistance, a common problem when nailing is splitting of the wood at the end of the member, which is a shear failure. Tables list the shear force capacity per unit length of embedment per nail. Jointed members used for beams will have shear stress across the connector, and the pitch spacing, \( p \), can be determined from the shear stress equation when the capacity, \( F \), is known.

Other Connectors

**Screws** - Range in sizes from #6 (0.138 in. shank diameter) to #24 (0.372 in. shank diameter) in lengths up to five inches. Like nails, they are best used laterally loaded in side grain rather than in withdrawal from side grain. Withdrawal from end is not permitted.

**Lag screws (or bolts)** – Similar to wood screw, but has a head like a bolt. It must have a load hole drilled and inserted along with a washer.

**Split ring and shear plate connectors** – Grooves are cut in each piece of the wood members to be joined so that half the ring is in each section. The members are held together with a bolt concentric with the ring. Shear plate connectors have a central plate within the ring.

**Splice plates** – These are common in pre-manufactured joists and consist of a sheet of metal with punched spikes.

**Framing seats & anchors** – for instance, joist hangers and post bases...

Connections in Steel

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

High strength bolts resist shear (primarily), while the connected part must resist yielding and rupture.

Welds must resist 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

A307: similar in strength to A36 steel (also known as ordinary, common or unfinished bolts)
A325: high strength bolts (Group A)
A490: high strength bolts (higher than A325, Group B)

Bearing-type connection: no frictional resistance in the contact surfaces is assumed and slip between members occurs as the load is applied. (Load transfer through bolt only).
Slip-critical connections: bolts are torqued to a high tensile stress in the shank, resulting in a clamping force on the connected parts. (Shear resisted by clamping force).
Requires inspections and is useful for structures seeing dynamic or fatigue loading.
Class A indicates the faying (contact) surfaces are clean mill scale or adequate paint system, while Class B indicates blast cleaning or paint for $\mu = 0.50$.

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

For the determination of the net area of a bolt hole the width is taken as 1/16” greater than the nominal dimension of the hole. Standard diameters for bolt holes are 1/16” larger than the bolt diameter. (This means the net width will be 1/8” larger than the bolt.)

Design for Bolts in Bearing, Shear and Tension

Available shear values are given by bolt type, diameter, and loading (Single or Double shear) in AISC manual tables. Available shear value for slip-critical connections are given for limit states of serviceability or strength by bolt type, hole type (standard, short-slotted, long-slotted or oversized), diameter, and loading. Available tension values are given by bolt type and diameter in AISC manual tables.

Available 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.

For shear OR tension (same equation) in bolts:  
\[ R_u \leq R_n / \Omega \text{ or } R_u \leq \phi R_n \]

where $R_u = \sum \gamma_i R_i$

- single shear (or tension)  
  \[ R_n = F_n A_b \]
- double shear  
  \[ R_n = F_n 2A_b \]
where \( \phi = 0.75 \) (LRFD)
\( \Omega = 2.00 \) (ASD)

\( A_s \) is the cross-section area of the bolt

**Table 7-3**

<table>
<thead>
<tr>
<th>Hole Type</th>
<th>Loading</th>
<th>ASTM Design</th>
<th>Thread Cond.</th>
<th>( F_{tu}(ksi) )</th>
<th>( F_{tu}(ksi) )</th>
<th>Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>STD/SSL</td>
<td>S</td>
<td>A325, A325M</td>
<td>ASDF</td>
<td>5.6</td>
<td>6.5</td>
<td>ASDF</td>
</tr>
<tr>
<td>D</td>
<td></td>
<td>F1858</td>
<td>ASDF</td>
<td>4.2</td>
<td>5.0</td>
<td>ASDF</td>
</tr>
<tr>
<td>OVS/SSL</td>
<td>S</td>
<td>A354 Grade BC</td>
<td>ASDF</td>
<td>3.5</td>
<td>4.4</td>
<td>ASDF</td>
</tr>
<tr>
<td>LSL</td>
<td>S</td>
<td>A449</td>
<td>ASDF</td>
<td>3.0</td>
<td>3.9</td>
<td>ASDF</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td></td>
<td>ASDF</td>
<td>2.8</td>
<td>3.7</td>
<td>ASDF</td>
</tr>
</tbody>
</table>

**Table 7-1**

<table>
<thead>
<tr>
<th>Nominal Bolt Diameter, d, in.</th>
<th>( \phi )</th>
<th>( \phi )</th>
<th>( \phi )</th>
<th>( \phi )</th>
<th>( \phi )</th>
<th>( \phi )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.307</td>
<td>0.442</td>
<td>0.601</td>
<td>0.785</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table 7-2**

