STEEL DESIGN

Chapter 7:
Bolted Connections

Dr. Hasan Katkhuda
Steel Design

Introduction

• Types of Bolts:
  1. Ordinary or Common bolts:
     • Classified by ASTM as A307 bolts.
     • Used in light structures subjected to static loading only.
  2. High strength bolts:
     • Classified by ASTM as A325, A490 bolts.
     • Have tensile strengths two or more times those of ordinary bolts.
     • Used in all types of structures (static + dynamic loads)
### Introduction

- **Simple Connections:**

  If the line of action of the resultant force to be resisted passes through the center of gravity of the connection, each part of the connection is assumed to resist an equal share of the load.

  ![Simple Connections Diagram](image)

- **Eccentrically Loaded Connections:**

  If the line of action of the resultant force to be resisted does not act through the center of gravity of the connection.

  ![Eccentrically Loaded Connections Diagram](image)
Bolted Shear Connections

• Failure modes:
  1. Shear failure of the bolts:

    (a) Single Shear
    (b) Double Shear

2. Tension failure in the member:
   • Yielding
   • Fracture
   • Block shear
Bolted Shear Connections

3. Bearing exerted by the bolts:

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Bolted Shear Connections

- Types of bolted shear connections:

1. **Bearing type connections:**
   - Slip is acceptable (loose in connection)
   - Load will be transferred through shear in bolts and bearing in the connected parts.

2. **Slip critical connections:**
   - No slippage is permitted (shear force < friction force)
   - No shear and bearing.
   - Load will be transferred through friction.

Bearing Type Connections

1. **Shear Strength:**
   
   \[ P = f_v A_b \]

   - \( f_v \): Shearing stress on the cross-sectional area of the bolt.
   - \( A_b \): Cross-sectional area of the unthreaded part of bolt.

   \[ R_n = F_{nv} A_b \]

   - \( R_n \): Nominal strength.
   - \( F_{nv} \): Nominal shear stress.
### Bearing Type Connections

#### TABLE J3.2
Nominal Stress of Fasteners and Threaded Parts, ksi (MPa)

<table>
<thead>
<tr>
<th>Description of Fasteners</th>
<th>Nominal Tensile Stress, $F_{tn}$ ksi (MPa)</th>
<th>Nominal Shear Stress in Bearing-Type Connections, $F_{mn}$ ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A307 bolts</td>
<td>45 (310) [A][B]</td>
<td>24 (165) [C][D][E]</td>
</tr>
<tr>
<td>A325 or A325M bolts, when threads are not excluded from shear planes</td>
<td>90 (620) [A]</td>
<td>48 (330) [D]</td>
</tr>
<tr>
<td>A325 or A325M bolts, when threads are excluded from shear planes</td>
<td>90 (620) [A]</td>
<td>60 (414) [D]</td>
</tr>
<tr>
<td>A490 or A490M bolts, when threads are not excluded from shear planes</td>
<td>113 (780) [A]</td>
<td>60 (414) [D]</td>
</tr>
<tr>
<td>A490 or A490M bolts, when threads are excluded from shear planes</td>
<td>113 (780) [A]</td>
<td>75 (520) [D]</td>
</tr>
</tbody>
</table>

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#### Bearing Type Connections

<table>
<thead>
<tr>
<th>Fastener</th>
<th>Nominal Shear Strength $R_n = F_{mn}A_b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A307</td>
<td>$24A_b$</td>
</tr>
<tr>
<td>A325, threads in plane of shear</td>
<td>$48A_b$</td>
</tr>
<tr>
<td>A325, threads not in plane of shear</td>
<td>$60A_b$</td>
</tr>
<tr>
<td>A490, threads in plane of shear</td>
<td>$60A_b$</td>
</tr>
<tr>
<td>A490, threads not in plane of shear</td>
<td>$75A_b$</td>
</tr>
</tbody>
</table>

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Steel Design
2. **Bearing Strength:**

- Bearing strength is independent of the type of fastener because the stress under consideration is on the part being connected rather than on the fastener.

\[
R_n = 1.2 L_c t F_{u} \leq 2.4 d t F_{u}
\]

\[
\phi R_n = 0.75 R_n
\]

where

- \(L_c\) = clear distance, in the direction parallel to the applied load, from the edge of the bolt hole to the edge of the adjacent hole or to the edge of the material
- \(t\) = thickness of the connected part
- \(F_{u}\) = ultimate tensile stress of the connected part (not the bolt)
Bearing Type Connections

For the edge bolts, use $L_c = L_e - h/2$. For other bolts, use $L_c = s - h$,

where

- $L_e$ = edge-distance to center of the hole
- $s$ = center-to-center spacing of holes
- $h$ = hole diameter

$h = d + \frac{1}{16}$ in.

