Examples:
Connections and Tension Members

Example 1
A nominal 4 x 6 in. redwood beam is to be supported by two 2 x 6 in. members acting as a spaced column. The minimum spacing and edge distances for the ½ inch bolts are shown. How many ½ in. bolts will be required to safely carry a load of 1500 lb? Use the chart provided.

SOLUTION:
The table requires that the length of the bolt in the main wood member be known, along with the diameter of bolt in inches, and if the bolt is seeing single shear or double shear and what direction it is bearing on the grain.

The main member is the beam. The 4 in. nominal size is actually 3 ½ in. finished:

The bolt is ½ inches in diameter, and sees two planes of shear at the interfaces with the 2 x 6’s. This means double shear.

The vertical force is pushing the beam down onto the bolt, so the bolt is in contact with the grain running horizontally. That means the bolt is bearing perpendicular to the grain, and we should look up q.

The allowable load per bolt multiplied by the number of bolts will determine the capacity, which we need to be at least 1500 lb:

\[ q \times n \geq P \]

knowing q & P, the equation for n becomes:

\[ n \geq \frac{P}{q} = \frac{1500\text{ lb}}{980\text{ lb/bolt}} = 1.5 \text{ bolt} \]

rounded up = 2 bolts required

Table: Holding Power of Bolts

<table>
<thead>
<tr>
<th>Length of Bolt in Main Wood Member</th>
<th>Diameter of Bolt (in Inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3/4</td>
</tr>
<tr>
<td>Single p</td>
<td>370</td>
</tr>
<tr>
<td>Shear q</td>
<td>430</td>
</tr>
<tr>
<td>Double p</td>
<td>650</td>
</tr>
<tr>
<td>Shear q</td>
<td>940</td>
</tr>
</tbody>
</table>

| Single p                          | 360 | 490 | 1155 | 1370 | 1575 |
| Shear q                           | 405 | 450 | 495  | 540  |
| Double p                          | 710 | 810 | 2310 | 2740 | 3150 |
| Shear q                           | 810 | 900 | 990  |      |      |

| 3/4                               | 360 | 490 | 1155 | 1370 | 1575 |
| Single p                          | 460 | 630 | 1400 | 1790 | 2135 |
| Shear q                           | 630 | 850 | 965  | 1260 | 1590 |
| Double p                          | 710 | 980 | 1980 | 2380 | 2820 |
| Shear q                           | 980 | 1200| 1390 | 1590 |      |

1Allowable values are on a normal load-duration basis and apply to joints made of seasoned lumber used in dry locations. See U.B.C. Standard No. 25-17 for other service conditions.
2Double shear values are for joints consisting of three wood members in which the side members are one half the thickness of the main member. Single shear values are for joints consisting of two wood members having a minimum thickness not less than that specified.
3The length specified is the length of the bolt in the main member of double shear joints or the length of the bolt in the thinner member of single shear joints.
4See U.B.C. Standard No. 25-17 for wood-to-metal bolted joints.
Example 2

8.11 A built-up plywood box beam with $2 \times 4$ S4S top and bottom flanges is held together by nails. Determine the pitch (spacing) of the nails if the beam supports a uniform load of 200 #/ft. along the 26-foot span. Assume the nails have a shear capacity of 80# each.

Solution:

Construct the shear ($V$) diagram to obtain the critical shear condition and its location.

Note that the condition of shear is critical at the supports and the shear intensity decreases as you approach the center line of the beam. This would indicate that the nail spacing $P$ varies from the support to midspan. Nails are closely spaced at the support, but increasing spacing occurs toward midspan, following the shear diagram.

\[ f_v = \frac{VQ}{lb} \]

\[ I_x = \frac{(4.5')(18')^3}{12} - \frac{(3.5')(15')^3}{12} = 1,202.6 \text{ in.}^4 \]

\[ Q = \sum A_y = (9')(\frac{1}{2}')(4.5') + (9')(\frac{1}{2}')(4.5') + (1.5')(3.5')(8.25') = 83.8 \text{ in}^3 \]

\[ f_{v\text{max}} = \frac{(2,600#)(83.3 \text{ in.}^3)}{(1,202.6 \text{ in.}^4)(\frac{1}{2}'' + \frac{1}{2}''')} = 180.2 \text{ psi} \]

\[ Q = A_y = (5.25 \text{ in.}^2)(8.25'') = 43.3 \text{ in.}^3 \]

Shear force \( f_v \times A_v \)

where:

\[ A_v = \text{shear area} \]

Assume:

\( nF = \text{Capacity of two nails (one each side) at the flange; representing two shear surfaces} \)

