



Shear Lug Verification Example

PIP STE05121  
Anchor Bolt Design Guide  
PIP - Oct 2006

## **9. Shear Lug Design**

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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,  $V_{app}$ , used to design the shear lug should be computed as follows:

$$V_{app} = V_{ua} - V_f$$

### 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:

$$A_{req} = V_{app} / (0.85 * \phi * f_c') \quad \phi = 0.65$$

- 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,  $G$ :

$$H = (A_{req} / W) + G$$

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

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

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

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

- 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:

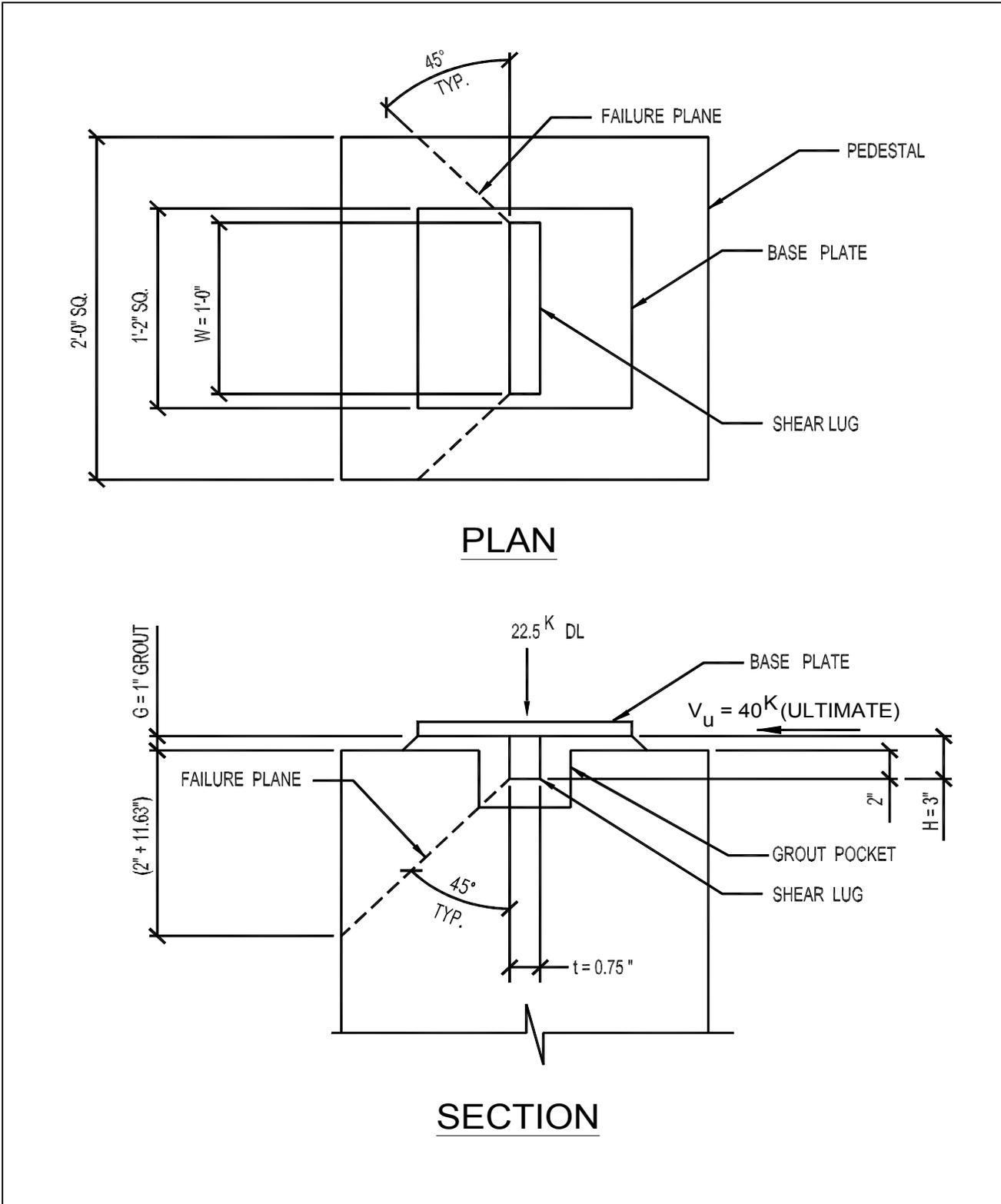
$$V_{cb} = A_{vc} * 4 * \phi * [f_c']^{0.5}$$

where

$A_{vc}$  = 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.