---

Bearing Type Connections

- **Spacing and Edge distance requirements:**

Minimum spacing and edge distance: In any direction, both in the line of force and transverse to the line of force,

1. $s \geq 2 \frac{3}{8}d$ (preferably $3d$)
2. $L_e \geq$ value from AISC Table J3.4
Bearing Type Connections

### Table J3.4

<table>
<thead>
<tr>
<th>Bolt Diameter (in.)</th>
<th>At Sheared Edges</th>
<th>At Roiled Edges of Plates, Shapes or Bars, or Thermally Cut Edges[a]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2</td>
<td>1/8</td>
<td>3/8</td>
</tr>
<tr>
<td>3/8</td>
<td>1/4</td>
<td>1</td>
</tr>
<tr>
<td>7/16</td>
<td>1/4</td>
<td>1/2</td>
</tr>
<tr>
<td>1/2</td>
<td>1</td>
<td>1/2</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>2/3</td>
</tr>
<tr>
<td>Over 1 3/8</td>
<td>1 1/2 d</td>
<td>1 1/2 d</td>
</tr>
</tbody>
</table>

[a] Lesser edge distances are permitted to be used, provided provisions of Section J3.10, as appropriate, are satisfied.
[b] For oversized or slotted holes, see Table J3.5.
[c] All edge distances in this column are permitted to be reduced 1/8 in. when the hole is at a point where required strength does not exceed 25 percent of the maximum strength in the element.
[d] These are permitted to be 1 1/4 in. at the ends of beam connection angles and shear end plates.

Example (Bearing Type Connections)

- Check bolt spacing, edge distances and bearing in the connection shown.
- Bolts used A325 with threads not in plane of shear.
Example (Bearing Type Connections)

\[ 2^{3/4}d = 2.667 \left(\frac{3}{4}\right) = 2.00 \text{ in.} \]

Actual spacing = 2.50 in. > 2.00 in. \hspace{1cm} (OK)

The minimum edge distance in any direction is obtained from AISC Table J3.4. If we assume sheared edges (the worst case), the minimum edge distance is 1\(\frac{1}{4}\) in., so

Actual edge distance = 1 \(\frac{1}{4}\) in. \hspace{1cm} (OK)

Example (Bearing Type Connections)

1. Shear strength:

\[ R_n = F_m A_p \]

\[ = (60) (\pi)(3/4)^2 / 4 = 26.46 \text{ kips} \]

\[ \varphi R_n = (0.75)(26.46) = 19.845 \text{ kips (for each bolt)} \]

For four bolts:

\[ \varphi R_n = (4)(19.845) = 79.38 \text{ kips} \]
Example (Bearing Type Connections)

2. Bearing Strength:
   - **Tension member:**
   - **Edge holes:**
     \[
     h = d + \frac{1}{16} = \frac{3}{4} + \frac{1}{16} = \frac{13}{16} \text{ in.}
     \]
     \[
     L_c = L_e - \frac{h}{2} = 1.25 - \frac{13/16}{2} = 0.8438 \text{ in.}
     \]
     \[
     R_n = 1.2L_c t F_u \leq 2.4d t F_u
     \]
     \[
     1.2L_c t F_u = 1.2(0.8438) \left( \frac{1}{2} \right) (58) = 29.36 \text{ kips}
     \]

Example (Bearing Type Connections)

Check upper limit:

\[
2.4d t F_u = 2.4 \left( \frac{3}{4} \right) \left( \frac{1}{2} \right) (58) = 52.20 \text{ kips}
\]

29.36 kips < 52.20 kips  \(\therefore\) use \(R_n = 29.36 \text{ kips/bolt}\)

- **Other holes:**
  \[
  L_c = s - h = 2.5 - \frac{13}{16} = 1.688 \text{ in.}
  \]
  \[
  R_n = 1.2L_c t F_u \leq 2.4d t F_u
  \]
  \[
  1.2L_c t F_u = 1.2(1.688) \left( \frac{1}{2} \right) (58) = 58.74 \text{ kips}
  \]
Example (Bearing Type Connections)

Upper limit (the upper limit is independent of $L_c$ and is the same for all bolts):

$$2.4dt F_u = 52.20 \text{ kips} < 58.74 \text{ kips} \quad \therefore \text{use } R_n = 52.20 \text{ kips/bolt}$$

The bearing strength for the tension member is

$$R_n = 2(29.36) + 2(52.20) = 163.1 \text{ kips}$$

- **Gusset Plate:**
- **Edge holes:**

\[
L_c = L_e - \frac{h}{2} = 1.25 - \frac{13/16}{2} = 0.8438 \text{ in.}
\]

Example (Bearing Type Connections)

\[
R_n = 1.2L_c t F_u \leq 2.4dt F_u
\]

$$1.2L_c t F_u = 1.2(0.8438) \left(\frac{3}{8}\right) (58) = 22.02 \text{ kips}$$

Upper limit = 2.4$dtF_u = 2.4 \left(\frac{3}{4}\right) \left(\frac{3}{8}\right) (58) = 39.15 \text{ kips} > 22.02 \text{ kips} \quad \therefore \text{use } R_n = 22.02 \text{ kips/bolt}

- **Other holes:**

\[
L_c = s - h = 2.5 - \frac{13}{16} = 1.688 \text{ in.}
\]
**Example (Bearing Type Connections)**

\[ R_n = 1.2L_v t F_u \leq 2.4dt F_u \]

\[ 1.2L_v t F_u = 1.2(1.688) \left( \frac{3}{8} \right)(58) = 44.06 \text{ kips} \]

Upper limit = \( 2.4dt F_u = 39.15 \text{ kips} < 44.06 \text{ kips} \)

\[ \therefore \text{use } R_n = 39.15 \text{ kips/bolt} \]

The bearing strength for the gusset plate is

\[ R_n = 2(22.02) + 2(39.15) = 122.3 \text{ kips} \]

The gusset plate controls. The nominal bearing strength for the connection is therefore

\[ R_n = 122.3 \text{ kips} \]

The design strength is \( \phi R_n = 0.75(122.3) = 91.7 \text{ kips} \).

