9. **Shear Lug Design**

Normally, friction and the shear capacity of the anchors used in a foundation adequately resist column base shear forces. In some cases, however, the engineer may find the shear force too great and may be required to transfer the excess shear force to the foundation by another means. If the total factored shear loads are transmitted through shear lugs or friction, the anchor bolts need not be designed for shear.

A shear lug (a plate or pipe stub section, welded perpendicularly to the bottom of the base plate) allows for complete transfer of the force through the shear lug, thus taking the shear load off of the anchors. The bearing on the shear lug is applied only on the portion of the lug adjacent to the concrete. Therefore, the engineer should disregard the portion of the lug immersed in the top layer of grout and uniformly distribute the bearing load through the remaining height.
The shear lug should be designed for the applied shear portion not resisted by friction between the base plate and concrete foundation. Grout must completely surround the lug plate or pipe section and must entirely fill the slot created in the concrete. When using a pipe section, a hole approximately 2 inches in diameter should be drilled through the base plate into the pipe section to allow grout placement and inspection to assure that grout is filling the entire pipe section.

9.1 Calculating Shear Load Applied to Shear Lug

The applied shear load, \( V_{\text{app}} \), used to design the shear lug should be computed as follows:

\[
V_{\text{app}} = V_{\text{ua}} - V_{\text{f}}
\]

9.2 Design Procedure for Shear Lug Plate

Design of a shear lug plate follows (for an example calculation, see Appendix Example 3, this Practice):

a. Calculate the required bearing area for the shear lug:

\[
A_{\text{req}} = V_{\text{app}} / (0.85 \times \phi \times f_{c}^\prime)
\]

\[\phi = 0.65\]

b. Determine the shear lug dimensions, assuming that bearing occurs only on the portion of the lug below the grout level. Assume a value of \( W \), the lug width, on the basis of the known base plate size to find \( H \), the total height of the lug, including the grout thickness, \( G \):

\[
H = (A_{\text{req}} / W) + G
\]

c. Calculate the factored cantilever end moment acting on a unit length of the shear lug:

\[
M_u = (V_{\text{app}}/W) \times (G + (H-G)/2)
\]

d. With the value for the moment, the lug thickness can be found. The shear lug should not be thicker than the base plate:

\[
t = [(4 \times M_u)/(0.9 \times f_{ya})]^{0.5}
\]

e. Design weld between plate section and base plate.

f. Calculate the breakout strength of the shear lug in shear. The method shown as follows is from ACI 349-01, Appendix B, section B.11:

\[
V_{cb} = A_{Vc} \times 4 \times \phi \times [f_{c}^\prime]^{0.5}
\]

where

\[A_{Vc} = \text{the projected area of the failure half-truncated pyramid defined by projecting a 45-degree plane from the bearing edges of the shear lug to the free edge. The bearing area of the shear lug shall be excluded from the projected area.}\]

\[\phi = \text{concrete strength reduction factor} = 0.85\]
Example 3 - Shear Lug Plate Section Design

**PLAN**

**SECTION**

- **V** = 40 (ULTIMATE)
- **W** = 1'0"
EXAMPLE 3 - Shear Lug Plate Section Design

Design a shear lug plate for a 14-in. square base plate, subject to a factored axial dead load of 22.5 kips, factored live load of 65 kips, and a factored shear load of 40 kips. The base plate and shear lug have \( f_{ya} = 36 \) ksi and \( f_{c'} = 3 \) ksi. The contact plane between the grout and base plate is assumed to be 1 in. above the concrete. A 2-ft 0-in. square pedestal is assumed. Ductility is not required.

\[
V_{app} = V_{ua} - V_l = 40 - (0.55)(22.5) = 27.6 \text{ kips}
\]

Bearing area = \( A_{req} = \frac{V_{app}}{(0.85 \phi f_{c'})} = \frac{27.6 \text{ kips}}{(0.85*0.65*3 \text{ ksi})} = 16.67 \text{ in.}^2 \)

On the basis of base plate size, assume the plate width, \( W \), will be 12 in.