<table>
<thead>
<tr>
<th>Nominal Bolt Diameter, d, in.</th>
<th>( \phi )</th>
<th>( \phi )</th>
<th>( \phi )</th>
<th>( \phi )</th>
<th>( \phi )</th>
<th>( \phi )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.904</td>
<td>1.233</td>
<td>1.483</td>
<td>1.717</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Note**: Slip-critical bolt values assume no more than one filler has been provided or bolts have been added to distribute loads in the fillers. See ASCE Specification Sections J3.5 and J3.6 for provisions when fillers are present.

For Class B faying surfaces, multiply the tabulated available strength by 1.67.
For bearing of plate material at bolt holes:

\[ R_u \leq \frac{R_n}{\Omega} \quad \text{or} \quad R_u \leq \phi R_n \]

where \( R_u = \Sigma \gamma_i R_j \)

- deformation at bolt hole is a concern
  \[ R_u = 1.2 L_c F_u \leq 2.4 d t F_u \]
- deformation at bolt hole is not a concern
  \[ R_u = 1.5 L_c F_u \leq 3.0 d t F_u \]
- long slotted holes with the slot perpendicular to the load
  \[ R_u = 1.0 L_c F_u \leq 2.0 d t F_u \]

where

- \( R_n \) = the nominal bearing strength
- \( F_u \) = specified minimum tensile strength
- \( L_c \) = clear distance between the edges of the hole and the next hole or edge in the direction of the load
- \( d \) = nominal bolt diameter
- \( t \) = thickness of connected material
- \( \phi = 0.75 \) (LRFD) \quad \( \Omega = 2.00 \) (ASD)

The minimum edge distance from the center of the outer most bolt to the edge of a member is generally 1¾ times the bolt diameter for the sheared edge and 1¼ times the bolt diameter for the rolled or gas cut edges.

The maximum edge distance should not exceed 12 times the thickness of thinner member or 6 in.

Standard bolt hole spacing is 3 in. with the minimum spacing of 2 3/4 times the diameter of the bolt, \( d_b \). Common edge distance from the center of last hole to the edge is 1¾ in.
Tension Member Design

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

- $g$ refers to the row spacing or gage
- $p$ refers to the bolt spacing or pitch
- $s$ refers to the longitudinal spacing of two consecutive holes

**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.

The effective net area, $A_e$, is determined from the net area, $A_n$, multiplied by a shear lag factor, $U$, which depends on the element type and connection configuration. If a portion of a connected member is not fully connected (like the leg of an angle), the unconnected part is not subject to the full stress and the shear lag factor can range from 0.6 to 1.0: 

$$A_e = A_n U$$

The staggered hole path area is determined by:

$$A_n = A_g - A_{all\ holes} + t \sum \frac{s}{4g}$$

where $t$ is the plate thickness, $s$ is each stagger spacing, and $g$ is the gage spacing.
For tension elements: \[ R_u \leq R_y / \Omega \text{ or } R_u \leq \phi R_n \]
where \( R_u = \sum \gamma_i R_i \)

1. yielding \[ R_n = F_y A_g \]
   \[ \phi = 0.90 \text{ (LRFD)} \quad \Omega = 1.67 \text{ (ASD)} \]
2. rupture \[ R_n = F_u A_e \]
   \[ \phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)} \]

where \( A_g \) = the gross area of the member (excluding holes)
\( A_e \) = the effective net area (with holes, etc.)
\( F_y \) = the yield strength of the steel
\( F_u \) = the tensile strength of the steel
(ultimate)

For shear elements: \[ R_u \leq R_y / \Omega \text{ or } R_u \leq \phi R_n \]
where \( R_u = \sum \gamma_i R_i \)

1. yielding \[ R_n = 0.6 F_y A_g \]
   \[ \phi = 1.00 \text{ (LRFD)} \quad \Omega = 1.50 \text{ (ASD)} \]
2. rupture \[ R_n = 0.6 F_u A_{nv} \]
   \[ \phi = 0.75 \text{ (LRFD)} \quad \Omega = 2.00 \text{ (ASD)} \]

where \( A_g \) = the gross area of the member (excluding holes)
\( A_{nv} \) = the net area subject to shear (with holes, etc.)
\( F_y \) = the yield strength of the steel
\( F_u \) = 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. Welds are weakest in shear and
are assumed to always fail in the shear mode.