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**Example (Bearing Type Connections)**

- **Tension Failure:**
- **Yielding:**
  \[ \phi t P_n = (0.9)(36)(5)(0.5) = 81 \text{ kips} \]
- **Fracture:** ..................
- **Block shear:** ..................
- **Control (without calculating Tension failure) = 79.38 kips**
- \[ Ru = 1.2(15) + (1.6)(45) = 90 \text{ kips} \]
- \[ 79.38 < 90 \text{ kips (N.G)} \]
Slip Critical Connections

\[ R_n = \mu D_u h_{sc} T_f N_s \]

where
\( \mu \) = mean slip coefficient (coefficient of static friction) = 0.35 for Class A surfaces
\( D_u \) = ratio of mean actual bolt pretension to the specified minimum pretension
This is to be taken as 1.13 unless another factor can be justified.
\( h_{sc} \) = hole factor = 1.0 for standard holes
\( T_f \) = minimum fastener tension from AISC Table J3.1
\( N_s \) = number of slip planes (shear planes)

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**TABLE J3.1**
Minimum Bolt Pretension, kips*

<table>
<thead>
<tr>
<th>Bolt Size, in.</th>
<th>A325 Bolts</th>
<th>A490 Bolts</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/8</td>
<td>12</td>
<td>15</td>
</tr>
<tr>
<td>5/32</td>
<td>19</td>
<td>24</td>
</tr>
<tr>
<td>3/32</td>
<td>28</td>
<td>35</td>
</tr>
<tr>
<td>1/8</td>
<td>39</td>
<td>49</td>
</tr>
<tr>
<td>1/4</td>
<td>51</td>
<td>64</td>
</tr>
<tr>
<td>7/32</td>
<td>56</td>
<td>60</td>
</tr>
<tr>
<td>11/32</td>
<td>71</td>
<td>102</td>
</tr>
<tr>
<td>11/64</td>
<td>85</td>
<td>121</td>
</tr>
<tr>
<td>11/32</td>
<td>103</td>
<td>148</td>
</tr>
</tbody>
</table>

*Equal to 0.70 times the minimum tensile strength of bolts, rounded off to nearest kip, as specified in ASTM specifications for A325 and A490 bolts with UNC threads.
### Slip Critical Connections

If slip is treated as a serviceability limit state, then

$$\phi = 1.0$$

If slip is treated as a strength limit state,

$$\phi = 0.85$$

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### Example (Slip Critical Connections)

- ¾ inch diameter, A325 bolts with threads in the shear plane no slip is permitted, A36.

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(a) [Diagram of connection setup]

(b) [Diagram of gusset plate and tension member]
Example (Slip Critical Connections)

Shear strength: For one bolt,

\[ A_b = \frac{\pi (3/4)^2}{4} = 0.4418 \text{ in.}^2 \]

\[ R_n = F_w A_b = 48(0.4418) = 21.21 \text{ kips/bolt} \]

For four bolts,

\[ R_n = 4(21.21) = 84.84 \text{ kips} \]

Slip-critical strength: Because no slippage is permitted, this connection is classified as slip-critical (and we will treat slip as a serviceability limit state). From AISC Table J3-1, the minimum bolt tension is \( T_B = 28 \text{ kips} \). From AISC Equation J3-4,

\[ R_n = \mu D h B N_s = 0.35(1.13)(1.0)(28)(1.0) = 11.07 \text{ kips/bolt} \]

For four bolts,

\[ R_n = 4(11.07) = 44.28 \text{ kips} \]

Example (Slip Critical Connections)

Bearing strength: Since both edge distances are the same, and the gusset plate is thinner than the tension member, the gusset plate thickness of \( \frac{3}{8} \) inch will be used.

\[ h = d + \frac{1}{16} = \frac{3}{4} + \frac{1}{16} = \frac{13}{16} \text{ in.} \]

For the holes nearest the edge of the gusset plate,

\[ L_e = L - \frac{h}{2} = 1.5 - \frac{13/16}{2} = 1.094 \text{ in.} \]

\[ R_n = 1.2 L_e f_u = 1.2(1.094)(\frac{3}{8}) \overset{(58)}{= 28.55 \text{ kips}} \]

Upper limit = 2.4 \( d t f_u \)

\[ = 2.4 \left( \frac{3}{4} \right) \left( \frac{3}{8} \right) \overset{(58)}{= 28.55 \text{ kips}} \]

\[ = 39.15 \text{ kips} > 28.55 \text{ kips} \quad \therefore \text{use } R_n = 28.55 \text{ kips for this bolt} \]
**Example (Slip Critical Connections)**

For the other holes,

\[ L_c = s - h = 3 - \frac{13}{16} = 2.188 \text{ in.} \]

\[ R_e = 1.2L_c t F_p = 1.2(2.188) \left( \frac{3}{8} \right) (58) = 57.11 \text{ kips} \]

Upper limit \( = 2.4d t F_p \)

\[ = 39.15 \text{ kips} < 57.11 \text{ kips} \quad \text{use } R_e = 39.15 \text{ kips for this bolt} \]