Height of plate = \( H = \frac{A_{req}}{W} + G = \frac{16.67 \text{ in.}^2}{12 \text{ in.}} + 1 \text{ in.} = 2.39 \text{ in.} \)

Use 3 in.

Ultimate moment = \( M_u = (V_{app} / W) * (G + (H - G)/2) \)

\[
= (27.6 \text{ kips} / 12 \text{ in.}) * (1 \text{ in.} + (3 \text{ in.}-1 \text{ in.})/2) = 4.61 \text{ k-in.} / \text{in.}
\]

Thickness = \( t = \left(\frac{4 * M_u}{(\phi f_{ya})}\right)^{\frac{1}{2}} = \left(\frac{4*4.61 \text{ kip-in.}}{(0.9*36 \text{ ksi})}\right)^{\frac{1}{2}} = 0.754 \text{ in.} \)

Use 0.75 in.

This 12-in. x 3-in. x 0.75-in. plate will be sufficient to carry the applied shear load and resulting moment. Design of the weld between the plate section and the base plate is left to the engineer.

Check concrete breakout strength of the shear lug in shear.

Distance from shear lug to edge of concrete = \( (24 - 0.75) / 2 = 11.63 \text{ in.} \)

\[
A_v = 24 * (2+11.63) - (12 * 2) = 303 \text{ in.}^2
\]

\[
V_{cb} = A_{vc} * 4 * \phi * [f_{c'}^{0.5}] = 303 * 4 * 0.85 * [3000]^{0.5} = 56400 \text{ lb} = 56.4 \text{ kips} > 27.3 \text{ kips} \quad \text{OK}
\]
### GEOMETRY

<table>
<thead>
<tr>
<th>Column Section</th>
<th>Width</th>
<th>Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>W8X31</td>
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<td>8.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Column</th>
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<th>14.0</th>
<th>OK</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate</td>
<td>12.0</td>
<td>12.0</td>
<td>OK</td>
</tr>
<tr>
<td>Concrete Wp1</td>
<td>12.0</td>
<td>12.0</td>
<td>OK</td>
</tr>
<tr>
<td>Concrete Lp1</td>
<td>12.0</td>
<td>12.0</td>
<td>OK</td>
</tr>
<tr>
<td>Rod Offset</td>
<td>4.0</td>
<td>5.5</td>
<td>OK</td>
</tr>
<tr>
<td>Thickness of Grout</td>
<td>1.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### FACTORED LOADS (LRFD)

<table>
<thead>
<tr>
<th>Vertical Load P</th>
<th>22.5 kip</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending Moment M</td>
<td>0.0 k-ft</td>
</tr>
<tr>
<td>Horizontal Load V</td>
<td>40.0 kip</td>
</tr>
<tr>
<td>Design Eccentricity e</td>
<td>0.0 in</td>
</tr>
<tr>
<td>Design Eccentricity Is &lt; L/6</td>
<td></td>
</tr>
</tbody>
</table>

### MATERIALS

<table>
<thead>
<tr>
<th>Plate Steel Strength Fy</th>
<th>36.0 ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pier Concrete Strength f'c</td>
<td>3.0 ksi</td>
</tr>
</tbody>
</table>

### AXIALLY LOADED PLATES

#### Cantilever Model

<table>
<thead>
<tr>
<th>Bearing Stress fp</th>
<th>0.11 ksi</th>
<th>OK</th>
</tr>
</thead>
<tbody>
<tr>
<td>Critical Section @ Long m</td>
<td>3.20</td>
<td>in</td>
</tr>
<tr>
<td>Critical Section @ Short n</td>
<td>3.80</td>
<td>in</td>
</tr>
<tr>
<td>Plate Thickness tp</td>
<td>0.32</td>
<td>in</td>
</tr>
</tbody>
</table>