\( (n)F \geq f_v \times b \times p = \frac{VQ}{lb} \times bp \)

\( \therefore (n)F \geq p \times \frac{VQ}{I}; \quad p \leq \frac{(n)FI}{VQ} \)

At the maximum shear location (support) where \( V = 2,600# \)

\[ p = \frac{(2 \text{ nails} \times 80 #/\text{nail})(1,202.6 \text{ in.}^4)}{(2,600#)(43.3 \text{ in.}^3)} = 1.71'' \]
Example 3
10.2 The butt splice shown in Figure 10.22 uses two 8 x 
¾” plates to “sandwich” in the 8 × ½” plates being joined. 
Four ¾″ 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 conditions 
that governs the capacity of the connection. (The edge distance to the holes is 
presumed to be adequate.)

Shear: Using the AISC available shear in Table 7-3 (Group A):

\[ \phi R_n = 26.4 \text{k/bolt} \times 4 \text{ bolts} = 105.6 \text{k} \]

Bearing: Using the AISC available bearing in Table 7-4:

There are 4 bolts bearing on the center (1/2″) plate, while there are 4 bolts bearing on 
a total width of two sandwich plates (3/4″ total). The thinner bearing width will govern. 
Assume 3 in. spacing (center to center) of bolts. For A36 steel, \( F_u = 58 \text{ ksi} \).

\[ \phi R_n = 91.4 \text{k/bolt/in.} \times 0.5 \text{ in.} \times 4 \text{ bolts} = 182.8 \text{k} \]

Tension: The center plate is critical, again, because its thickness is less than the combined 
thicknesses of the two outer plates. We must consider tension yielding and tension rupture:

\[ \phi R_n = \phi F_y A_g \quad \text{and} \quad \phi R_n = \phi F_u A_e \] where \( A_e = A_{net} U \)

\[ A_g = 8 \text{ in.} \times \frac{1}{2} \text{ in.} = 4 \text{ in}^2 \]

\[ A_n = (8 \text{ in.} - 2 \text{ holes} \times 1.0 \text{ in.}) \times \frac{1}{2} \text{ in.} = 3.0 \text{ in}^2 \]

\[ A_{net} \] is the area resisting shear, \( A_{nt} \) is the area resisting tension, \( A_{gv} \) is the gross area resisting shear, and \( U = 1 \) when the tensile stress is uniform.

\[ \phi F_y A_g = 0.9 \times 36 \text{ ksi} \times 4 \text{ in}^2 = 129.6 \text{k} \]

\[ \phi F_u A_e = 0.75 \times 58 \text{ ksi} \times (1) \times 3.0 \text{ in}^2 = 130.5 \text{k} \]

The maximum connection capacity (smallest value) so far is governed by bolt shear: \( \phi R_n = 105.6 \text{k} \)

Block Shear Rupture: It is possible for the center plate to rip away from the sandwich plates 
leaving the block (shown hatched) behind:

\[ \phi R_n = \phi(0.6F_y A_{nv} + U_{bo} F_u A_{vo}) \leq \phi(0.6F_y A_{gv} + U_{bo} F_u A_{vo}) \]

where \( A_{nv} \) is the area resisting shear, \( A_{nt} \) is the area resisting tension, \( A_{gv} \) is the gross area resisting shear, and \( U_{bo} = 1 \) when the tensile stress is uniform.

\[ A_{gv} = 2 \times (4 + 2 \text{ in.}) \times \frac{1}{2} \text{ in.} = 6 \text{ in}^2 \]

\[ A_{nv} = A_{gv} - \frac{1}{2} \text{ holes areas} = 6 \text{ in}^2 - 1.5 \times 1 \text{ in.} \times \frac{1}{2} \text{ in.} = 5.25 \text{ in}^2 \]

\[ A_{nt} = 3.5 \text{ in.} \times t - 2(\frac{1}{2} \text{ hole areas}) = 3.5 \text{ in.} \times \frac{1}{2} \text{ in.} - 1 \times 1 \text{ in.} \times \frac{1}{2} \text{ in.} = 1.25 \text{ in}^2 \]

\[ \phi(0.6F_y A_{nv} + U_{bo} F_u A_{vo}) = 0.75 \times (0.6 \times 58 \text{ ksi} \times 5.25 \text{ in}^2 + 1 \times 58 \text{ ksi} \times 1.25 \text{ in}^2) = 191.4 \text{k} \]

\[ \phi(0.6F_y A_{gv} + U_{bo} F_u A_{vo}) = 0.75 \times (0.6 \times 36 \text{ ksi} \times 6 \text{ in}^2 + 1 \times 58 \text{ ksi} \times 1.25 \text{ in}^2) = 151.6 \text{k} \]

The maximum connection capacity (smallest value) is governed by block shear rupture: \( \phi R_n = 151.6 \text{k} \)
Example 4

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

Solution:

Capacity of weld:

For a $\frac{3}{16}$" fillet weld, $\phi S = 6.96$ k/in

Weld length = 8 in + 6 in + 8 in = 22 in.