The nominal bearing strength for the connection is

\[ R_n = 2(28.55) + 2(39.15) = 135.4 \text{ kips} \]

---

**Example (Slip Critical Connections)**

**Tension on the gross area:**

\[ P_n = F_s A_g = 36 \left( 6 \times \frac{1}{2} \right) = 108.0 \text{ kips} \]

**Tension on the net area:** All elements of the cross section are connected, so shear lag is not a factor and \( A_v = A_m \). For the hole diameter, use

\[ h = d + \frac{1}{8} = \frac{3}{4} + \frac{1}{8} = \frac{7}{8} \text{ in.} \]

The nominal strength is

\[ P_n = F_s A_v = F_s t (w_e - \Sigma h) = 58 \left( \frac{1}{2} \right) \left[ 6 - 2 \left( \frac{7}{8} \right) \right] = 123.3 \text{ kips} \]
Example (Slip Critical Connections)

**Block shear strength:**

\[
A_w = 2 \times \frac{3}{8} \left(3 + 1.5 \right) = 3.375 \text{ in}^2
\]

Since there are 1.5 hole diameters per horizontal line of bolts,

\[
A_w = 2 \times \frac{3}{8} \left[ 3 + 1.5 - 1.5 \left( \frac{7}{8} \right) \right] = 2.391 \text{ in}^2
\]

For the tension area,

\[
A_t = \frac{3}{8} \left( 3 - \frac{7}{8} \right) = 0.7969 \text{ in}^2
\]

Since the block shear will occur in a gusset plate, \(U_n = 1.0\). From AISC Equation J4-5,

\[
R_n = 0.6F_vA_w + U_nF_vA_w = 0.6(58)(2.391) + 1.0(58)(0.7969) = 129.4 \text{ kips}
\]

with an upper limit of

\[
0.6F_vA_w + U_nF_vA_w = 0.6(36)(3.375) + 1.0(58)(0.7969) = 119.1 \text{ kips}
\]

The nominal block shear strength is therefore 119.1 kips.

---

Example (Slip Critical Connections)

**Bolt shear strength:**

\[
\phi R_n = 0.75(84.84) = 63.6 \text{ kips}
\]

**Slip-critical strength:** Since slip is being treated as a serviceability limit state, \(\phi = 1.0\).

\[
\phi R_n = 1.0(44.28) = 44.3 \text{ kips}
\]

**Bearing strength:**

\[
\phi R_n = 0.75(135.4) = 102 \text{ kips}
\]

**Tension on the gross area:**

\[
\phi P_n = 0.90(108.0) = 97.2 \text{ kips}
\]

**Tension on the net area:**

\[
\phi P_n = 0.75(123.3) = 92.5 \text{ kips}
\]

**Block shear strength:**

\[
\phi R_n = 0.75(119.1) = 89.3 \text{ kips}
\]

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Design Example

The C8 × 18.7 shown in Figure 7.15 has been selected to resist a service dead load of 18 kips and a service live load of 54 kips. It is to be attached to a 3/8-inch gusset plate with 3/8-inch-diameter, A325 bolts. Assume that the threads are in the plane of shear and that slip of the connection is permissible. Determine the number and required layout of bolts such that the length of connection L is a minimum. A36 steel is used.

Design Example

Shear:

\[ A_b = \frac{\pi (7/8)^2}{4} = 0.6013 \text{ in.}^2 \]

\[ R_n = F_{n}, A_b = 48(0.6013) = 28.86 \text{ kips/bolt} \]

Bearing: The gusset plate is thinner than the web of the channel and will control. Assume that along a line parallel to the force, the length \( L_c \) is large enough so that the upper limit will control. Then

\[ R_n = 2.4 \times d F_u = 2.4 \left( \frac{7}{8} \right) \left( \frac{3}{8} \right) (58) = 45.68 \text{ kips} \]

and shear controls. The bearing strength will need to be verified once the actual bolt layout is determined.
Design Example

The factored load is

\[ 1.2D + 1.6L = 1.2(18) + 1.6(54) = 108.0 \text{ kips} \]

The design strength per bolt, based on shear, is

\[ \phi R_n = 0.75(28.86) = 21.65 \text{ kips} \]

The number of bolts required is

\[ \frac{108}{21.65} = 4.99 \text{ bolts} \]

Design Example

\[ P_n = F_e A_e = 36(5.51) = 198.4 \text{ kips} \]

The design strength is

\[ \phi P_n = 0.90(198.4) = 179 \text{ kips} \]

Tension on the effective net area:

\[ A_n = 5.51 - 2 \left( \frac{7}{8} + \frac{1}{8} \right)(0.487) = 4.536 \text{ in.}^2 \]

\[ A_e = A_n U = 4.536(0.60) = 2.722 \text{ in.}^2 \]

\[ P_n = F_e A_e = 58(2.722) = 157.9 \text{ kips} \]

\[ \phi P_n = 0.75(157.9) = 118 \text{ kips} \quad \text{(controls)} \]
Design Example

Minimum spacing = \(2.667\left(\frac{7}{8}\right) = 2.33\) in.

From AISC Table J3.4,
Minimum edge distance = \(1\frac{1}{8}\) in.