#### Thornton Model

<table>
<thead>
<tr>
<th>Bearing Strength ϕFp</th>
<th>2.84 ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Critical Section @ Int λn'</td>
<td>0.41</td>
</tr>
<tr>
<td>Design Moment @ Plate</td>
<td>0.01 k-in/in</td>
</tr>
<tr>
<td>Plate Thickness tp</td>
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</tbody>
</table>

### BASE PLATES WITH MOMENT

#### Blodgett Method

<table>
<thead>
<tr>
<th>Max. Bearing Stress fp</th>
<th>0.11 ksi</th>
<th>OK</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing @ Critical Section</td>
<td>0.11</td>
<td>ksi</td>
</tr>
<tr>
<td>Moment @ Critical Section</td>
<td>0.59</td>
<td>k-in/in</td>
</tr>
<tr>
<td>Moment due to Rod Tension</td>
<td>0.00</td>
<td>k-in/in</td>
</tr>
<tr>
<td>Design Moment @ Plate</td>
<td>0.59</td>
<td>k-in/in</td>
</tr>
<tr>
<td>Plate Thickness tp</td>
<td>0.27</td>
<td>in</td>
</tr>
</tbody>
</table>

#### DeWolf Method

<table>
<thead>
<tr>
<th>Max. Bearing Stress fp</th>
<th>0.11 ksi</th>
<th>OK</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing @ Critical Section</td>
<td>0.11</td>
<td>ksi</td>
</tr>
<tr>
<td>Moment @ Critical Section</td>
<td>0.59</td>
<td>k-in/in</td>
</tr>
<tr>
<td>Moment due to Rod Tension</td>
<td>0.00</td>
<td>k-in/in</td>
</tr>
<tr>
<td>Design Moment @ Plate</td>
<td>0.59</td>
<td>k-in/in</td>
</tr>
<tr>
<td>Plate Thickness tp</td>
<td>0.27</td>
<td>in</td>
</tr>
</tbody>
</table>
Project: Verification Example  
Engineer: Javier Encinas, PE  
Descrip: Shear Lug Verification  

**ASDIP Steel 3.2.5**  
STEEL BASE PLATE DESIGN  
www.asdipsoft.com

### ANCHORAGE DESIGN

<table>
<thead>
<tr>
<th>Rod Material Specification</th>
<th>F1554-36</th>
</tr>
</thead>
<tbody>
<tr>
<td>(4) Rods, $f_{ya} = 36.0$ ksi, $f_{uta} = 58.0$ ksi</td>
<td></td>
</tr>
<tr>
<td>Anchor Rod Size</td>
<td>1&quot; diam. x 12.0 in emb.</td>
</tr>
</tbody>
</table>

*Concrete Is Uncracked at Service Load Level*

---

### Tension Analysis (kip)

<table>
<thead>
<tr>
<th>Failure Mode</th>
<th>$\phi$</th>
<th>$N_n$</th>
<th>$N_u / \phi N_n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Strength</td>
<td>0.75</td>
<td>35.1</td>
<td>0.00</td>
</tr>
<tr>
<td>Rebars Strength</td>
<td>0.75</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
<tr>
<td>Conc. Breakout</td>
<td>0.70</td>
<td>25.0</td>
<td>0.00</td>
</tr>
<tr>
<td>Pullout Strength</td>
<td>0.70</td>
<td>50.4</td>
<td>0.00</td>
</tr>
<tr>
<td>Side Blowout</td>
<td>0.70</td>
<td>N.A.</td>
<td>N.A.</td>
</tr>
</tbody>
</table>

*No Reinforcing Bars Provided*

| Nu / $\phi N_n$ Tension Design Ratio | 0.00 | OK |

---

### Shear Analysis (kip)

**Shear Taken by Shear Lug + Friction**

<table>
<thead>
<tr>
<th>Total Shear Force</th>
<th>40.0 kip</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression Force</td>
<td>22.5 kip</td>
</tr>
<tr>
<td>Friction Coefficient</td>
<td>0.20</td>
</tr>
<tr>
<td>Friction Strength</td>
<td>3.4 kip</td>
</tr>
</tbody>
</table>