Weld capacity = $22'' \times 6.96$ k/in = 153.1 k

Capacity of plate: $0.9 \times 36$ k/in$^2 \times 3/8'' \times 6'' = 72.9$ k

$\phi P_n = \phi e A_e \phi = 0.9$

Plate capacity = $0.9 \times 36$ k/in$^2 \times 3/8'' \times 6'' = 72.9$ k

$\therefore$ Plate capacity governs, $P_n = 72.9$ 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:

$22'' \times ($weld capacity per in.) = 72.9$ k

Weld capacity per inch = $\frac{72.9}{22}\text{ k/in.} = 3.31$ k/in.

From Available Strength table, use $3/16''$ weld ($\phi S = 4.18$ k/in.)

Minimum size fillet = $\frac{3}{32}$" based on a $\frac{1}{8}$" thick plate.

<table>
<thead>
<tr>
<th>Weld Size</th>
<th>E60XX (k/in.)</th>
<th>E70XX (k/in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\frac{3}{16}$</td>
<td>3.58</td>
<td>4.18</td>
</tr>
<tr>
<td>$\frac{1}{4}$</td>
<td>4.77</td>
<td>5.57</td>
</tr>
<tr>
<td>$\frac{5}{16}$</td>
<td>5.97</td>
<td>6.96</td>
</tr>
<tr>
<td>$\frac{3}{8}$</td>
<td>7.16</td>
<td>8.35</td>
</tr>
<tr>
<td>$\frac{1}{2}$</td>
<td>8.35</td>
<td>9.74</td>
</tr>
<tr>
<td>$\frac{5}{8}$</td>
<td>9.55</td>
<td>11.14</td>
</tr>
<tr>
<td>$\frac{7}{8}$</td>
<td>11.93</td>
<td>13.92</td>
</tr>
<tr>
<td>$\frac{1}{4}$</td>
<td>14.32</td>
<td>16.70</td>
</tr>
</tbody>
</table>

Table 7-1: Available Shear Strength of Bolts, kips

<table>
<thead>
<tr>
<th>Nominal Bolt Diameter, d, in</th>
<th>$\frac{3}{16}$</th>
<th>$\frac{1}{4}$</th>
<th>$\frac{5}{16}$</th>
<th>$\frac{3}{8}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal Bolt Area, in$^2$</td>
<td>0.307</td>
<td>0.442</td>
<td>0.601</td>
<td>0.785</td>
</tr>
<tr>
<td>ASTM Desig.</td>
<td>Thread Cond.</td>
<td>$f_{Pu/2}$ (kips)</td>
<td>$f_{Pu}$ (kips)</td>
<td>Loading</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$f_{Pu/2}$</td>
<td>$f_{Pu}$</td>
<td></td>
</tr>
<tr>
<td>Group A</td>
<td>N</td>
<td>27.0</td>
<td>40.5</td>
<td>S</td>
</tr>
<tr>
<td></td>
<td>X</td>
<td>34.0</td>
<td>51.0</td>
<td>S</td>
</tr>
<tr>
<td>Group B</td>
<td>N</td>
<td>34.0</td>
<td>51.0</td>
<td>S</td>
</tr>
<tr>
<td></td>
<td>X</td>
<td>42.0</td>
<td>63.0</td>
<td>S</td>
</tr>
<tr>
<td>A307</td>
<td>–</td>
<td>13.5</td>
<td>20.3</td>
<td>S</td>
</tr>
</tbody>
</table>

Table 7-2: Available Strength of Fillet Welds per inch of weld ($\phi S$)