Design Example

\[
h = d + \frac{1}{16} = \frac{7}{8} + \frac{1}{16} = \frac{15}{16} \text{ in.}
\]

For the holes nearest the edge of the gusset plate,
\[
L_e = L_e - \frac{h}{2} = 1.125 - \frac{15/16}{2} = 0.6563 \text{ in.}
\]
\[
R_u = 1.2L_eF_u = 1.2(0.6563)\left(\frac{3}{8}\right) (58) = 17.13 \text{ kips}
\]
Upper limit = \(2.4dF_u = 2.4\left(\frac{7}{8}\right)\left(\frac{3}{8}\right) (58)
= 45.68 \text{ kips} > 17.13 \text{ kips} \quad \therefore \text{use } R_u = 17.13 \text{ kips for this bolt}
\]
Design Example

For the other holes,

\[ L_v = s - h = 2.5 - \frac{15}{16} = 1.563 \text{ in.} \]

\[ R_n = 1.2 L_v tF_{u} = 1.2 (1.563) \left( \frac{3}{8} \right) (58) = 40.79 \text{ kips} \]

Upper limit = \( 2.4 dtF_{u} \)

\[ = 45.68 \text{ kips} > 40.79 \text{ kips} \]

\[ \therefore \text{use } R_n = 40.79 \text{ kips for this bolt} \]

The total nominal bearing strength for the connection is

\[ R_n = 2(17.13) + 4(40.79) = 197.4 \text{ kips} \]

The design bearing strength is

\[ \phi R_n = 0.75 (197.4) = 148 \text{ kips} > P_u = 108 \text{ kips } \text{(OK)} \]

Design Example

Shear areas:

\[ A_{sv} = 2 \times \frac{3}{8} (2.5 + 2.5 + 1.125) = 4.594 \text{ in.}^2 \]

\[ A_{nv} = 2 \times \frac{3}{8} [6.125 - 2.5(1.0)] = 2.719 \text{ in.}^2 \]

Tension area:

\[ A_{nt} = \frac{3}{8} (3 - 1.0) = 0.7500 \text{ in.}^2 \]

\[ R_n = 0.6F_uA_{nv} + U_{ku} F_u A_{nt} \]

\[ = 0.6(58)(2.719) + 1.0(58)(0.7500) = 138.1 \text{ kips} \]

with an upper limit of

\[ 0.6F_u A_{nv} + U_{ku} F_u A_{nt} = 0.6(36)(4.594) + 1.0(58)(0.7500) = 142.7 \text{ kips} \]
Design Example

The nominal block shear strength is therefore 138.1 kips, and the design strength is

$$\phi R_u = 0.75(138.1) = 104 \text{ kips} < 108 \text{ kips} \quad \text{(N.G.)}$$

Design Example

$$\phi R_u = 0.75(0.6F_u A_{nv} + U_{hs} F_u A_{nt})$$
$$= 0.75[0.6(58)A_{nv} + 1.0(58)(0.7500)] = 108 \text{ kips}$$

Required $A_{nv} = 2.888 \text{ in.}^2$

$$A_{nv} = \frac{3}{8} [s + s + 1.125 - 2.5(1.0)](2) = 2.888 \text{ in.}^2$$

Required $s = 2.61 \text{ in.} \quad \therefore \text{use } s = 3 \text{ in.}$

Compute the actual block shear strength.

$$A_{nv} = 2 \times \frac{3}{8} (3 + 3 + 1.125) = 5.344 \text{ in.}^2$$

$$A_{nv} = 5.344 - \frac{3}{8} (2.5 \times 1.0)(2) = 3.469 \text{ in.}^2$$

$$\phi R_u = 0.75(0.6F_u A_{nv} + U_{hs} F_u A_{nt})$$
$$= 0.75[0.6(58)(3.469) + 1.0(58)(0.7500)] = 123 \text{ kips} > 108 \text{ kips} \quad \text{(OK)}$$
Design Example

Check the upper limit:

\[ \phi [0.6F_{a}\text{A}_{v} + U_{b}\text{F}_{w}\text{A}_{w}] = 0.75[0.6(36)(5.344) + 1.0(58)(0.7500)] \]

\[ = 119 \text{ kips} < 123 \text{ kips} \]

Therefore, the upper limit controls, but the strength is still adequate.

Using the spacing and edge distances selected, the minimum length is, therefore,

\[ L = 1\frac{1}{2} \text{ in. at the end of the channel} \]

+ 2 spaces at 3 in.

+ 1\frac{1}{2} \text{ in. at the end of the gusset plate}

= 8\frac{1}{4} \text{ in. total} \]

Bolts Subjected to Shear and Tension

- The vertical component of the force will put the bolts in shear, while the horizontal component will cause tension on the bolts.
Bolts Subjected to Shear and Tension

- Bearing Type connection:

\[
\left( \frac{f_t}{F_t} \right)^2 + \left( \frac{f_v}{F_v} \right)^2 = 1.0
\]

\[
\left( \frac{f_t}{F_t} \right) + \left( \frac{f_v}{F_v} \right) = 1.3
\]

Bolts Subjected to Shear and Tension

\[
F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \leq F_{nt}
\]

where \( \phi = 0.75 \).

where
- \( F'_{nt} \) = nominal tensile stress in the presence of shear
- \( F_{nt} \) = nominal tensile stress in the absence of shear
- \( F_{nv} \) = nominal shear stress in the absence of tension
- \( f_v \) = required shear stress