*Shear Force in Lug 36.6 kip*  
*Shear Lug Width 12.0 in*  
*Shear Lug Height 3.0 in*  
*Shear Lug Thickness 1.0 in*

<table>
<thead>
<tr>
<th>Failure Mode</th>
<th>$\phi$</th>
<th>$V_n$</th>
<th>$V_u / \phi V_n$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conc. Bearing</td>
<td>0.65</td>
<td>93.6</td>
<td>0.60</td>
</tr>
<tr>
<td>Conc. Breakout</td>
<td>0.75</td>
<td>57.8</td>
<td>0.84</td>
</tr>
</tbody>
</table>

*V / $\phi V_n$ Shear Design Ratio 0.84 OK*

---

### SUMMARY OF RESULTS

<table>
<thead>
<tr>
<th>Design Moment @ Plate</th>
<th>0.8 k-in/in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate Thickness $t_p$</td>
<td>0.32 in</td>
</tr>
<tr>
<td>Max. Bearing Stress $f_p$</td>
<td>0.11 ksi</td>
</tr>
<tr>
<td>Bearing Strength $f_{Fp}$</td>
<td>2.84 ksi</td>
</tr>
<tr>
<td>$f_p / \phi F_p$ Design Ratio</td>
<td>0.04 OK</td>
</tr>
</tbody>
</table>

**DESIGN IS DUCTILE**

---

**DESIGN CODES**

- Steel design .......... AISC 360-10 (14th Ed.)
- Base plate design .... AISC Design Series # 1
- Anchorage design ... ACI 318-11 Appendix D
Area A = 0 (No anchors in tension)

**PLAN**

Area A = 264.3 in²

**PLAN**

**SECTION**

**Tension Breakout**

**SECTION**

**Shear Breakout**
### STEEL BASE PLATE DESIGN

<table>
<thead>
<tr>
<th>GEOMETRY</th>
<th>FACTORED LOADS (LRFD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column Section .................</td>
<td>W8X31</td>
</tr>
<tr>
<td>Width</td>
<td>Length</td>
</tr>
<tr>
<td>Column ..........</td>
<td>8.0</td>
</tr>
<tr>
<td>Plate ..........</td>
<td>14.0</td>
</tr>
<tr>
<td>Concrete Wp1</td>
<td>12.0</td>
</tr>
<tr>
<td>Support Wp2</td>
<td>12.0</td>
</tr>
<tr>
<td>Rod Offset ......</td>
<td>4.0</td>
</tr>
<tr>
<td>Thickness of Grout ..........</td>
<td>1.0 in</td>
</tr>
<tr>
<td>Vertical Load P ..</td>
<td>22.5 kip</td>
</tr>
<tr>
<td>Bending Moment M ......</td>
<td>0.0 k-ft</td>
</tr>
<tr>
<td>Horizontal Load V ........</td>
<td>40.0 kip</td>
</tr>
<tr>
<td>Design Eccentricity e ....</td>
<td>0.0 in</td>
</tr>
</tbody>
</table>