The throat size, \( T \), of a fillet weld is determined
trigonometry by: \( T = 0.707 \times \text{weld size}^* \)
* When the submerged arc weld process is used, welds over 3/8” will have a
  throat thickness of 0.11 in. larger than the formula.
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 permitted along edges of connected parts shall be:

- Material less than ¼ in. thick, not greater than the thickness of the material.
- Material ¼ in. or more in thickness, not greater than the thickness of the material minus 1/16 in., unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness.

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 cannot be less than four times the weld size, not to be less than 1 ½”.

### TABLE J2.4

<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 ¼ inclusive</td>
<td>¼</td>
</tr>
<tr>
<td>Over ¼ to ½</td>
<td>¾</td>
</tr>
<tr>
<td>Over ½ to ¾</td>
<td>¾</td>
</tr>
<tr>
<td>Over ¾</td>
<td>¾</td>
</tr>
</tbody>
</table>

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

For fillet welds: $R_a \leq R_n / \Omega$ or $R_u \leq \phi R_n$

where $R_u = \sum \gamma_i R_i$

for the weld metal: $R_n = 0.6F \cdot TL = SI$

$\phi = 0.75$ (LRFD) \hspace{1cm} $\Omega = 2.00$ (ASD)

where:

- $T$ is throat thickness
- $l$ is length of the weld

For a connected part, the other limit states for the base metal, such as tension yield, tension rupture, shear yield, or shear rupture must be considered.

### Available Strength of Fillet Welds per inch of weld ($\phi S$)

<table>
<thead>
<tr>
<th>Weld Size (in.)</th>
<th>E60XX (k/in.)</th>
<th>E70XX (k/in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\frac{3}{8}$</td>
<td>3.58</td>
<td>4.18</td>
</tr>
<tr>
<td>$\frac{1}{2}$</td>
<td>4.77</td>
<td>5.57</td>
</tr>
<tr>
<td>$\frac{5}{8}$</td>
<td>5.97</td>
<td>6.96</td>
</tr>
<tr>
<td>$\frac{3}{4}$</td>
<td>7.16</td>
<td>8.35</td>
</tr>
<tr>
<td>$\frac{7}{8}$</td>
<td>8.35</td>
<td>9.74</td>
</tr>
<tr>
<td>$\frac{1}{2}$</td>
<td>9.55</td>
<td>11.14</td>
</tr>
<tr>
<td>$\frac{3}{4}$</td>
<td>11.93</td>
<td>13.92</td>
</tr>
</tbody>
</table>

(Not considering increase in throat with submerged arc weld process)
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 bolt and angle available strength knowing number of bolts, bolt type, bolt diameter, angle leg thickness, hole type and coping, and the wide flange beam being connected. For the connections the limit-state of bolt shear, bolts bearing on the angles, shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles, and bolt bearing on the beam web are considered.

Group A bolts include A325, while Group B includes A490.

Here are also tables for bolted/welded double-angle connections and all-welded double-angle connections.

Sample AISC Table for Bolt and Angle Available Strength in All-Bolted Double-Angle Connections
Limiting Strength or Stability States

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

Block Shear Strength (or Rupture):

\[ R_u \leq R_n \big/ \Omega \text{ or } R_u \leq \phi R_n \]

where \( R_u = \sum \gamma_i R_i \)

\[ R_n = 0.6 F_y A_{nv} + U_{bs} F_u A_{nt} \leq 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} \]

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

where:

- \( A_{nv} \) is the net area subjected to shear
- \( A_{nt} \) is the net area subjected to tension
- \( A_{gv} \) is the gross area subjected to shear
- \( U_{bs} = 1.0 \) when the tensile stress is uniform (most cases)
- \( = 0.5 \) when the tensile stress is non-uniform

Local Buckling in Steel I Beams– Web Crippling or Flange Buckling

Concentrated forces on a steel beam can cause the web to buckle (called web crippling). Web stiffeners under the beam loads and bearing plates at the supports reduce that tendency. Web stiffeners also prevent the web from shearing in plate girders.
The maximum support load and interior load can be determined from:

\[ P_{\text{max-end}} = (2.5k + N)F_{yw}t_w \]
\[ P_{\text{interior}} = (5k + N)F_{yw}t_w \]

where \( t_w \) = thickness of the web
\( N \) = bearing length
\( k \) = dimension to fillet found in beam section tables

\( \phi = 1.00 \) (LRFD) \hspace{1cm} \( \Omega = 1.50 \) (ASD)