<table>
<thead>
<tr>
<th>Weld Size</th>
<th>E60XX (k/in.)</th>
<th>E70XX (k/in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\frac{3}{16}$</td>
<td>3.58</td>
<td>4.18</td>
</tr>
<tr>
<td>$\frac{1}{4}$</td>
<td>4.77</td>
<td>5.57</td>
</tr>
<tr>
<td>$\frac{5}{16}$</td>
<td>5.97</td>
<td>6.96</td>
</tr>
<tr>
<td>$\frac{3}{8}$</td>
<td>7.16</td>
<td>8.35</td>
</tr>
<tr>
<td>$\frac{1}{2}$</td>
<td>8.35</td>
<td>9.74</td>
</tr>
<tr>
<td>$\frac{5}{8}$</td>
<td>9.55</td>
<td>11.14</td>
</tr>
<tr>
<td>$\frac{7}{8}$</td>
<td>11.93</td>
<td>13.92</td>
</tr>
<tr>
<td>$\frac{1}{4}$</td>
<td>14.32</td>
<td>16.70</td>
</tr>
</tbody>
</table>

(not considering increase in throat with submerged arc weld process)
Example 5

Determine the capacity of the framed beam connection. The ¼" thick angles are ASTM A36, while the column and the steel are A592 Grade 50. Assume standard holes and spacing of 3 in. with adequate edge distances for the angles.

**SOLUTION:**

Shear, bearing and angle capacity will be checked to determine the critical conditions that governs the capacity of the connection. (The edge distance to the holes is presumed to be adequate.)

**Shear:** Using the AISC available shear in Table 7-3 (Group A):

\[ \phi R_n = 35.8 \text{ k/bolt} \times 4 \text{ bolts} = 143.2 \text{ k} \]

**Angle Capacity:** Using the AISC all-bolted double angle connection available strength in Table 10-1

\[ \phi R_n = 101. \text{ k} \]

**Bearing:** Using the AISC available bearing in Table 7-4:

There are 4 bolts bearing on the beam web, while there are 8 bolts bearing on the column flange. The beam bearing (less bolts) will commonly govern. For A592 steel, \( F_u = 65 \text{ ksi} \).

- **Beam:** \( \phi R_n = 87.8 \text{ k/bolt/in.} \times 0.43 \text{ in.} \times 4 \text{ bolts} = 151.0 \text{ k} \)
- **Column:** \( \phi R_n = 87.8 \text{ k/bolt/in.} \times 0.575 \text{ in.} \times 8 \text{ bolts} = 403.9 \text{ k} \)

The maximum connection capacity (smallest value) is governed by the angle capacity.

\[ \phi R_n = 101 \text{ k} \]
Example 6

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.

W21 x 93: \( d = 21.62 \) in, \( t_w = 0.58 \) in, \( t_f = 0.93 \) in

W10 x 54: \( t_f = 0.615 \) 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:

\[
\text{Available length} = \text{beam depth} - \text{both flange thicknesses} - 1" \text{ clearance at top} & 1" \text{ at bottom} \\
= 21.62 \text{ in} - 2(0.93 \text{ in}) - 2(1 \text{ in}) = 17.76 \text{ in.}
\]

With the spaced at 3 in. and 1 ¼ in. end lengths (each end), the maximum number of bolts can be determined:

\[
\text{Available length} \geq 1.25 \text{ in.} + 1.25 \text{ in.} + 3 \text{ in.} \times (\text{number of bolts} - 1) \\
\text{number of bolts} \leq (17.76 \text{ in} - 2.5 \text{ in.} - (-3 \text{ in.}))/3 \text{ in.} = 6.1, \text{ so 6 bolts.}
\]

It is helpful to have the All-bolted Double-Angle Connection Tables 10-1. They are available for \( \frac{3}{4}" \), \( 7/8" \), and 1" bolt diameters and list angle thicknesses of \( \frac{1}{4}" \), 5/16", 3/8", and \( \frac{1}{2}" \). Increasing the angle thickness is likely to increase the angle strength, although the limit states include shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles.

For these diameters the available shear (double) from Table 7-1 for 6 bolts is (6)41.5 k/bolt = 270.6 kips, (6)61.3 k/bolt = 367.8 kips, and (6)80.1 k/bolt = 480.6 kips.

Tables 10-1 (not all provided here) list a bolt and angle available strength of 271 kips for the \( \frac{3}{4}" \) bolts, 296 kips for the \( 7/8" \) bolts, and 281 kips for the 1" bolts. It appears that increasing the bolt diameter to 1" will not gain additional load. Use 7/8" bolts.

\[
\phi R_n = 368.7 \text{ kips for double shear of } 7/8" \text{ bolts} \\
\phi R_n = 296 \text{ kips for limit state in angles}
\]

We also need to evaluate bearing of bolts on the beam web, and column flange where there are bolt holes. Table 7-5 provides available bearing strength for the material type, bolt diameter, hole type, and spacing per inch of material thicknesses.