Bearing-type connections:
1. Check shear and bearing against the usual strengths.
2. Check tension against the reduced tensile strength usi
Example (Bolts Subjected to Shear and Tension)

A WT10.5 × 31 is used as a bracket to transmit a 60-kip service load to a W14 × 90 column, as previously shown in Figure 7.30. The load consists of 15 kips dead load and 45 kips live load. Four 7/8-inch-diameter A325 bolts are used. The column is of A992 steel, and the bracket is A36. Assume all spacing and edge-distance requirements are satisfied, including those necessary for the use of the maximum nominal strength in bearing (i.e., \(2.4 \times \delta F_u\)), and determine the adequacy of the bolts for the fol-

\[
R_n = 2.4 \delta F_u = 2.4 \left(\frac{7}{8}\right)(0.615)(58) = 74.91 \text{ kips}
\]

Nominal shear strength:

\[
A_b = \frac{\pi (7/8)^2}{4} = 0.6013 \text{ in.}^2
\]

\[
R_n = F_{nv} A_b = 48(0.6013) = 28.9 \text{ kips}
\]

\[
P_u = 1.2D + 1.6L = 1.2(15) + 1.6(45) = 90 \text{ kips}
\]
Example (Bolts Subjected to Shear and Tension)

The total shear/bearing load is
\[ V_u = \frac{3}{5} \times 90 = 54 \text{ kips} \]

The shear/bearing force per bolt is
\[ V_{u,bolt} = \frac{54}{4} = 13.5 \text{ kips} \]

The design bearing strength is
\[ \phi R_u = 0.75(74.91) = 56.2 \text{ kips} > 13.5 \text{ kips} \quad \text{(OK)} \]

The design shear strength is
\[ \phi R_u = 0.75(28.9) = 21.7 \text{ kips} > 13.5 \text{ kips} \quad \text{(OK)} \]

Example (Bolts Subjected to Shear and Tension)

The total tension load is
\[ T_u = \frac{4}{5} \times 90 = 72 \text{ kips} \]

The tensile force per bolt is
\[ T_{u,bolt} = \frac{72}{4} = 18 \text{ kips} \]

\[ F'_{nt} = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \leq F_{nt} \]

where
\[ F_{nt} = \text{nominal tensile stress in the absence of shear} = 90 \text{ ksi} \]
\[ F_{nv} = \text{nominal shear stress in the absence of tension} = 48 \text{ ksi} \]
\[ f_v = \frac{\nu_{u,bolt}}{A_0} = \frac{13.5}{0.6013} = 22.45 \text{ ksi} \]
Example (Bolts Subjected to Shear and Tension)

\[ F'_{u'} = 1.3(90) - \frac{90}{0.75(48)}(22.45) = 60.88 \text{ ksi} < 90 \text{ ksi} \]

The nominal tensile strength is

\[ R_n = F'_{u'}A_b = 60.88(0.6013) = 36.61 \text{ kips} \]

and the available tensile strength is

\[ \phi R_n = 0.75(36.61) = 27.5 \text{ kips} > 18 \text{ kips} \quad (OK) \]

Bolts Subjected to Shear and Tension

- **Slip critical connections:**

\[ k_s = 1 - \frac{T_u}{D_u T_b N_b} \]

where

- \( T_u \) = total factored tensile load on the connection
- \( D_u \) = ratio of mean bolt pretension to specified minimum pretension; default value is 1.13
- \( T_b \) = prescribed initial bolt tension from AISC Table J3.1
- \( N_b \) = number of bolts in the connection

Slip-critical connections:
1. Check tension, shear, and bearing against the usual strengths.
2. Check the slip-critical load against the reduced slip-critical strength.
Example (Bolts Subjected to Shear and Tension- slip critical connection)

A WT10.5 × 31 is used as a bracket to transmit a 60-kip service load to a W14 × 90 column, as previously shown in Figure 7.30. The load consists of 15 kips dead load and 45 kips live load. Four 7/8-inch-diameter A325 bolts are used. The column is of A992 steel, and the bracket is A36. Assume all spacing and edge-distance requirements are satisfied, including those necessary for the use of the maximum nominal strength in bearing (i.e., $2.4d_tF_p$), and determine the adequacy of the bolts for the fol-

![Diagram of WT10.5 × 31 and W14 × 90 with 4 bolts]

Example (Bolts Subjected to Shear and Tension- slip critical connection)

\[ R_n = \mu D_t h_a T_p N_r \times 4 = 0.35(1.13)(1.0)(39)(1) \times 4 = 61.70 \text{ kips} \]
\[ \phi R_n = 1.0(61.70) = 61.70 \text{ kips} \]

\[ k_s = 1 - \frac{T_u}{D_t T_p N_b} = 1 - \frac{72}{1.13(39)(4)} = 0.5916 \]

The reduced strength is therefore
\[ k_s(61.70) = 0.5916(61.70) = 36.5 \text{ kips} < 54 \text{ kips} \quad \text{(N.G.)} \]

The connection is inadequate as a slip-critical connection.
Eccentric Connections (Shear Only)

- **Elastic Analysis:**

\[ P = M = P_c \]

(a) + (b)
Eccentric Connections (Shear Only)

\[ p_c = \frac{P}{n}, \]

\[ f_v = \frac{Md}{J} \]

where

\( d = \) distance from the centroid of the area to the point where the stress is being computed

\( J = \) polar moment of inertia of the area about the centroid

- \( f_v \): Shearing stress in each bolt.