**Materials**

| Plate Steel Strength Fy | 36.0 ksi |
| Pier Concrete Strength f'c | 3.0 ksi |

### AXIALLY LOADED PLATES

- **Bearing stress** $fp = P / (W \cdot L) = 22.5 / (14.0 \cdot 14.0) = 0.1$ ksi
- **Bearing strength** $Fp \geq 0.85 \cdot fL \cdot \sqrt{\frac{A^2}{AT}} = 0.85 \cdot 3.0 \cdot \sqrt{\frac{5.76}{190}} = 4.4$ ksi
- Under-strength factor $\phi = 0.65$
- **Bearing strength ratio** $\frac{fp}{\phi Fp} = \frac{0.1}{0.65 \cdot 2.8} = 0.04 < 1.0$ OK
- Critical section $m = 0.5 \cdot (L - 0.95 \cdot d) = 0.5 \cdot (14.0 - 0.95 \cdot 8.0) = 3.2$ in
- Critical section $n = 0.5 \cdot (W - 0.80 \cdot bf) = 0.5 \cdot (14.0 - 0.80 \cdot 8.0) = 3.8$ in
- $X = \frac{4 \cdot d \cdot bf}{(d + bf)^2} \cdot \text{Bearing ratio} = \frac{4 \cdot 8.0 \cdot 8.0}{(8.0 + 8.0)^2} \cdot 0.04 = 0.04$
- $\lambda = \frac{2 \cdot \sqrt{X}}{\sqrt{1 + \frac{n^2}{X}}} = \frac{2 \cdot \sqrt{0.04}}{\sqrt{1 + \frac{0.04}{0.04}}} = 0.20$
- $n' = 0.25 \cdot \sqrt{d \cdot bf} = 0.25 \cdot \sqrt{8.0 \cdot 8.0} = 2.0$ in
- Controlling section $k = \text{Max}(m, n, \lambda n') = \text{Max}(3.2, 3.8, 0.20 \cdot 2.0) = 3.8$ in
- **Plate moment** $M = fp \cdot k^2 / 2 = 0.1 \cdot 3.8^2 / 2 = 0.8$ k-in/in
- **Plate thickness** $t = \frac{k^2}{(\phi \cdot f) \cdot (2 - \lambda)} = \frac{3.8^2}{\sqrt{0.9 \cdot 36}} = 0.32$ in
BASE PLATES WITH MOMENT

- Blodgett Method

Eccentricity \( e = \frac{M}{P} = 0.0 \times 12 / 22.5 = 0.0 \text{ in} \)  \( < \frac{L}{6} = 14.0 / 6 = 2.3 \text{ in} \)

Max bearing stress \( f_p = \frac{P}{W^*L} + \frac{6 \times M}{W^*L^2} = \frac{22.5}{14.0 \times 14.0} + \frac{6 \times 0.0 \times 12}{14.0 \times 14.0^2} = 0.1 \text{ ksi} \)

Min bearing stress = \( \frac{P}{W^*L} - \frac{6 \times M}{W^*L^2} = \frac{22.5}{14.0 \times 14.0} - \frac{6 \times 0.0 \times 12}{14.0 \times 14.0^2} = 0.1 \text{ ksi} \)

Bearing at critical section \( f_p = f_p - m / L \times (f_p - \text{fpmin}) = 0.1 - 3.2 / 14.0 \times (0.1 - 0.1) = 0.1 \text{ ksi} \)

Moment due to bearing \( M_b = f_p(1/2) \times 1/2 \times (f_p - \text{fpmin}) \times m^2 / 3 \)
\[ M_b = 0.1 \times 3.2^2 / 3 \times (0.1 - 0.1) \times 3.2^2 / 3 = 0.6 \text{ k-in/in} \]

Plate thickness \( t = \frac{4 \cdot M_c}{(P_h \cdot f_y)} = \frac{4 \cdot 0.6}{0.9 \cdot 36} = 0.27 \text{ in} \)

\[ \text{AISC-DG#1 3.1.2} \]

![Diagram of base plate design](https://www.asdipsoft.com)
ANCHORAGE DESIGN

Rod Material Specification .... F1554-36, Use (4) Rods, fy = 36.0 ksi, futa = 58.0 ksi
Anchor Rod Size .... 1" diam. x 12.0 in emb., Ase = 0.61 in², Abrg = 1.50 in²

- Tension Analysis

Total tension force Nu = 0.0 kip, # of tension rods = 0, Tension force per rod Nui = 0.0 kip

Steel strength Nsa = Ase * futa = 0.606 * 58.0 = 35.1 kip

Under-strength factor \( \phi = 0.75 \)

Steel strength ratio \( \frac{Nui}{\phi Nsa} = \frac{0.0}{0.75 * 35.1} = 0.00 \) < 1.0 OK