a) Bearing for beam web: There are 6 bolt holes through the beam web. This is typically the critical bearing limit value because there are two angle legs that resist bolt bearing and twice as many bolt holes to the column. The material is A992 (\( F_u = 65 \) ksi), 0.58" thick, with 7/8" bolt diameters at 3 in. spacing.

\[
\phi R_n = 6 \text{ bolts} \times (102 \text{ k/bolt/inch}) \times (0.58 \text{ in}) = 355.0 \text{ kips}
\]

b) Bearing for column flange: There are 12 bolt holes through the column. The material is A992 (\( F_u = 65 \) ksi), 0.615" thick, with 1" bolt diameters.

\[
\phi R_n = 12 \text{ bolts} \times (102 \text{ k/bolt/inch}) \times (0.615 \text{ in}) = 752.8 \text{ kips}
\]

Although, the bearing in the beam web is the smallest at 355 kips, with the shear on the bolts even smaller at 324.6 kips, the maximum capacity for the simple-shear connector is 296 kips limited by the critical capacity of the angles.
### Table 7-4
Available Bearing Strength at Bolt Holes 
Based on Bolt Spacing 
kip/s in. thickness

<table>
<thead>
<tr>
<th>Hole Type</th>
<th>Bolt Spacing, d, in.</th>
<th>$F_a$, kip</th>
<th>$v_1$</th>
<th>$v_2$</th>
<th>$v_3$</th>
<th>$v_4$</th>
<th>$v_5$</th>
<th>$v_6$</th>
<th>$v_7$</th>
<th>$v_8$</th>
<th>$v_9$</th>
<th>$v_{12}$</th>
<th>$v_{13}$</th>
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<tbody>
<tr>
<td>STDSLTL</td>
<td>2$d_3$</td>
<td>58</td>
<td>34.1</td>
<td>51.1</td>
<td>41.3</td>
<td>62.0</td>
<td>48.6</td>
<td>72.9</td>
<td>55.8</td>
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<td>55.8</td>
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<tr>
<td></td>
<td>65</td>
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<td>46.3</td>
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<td>50.0</td>
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<td>43.6</td>
<td>74.3</td>
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<td>OVS/SSLPL</td>
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<td>56.7</td>
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</tr>
<tr>
<td>LSL</td>
<td>2$d_3$</td>
<td>58</td>
<td>38.9</td>
<td>46.3</td>
<td>39.0</td>
<td>58.5</td>
<td>47.1</td>
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<tr>
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<td>65.3</td>
<td>52.2</td>
<td>73.3</td>
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<td>91.4</td>
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</table>

### Table 7-3
Slip-Critical Connections
Available Shear Strength, kips 
(Class A Faying Surface, $\mu = 0.30$)

<table>
<thead>
<tr>
<th>Hole Type</th>
<th>Loading</th>
<th>$v_1$</th>
<th>$v_2$</th>
<th>$v_3$</th>
<th>$v_4$</th>
<th>$v_5$</th>
<th>$v_6$</th>
<th>$v_7$</th>
<th>$v_8$</th>
<th>$v_9$</th>
<th>$v_{12}$</th>
<th>$v_{13}$</th>
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<tbody>
<tr>
<td>STDSLTL</td>
<td>S</td>
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<td>3.64</td>
<td>3.63</td>
<td>4.49</td>
<td>8.81</td>
<td>13.2</td>
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<td>5.99</td>
<td>12.7</td>
<td>19.0</td>
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<td>D</td>
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<td>6.78</td>
<td>6.77</td>
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</tr>
<tr>
<td>OVS/SSLPL</td>
<td>S</td>
<td>3.66</td>
<td>5.47</td>
<td>5.39</td>
<td>6.07</td>
<td>7.51</td>
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<tr>
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<td>9.25</td>
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<td>5.67</td>
</tr>
</tbody>
</table>

### Notes
- $v_1$, $v_2$, ..., $v_{13}$ are the shear stress values at different bolt pretension levels.
- $\mu = 0.30$ is the friction coefficient for Class A faying surfaces.
- $F_a$, $v_1$, $v_2$, ..., $v_{13}$ are stresses in kips per inch.

### Symbols
- $F_a$: Bolt pretension in kip
- $v_1$, $v_2$, ..., $v_{13}$: Shear stress values
- $\mu$: Friction coefficient

### Definitions
- **STD**: Standard hole
- **SSLT**: Short-slotted hole oriented transverse to the line of force
- **OVS**: Oversized hole
- **SSLPL**: Long-slotted hole oriented parallel to the line of force
- **LSL**: Long-slotted hole oriented transverse to the line of force

Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force.