Eccentric Connections (Shear Only)

\[ J = \sum Ad^2 = A \sum d^2 \]

\[ f_v = \frac{Md}{A \sum d^2} \]

\[ p_m = Af_v = A \frac{Md}{A \sum d^2} = \frac{Md}{\sum d^2} \]

\[ p_{cx} = \frac{P_x}{n} \text{ and } p_{cy} = \frac{P_y}{n} \]
Eccentric Connections (Shear Only)

\[ \sum d^2 = \sum (x^2 + y^2) \]

\[ p_{\text{mx}} = \frac{y}{d} p_m = \frac{y}{d} \frac{Md}{\sum d^2} = \frac{y}{d} \frac{Md}{\sum (x^2 + y^2)} = \frac{My}{\sum (x^2 + y^2)} \]

\[ p_{\text{my}} = \frac{Mx}{\sum (x^2 + y^2)} \]

and the total fastener force is

\[ p = \sqrt{\left(\sum p_x\right)^2 + \left(\sum p_y\right)^2} \]

\[ \sum p_x = p_{cx} + p_{mx} \]

\[ \sum p_y = p_{cy} + p_{my} \]

Example (Eccentric Connections (Elastic Analysis))

Determine the critical fastener force in the bracket connection
Example (Eccentric Connections (Elastic Analysis))

\[
\bar{y} = \frac{2(5) + 2(8) + 2(11)}{8} = 6 \text{ in.}
\]

\[
P_x = \frac{1}{\sqrt{5}} (50) = 22.36 \text{ kips} \quad \text{and} \quad P_y = \frac{2}{\sqrt{5}} (50) = 44.72 \text{ kips}
\]

\[
M = 44.72(12 + 2.75) - 22.36(14 - 6) = 480.7 \text{ in.-kips} \quad \text{(clockwise)}
\]

Example (Eccentric Connections (Elastic Analysis))

\[
p_{cx} = \frac{22.36}{8} = 2.795 \text{ kips} \quad \text{and} \quad p_{cy} = \frac{44.72}{8} = 5.590 \text{ kips}
\]

\[
\Sigma(x^2 + y^2) = 8(2.75)^2 + 2[(6)^2 + (1)^2 + (2)^2 + (5)^2] = 192.5 \text{ in.}^2
\]

\[
p_{mx} = \frac{M_y}{\Sigma(x^2 + y^2)} = \frac{480.7(6)}{192.5} = 14.98 \text{ kips}
\]

\[
p_{my} = \frac{M_x}{\Sigma(x^2 + y^2)} = \frac{480.7(2.75)}{192.5} = 6.867 \text{ kips}
\]

\[
\Sigma p_x = 2.795 + 14.98 = 17.78 \text{ kips} \quad \text{and} \quad \Sigma p_y = 5.590 + 6.867 = 12.46 \text{ kips}
\]

\[
p = \sqrt{(17.78)^2 + (12.46)^2} = 21.7 \text{ kips}
\]
Eccentric Connections (Shear Only)

- **Ultimate Strength Analysis:**

![Diagram](image)

---

Eccentric Connections (Shear Only)

The bolt force $R$ corresponding to a deformation $\Delta$ is

$$R = R_{ult}(1 - e^{-\mu \Delta})^\lambda$$

where

- $R_{ult}$ = bolt shear force at failure
- $\mu$ = a regression coefficient = 10
- $\lambda$ = a regression coefficient = 0.55

1. At failure, the fastener group rotates about an instantaneous center (IC).
2. The deformation of each fastener is proportional to its distance from the IC and acts perpendicularly to the radius of rotation.
3. The capacity of the connection is reached when the ultimate strength of the fastener farthest from the IC is reached.
### Eccentric Connections (Shear Only)

\[ \Delta = \frac{r}{r_{\text{max}}} \Delta_{\text{max}} = \frac{r}{r_{\text{max}}} (0.34) \]

where
- \( r \) = distance from the IC to the fastener
- \( r_{\text{max}} \) = distance to the farthest fastener
- \( \Delta_{\text{max}} \) = deformation of the farthest fastener at ultimate = 0.34 in. (determined experimentally)

\[ R_y = \frac{x}{r} R \quad \text{and} \quad R_x = \frac{y}{r} R \]

\[ \Sigma F_x = \sum_{n=1}^{m} (R_x)_n - P_x = 0 \]

\[ \Sigma F_y = \sum_{n=1}^{m} (R_y)_n - P_y = 0 \]

### Example (Eccentric Connections (Ultimate Analysis))

The bracket connection shown must support an eccentric load consisting of 9 kips of dead load and 27 kips of live load. The connection was designed to have two vertical rows of four bolts, but one bolt was inadvertently omitted. If 5/8-inch-diameter A325 bearing-type bolts are used, is the connection adequate? Assume that the bolt threads are in the plane of shear. Use A36 steel for the bracket, A992 steel for the W6 × 25.
Example (Eccentric Connections (Ultimate Analysis))

Compute the bolt shear strength.

\[ A_b = \frac{\pi(7/8)^2}{4} = 0.6013 \text{ in.}^2 \]

\[ R_u = F_{w}A_b = 48(0.6013) = 28.86 \text{ kips} \]

\[ \cdot \]