- Concrete breakout strength of anchors in tension

No Reinforcing Bars Provided

Effective embedment \( hef = Ca, max / 1.5 = 17.50 / 1.5 = 11.67 \) in

Anchor group area \( Anc = (Ca + Cb) * (Ca + Sb + Cb) \)

\( Anc = (17.5 + 6.5) * (8.0 + 8.0 + 6.5) = 576.0 \) in²

Single anchor area \( Anco = 9 * hef^2 = 9 * (11.7)^2 = 1225.0 \) in²

Single anchor strength \( Nb = 24 \sqrt{f'c} * hef^{1.5} = 24 \sqrt{3000} * 11.7^{1.5} = 52.4 \) kip

Eccentricity factor \( \psi_{ec} = 1.00 \) (No eccentric load)

Edge effects factor \( \psi_{ed} = 0.7 + 0.3 \frac{Ca, min}{1.5 hef} = 0.7 + 0.3 \frac{6.5}{1.5 * 11.7} = 0.81 \)

Cracking factor \( \psi_{cn} = 1.25 \) (Uncracked concrete at service load level)

Breakout strength \( Ncbg = \frac{Anc}{Anco} \psi_{ec} \psi_{ed} \psi_{cn} Nb \)

\( Ncbg = \frac{576.0}{1225.0} * 1.00 * 0.81 * 1.25 * 25.4 = 25.0 \) kip

Under-strength factor \( \phi = 0.70 \)

Breakout strength ratio \( \frac{Nu}{\phi Ncbg} = \frac{0.0}{0.70 * 25.0} = 0.00 \) < 1.0 OK

Breakout strength ratio controls (0.00 < 0.00)

- Concrete pullout strength of anchors in tension

Single anchor strength \( Np = 8 * Abrg * f_c = 8 * 1.50 * 3.0 = 36.0 \) kip

Cracking factor \( \psi_{cp} = 1.25 \) (Uncracked concrete at service load level)

Pullout strength \( Npn = \psi_{cp} Np = 1.25 * 36.0 = 50.4 \) kip

Under-strength factor \( \phi = 0.70 \)

Pullout strength ratio \( \frac{Nu}{\phi Npn} = \frac{0.0}{0.70 * 50.4} = 0.00 \) < 1.0 OK

ACI D.5.2.5

ASDIP Steel 3.2.5
STEEL BASE PLATE DESIGN
www.asdipsoft.com
- Concrete side-face blowout strength of anchors in tension
  ACI D.5.4
  Side-face blowout $N_{sbg} = N.A. \text{ (Embed } < 2.5 \text{ Ca}, 12.0 < 2.5 \times 6.5 = 16.3)$
  Tension Design Ratio $\frac{N_U}{\phi N_n} = 0.00 < 1.0 \text{ OK}$
  ACI D.5.4.1

- Shear Analysis
  ACI D.5

Shear resisted by Shear Lug + Friction

Total shear force $V_u = 40.0 \text{ kip}$,  Compression force $C = 22.5 \text{ kip}$,  Friction coeff. = 0.20
Friction strength $\frac{\phi F_r}{\phi} = 0.75 \times 22.5 \times 0.20 = 3.4 \text{ kip}$
Friction strength ratio $\frac{V_u}{\phi F_r} = \frac{40.0}{0.70 \times 3.4} = 1.00 \leq 1.0 \text{ OK}$

Shear lug width $W_l = 12.0 \text{ in}$,  Shear lug height $H_l = 3.0 \text{ in}$,  Shear lug thickness $t_l = 1.0 \text{ in}$

- Steel strength of lug in flexure

Lug moment $M_{lug} = V_{lug} \times (\text{grout} + (H_l - \text{grout}) / 2) = 36.6 \times (1.0 + (3.0 - 1.0) / 2) = 73.3 \text{ k-in}$
Lug flexural strength $M_n = W_l \times f_y \times t_l / 4 = 12.0 \times 36 \times 1.0 / 4 = 108.0 \text{ k-in}$  AISC F.11
Under-strength factor $\phi = 0.90$  AISC F.1

Flexural strength ratio $\frac{M_{lug}}{M_n} = \frac{73.3}{0.90 \times 108.0} = 0.75 < 1.0 \text{ OK}$  AISC B3.4

- Steel strength of lug in shear

Shear force in lug $V_{lug} = V_u - \phi F_r = 40.0 - 3.4 = 36.6 \text{ kip}$
Lug shear strength $V_{sln} = 0.6 * f_y * W_l = 0.6 * 36 * 12.0 = 252.9 \text{ kip}$  AISC Eq. (G2.1)
Under-strength factor $\phi = 0.90$  AISC G.1

Shear strength ratio $\frac{V_{lug}}{V_{sln}} = \frac{36.6}{0.90 \times 259.2} = 0.16 < 1.0 \text{ OK}$  AISC B3.4

- Weld strength in shear lug

Shear lug fillet weld size $a = 0.250 \text{ in}$  (Min size = 0.313 in)  NG
Shear per unit width $f_v = V_{lug} / (2 \times W_l) = 36.6 / (2 \times 12.0) = 1.5 \text{ kip}$
Tension per unit width $f_t = M_{lug} / ([f_f + 2a / 3] \times W_l) = 73.3 / [(1.0 + 2 * 0.250 / 3) \times 12.0] = 5.2 \text{ kip}$
Resultant per lug width $R = \sqrt{f_v^2 + f_t^2} = \sqrt{1.5^2 + 5.2^2} = 5.65 \text{ kip}$
Weld stress $F_w = 0.6 * F_{exx} (1 + 0.5 \sin(90)) = 0.6 \times 70 \times (1 + 0.5) = 63.0 \text{ ksi}$  AISC Eq. (J2.5)
Weld strength $V_{wn} = F_w \times 0.707 a \times 2 W_l = 63.0 \times 0.707 \times 2 \times 12.0 = 267.3 \text{ kip}$
Under-strength factor $\phi = 0.75$

Weld strength ratio $\frac{R}{\phi V_{wn}} = \frac{130.8}{0.75 \times 267.3} = 0.65 < 1.0 \text{ OK}$  AISC B3.3

- Concrete bearing strength of lug in shear

Lug bearing area $A_p = (H_l - \text{grout}) \times W_l = (3.0 - 1.0) \times 12.0 = 24.0 \text{ in}^2$
Lug bearing strength $V_{pn} = 1.3 * f_c * A_p = 1.3 \times 3.0 \times 24.0 = 93.6 \text{ kip}$  ACI 349 D.4.6
Under-strength factor $\phi = 0.65$

Bearing strength ratio $\frac{V_{lug}}{\phi V_{pn}} = \frac{36.6}{0.65 \times 93.6} = 0.60 < 1.0 \text{ OK}$  ACI 4.1.1
- Concrete breakout strength of lug in shear

Lug breakout area: \( A_{vc} = (Ca_i + Hi \cdot grout) \cdot (Ca_x + Wi + Cba) - A_p \)

\[
A_{vc} = (10.0 + 3.0 - 1.0) \cdot (6.0 + 12.0 + 6.0) - 24.0 = 264.0 \text{ in}^2
\]

Breakout strength: \( V_{cb} = A_{vc} \cdot \sqrt{\frac{f_c}{f_y}} = 264.0 \cdot 4 = 57.8 \text{ kip} \)

Under-strength factor: \( \phi = 0.75 \)  

\[
\text{Breakout strength ratio} = \frac{V_{lug}}{\phi V_{cb}} = \frac{36.6}{0.75 \cdot 57.8} = 0.84 < 1.0 \text{ OK}
\]

\[
\text{Shear Design Ratio} = \frac{V_u}{\phi V_n} = 0.84 < 1.0 \text{ OK}
\]

Anchorage design is ductile

**DESIGN CODES**

- Steel design ................. AISC 360-10 (14th Ed.)
- Base plate design .......... AISC Design Series # 1
- Anchorage design .......... ACI 318-11 Appendix D