For the bearing strength, use a hole diameter of

\[ h = d + \frac{1}{16} = \frac{7}{8} + \frac{1}{16} = \frac{15}{16} \text{ in.} \]

For the holes nearest the edge, use

\[ L_e = \frac{h}{2} = \frac{15/16}{2} = 1.531 \text{ in.} \]

Example (Eccentric Connections (Ultimate Analysis))

The strength of the W6 × 25 will control.

\[ R_u = 1.2L_eF_w = 1.2(1.531)(0.455)(65) = 54.34 \text{ kips} \]

Upper limit = \( 2.4dtF_w = 2.4\left(\frac{7}{8}\right)(0.455)(65) \)

\[ = 62.11 \text{ kips} > 54.34 \text{ kips} \quad \therefore \text{use } R_u = 54.34 \text{ kips for this bolt} \]

For the other holes, use \( s = 3 \text{ in.} \). Then,

\[ L_e = s - h = 3 - \frac{15}{16} = 2.063 \text{ in.} \]

\[ R_u = 1.2L_eF_w = 1.2(2.063)(0.455)(65) = 73.22 \text{ kips} \]

\[ 2.4dtF_w = 62.11 \text{ kips} < 73.22 \text{ kips} \quad \therefore \text{use } R_u = 62.11 \text{ kips for these bolts} \]

Both bearing values are larger than the bolt shear strength, so the nominal shear strength of \( R_u = 28.86 \text{ kips} \) controls.
Example (Eccentric Connections (Ultimate Analysis))

![Diagram of Eccentric Connections](image)

<table>
<thead>
<tr>
<th>Fastener</th>
<th>Origin at Bolt 1</th>
<th>Origin at IC</th>
<th>r</th>
<th>Δ</th>
<th>R</th>
<th>rR</th>
<th>Ry</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.000, 0.000</td>
<td>0.286, -3.857</td>
<td>3.888</td>
<td>0.255</td>
<td>70.774</td>
<td>273.731</td>
<td>5.221</td>
</tr>
<tr>
<td>2</td>
<td>3.000, 0.000</td>
<td>3.285, -3.857</td>
<td>5.067</td>
<td>0.334</td>
<td>72.553</td>
<td>367.599</td>
<td>47.045</td>
</tr>
<tr>
<td>3</td>
<td>0.000, 3.000</td>
<td>0.286, -0.857</td>
<td>9.903</td>
<td>0.060</td>
<td>47.649</td>
<td>43.046</td>
<td>15.050</td>
</tr>
<tr>
<td>4</td>
<td>3.000, 3.000</td>
<td>3.285, -0.857</td>
<td>3.395</td>
<td>0.224</td>
<td>69.563</td>
<td>236.188</td>
<td>67.310</td>
</tr>
<tr>
<td>5</td>
<td>0.000, 6.000</td>
<td>0.286, 2.143</td>
<td>2.162</td>
<td>0.143</td>
<td>63.631</td>
<td>137.555</td>
<td>8.398</td>
</tr>
<tr>
<td>6</td>
<td>3.000, 6.000</td>
<td>3.285, 2.143</td>
<td>3.922</td>
<td>0.259</td>
<td>70.891</td>
<td>278.061</td>
<td>59.377</td>
</tr>
<tr>
<td>7</td>
<td>0.000, 9.000</td>
<td>0.286, 5.143</td>
<td>5.151</td>
<td>0.340</td>
<td>72.631</td>
<td>374.107</td>
<td>4.023</td>
</tr>
</tbody>
</table>

Sum: 1710.287, 206.424

Dr. Hasan Katkhuda
Steel Design
Example (Eccentric Connections (Ultimate Analysis))

\[ P(r_0 + e) = \sum rR \]
\[ P = \frac{\sum rR}{r_0 + e} = \frac{1710.29}{1.57104 + 6.71429} = 206.424 \text{kips} \]

\[ \sum F_y = \sum R_y - P = 206.424 - 206.424 = 0.000 \]

\[ P\left(\frac{R_y}{74}\right) = 206.4\left(\frac{28.86}{74}\right) = 80.50 \text{kips} \]

The design strength of the connection is

0.75(80.50) = 60.4 kips > 54 kips (OK)

Example (Eccentric Connections, Ultimate Analysis, Tables)

- Bolts are ¾ inch
- A325 bearing with threads in plane of shear.
- Bolts are in single shear.
Example (Eccentric Connections, Ultimate Analysis, Tables)

This connection corresponds to the connections in Table 7-8, for Angle = 0°. The eccentricity is

\[ e_s = 8 + 1.5 = 9.5 \text{ in.} \]

The number of bolts per vertical row is

\[ n = 3 \]

From Table 7-8,

\[ C = 1.53 \text{ by interpolation} \]

The nominal strength of a \( \frac{3}{4} \)-inch-diameter bolt in single shear is

\[ r_n = F_{wv} A_p = 48(0.4418) = 21.21 \text{ kips} \]

(Here we use \( r_n \) for the nominal strength of a single bolt and \( R_n \) for the strength of the connection.)

The nominal strength of the connection is

\[ R_n = C r_n = 1.53(21.21) = 32.45 \text{ kips} \]

\[ \phi R_n = 0.75(32.45) = 24.3 \text{ kips}. \]