$\phi$  = concrete strength reduction factor = **0.85**

### Example 3 - Shear Lug Plate Section Design



### **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.

$$V_{app} = V_{ua} - V_f = 40 - (0.55)(22.5) = 27.6 \text{ kips}$$

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

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

$$\text{Height of plate} = H = A_{req} / W + G = 16.67 \text{ in.}^2 / 12 \text{ in.} + 1 \text{ in.} = 2.39 \text{ in.} \quad \text{Use 3 in.}$$

$$\begin{aligned} \text{Ultimate moment} = M_u &= (V_{app} / W) * (G + (H - G)/2) \\ &= (27.6 \text{ kips} / 12 \text{ in.}) * (1 \text{ in.} + (3 \text{ in.} - 1 \text{ in.})/2) = 4.61 \text{ k-in.} / \text{in.} \end{aligned}$$

$$\text{Thickness} = t = [(4 * M_u) / (\phi * f_{ya})]^{1/2} = [(4 * 4.61 \text{ kip-in.}) / (0.9 * 36 \text{ ksi})]^{1/2} = 0.754 \text{ in.} \quad \text{Use 0.75 in.}$$

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.

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

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

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

**GEOMETRY**

Column Section .....	<b>W8X31</b>		
	<b>Width</b>	<b>Length</b>	
Column .....	<b>8.0</b>	<b>8.0</b>	in
Plate .....	<b>14.0</b>	<b>14.0</b>	in <b>OK</b>
Concrete Wp1	<b>12.0</b>	Lp1 <b>12.0</b>	in <b>OK</b>
Support Wp2	<b>12.0</b>	Lp2 <b>12.0</b>	in <b>OK</b>
Rod Offset .....	<b>4.0</b>	<b>5.5</b>	in <b>OK</b>
Thickness of Grout .....		<b>1.0</b>	in

**FACTORED LOADS (LRFD)**

Vertical Load P .....	<b>22.5</b>	kip
Bending Moment M .....	<b>0.0</b>	k-ft
Horizontal Load V .....	<b>40.0</b>	kip
Design Eccentricity e .....	0.0	in
Design Eccentricity $Is < L/6$		

**MATERIALS**

Plate Steel Strength $F_y$ ....	<b>36.0</b>	ksi
Pier Concrete Strength $f'_c$ .....	<b>3.0</b>	ksi

**AXIALLY LOADED PLATES**

Cantilever Model				Thornton Model			
Bearing Stress $f_p$ .....	0.11	ksi	<b>OK</b>	Bearing Strength $\phi F_p$ .....	2.84	ksi	
Critical Section @ Long m	3.20	in		Critical Section @ Int $\lambda n'$ .	0.41	in	
Critical Section @ Short n	3.80	in		Design Moment @ Plate ...	0.01	k-in/in	
Plate Thickness $t_p$ .....	0.32	in		Plate Thickness $t_p$ .....	0.03	in	

**BASE PLATES WITH MOMENT**

Blodgett Method				DeWolf Method			
Max. Bearing Stress $f_p$ .....	0.11	ksi	<b>OK</b>	Max. Bearing Stress $f_p$ .....	0.11	ksi	<b>OK</b>
Bearing @ Critical Section	0.11	ksi		Bearing @ Critical Section	0.11	ksi	
Moment @ Critical Section	0.59	k-in/in		Moment @ Critical Section	0.59	k-in/in	
Moment due to Rod Tension	0.00	k-in/in		Moment due to Rod Tension	0.00	k-in/in	
Design Moment @ Plate ....	0.59	k-in/in		Design Moment @ Plate ....	0.59	k-in/in	
Plate Thickness $t_p$ .....	0.27	in		Plate Thickness $t_p$ .....	0.27	in	

**ANCHORAGE DESIGN**

Rod Material Specification ..... **F1554-36**  
*(4) Rods , fya = 36.0 ksi, futu = 58.0 ksi*  
 Anchor Rod Size .. **1" diam. x 12.0 in emb.**  
*Concrete Is Uncracked at Service Load Level*

**Tension Analysis (kip)**

Total Tension Force Nug ..... 0.0 kip  
 Tension Force per Rod Nui ... 0.0 kip

*No Reinforcing Bars Provided*

Failure Mode	$\phi$	Nn	Nu / $\phi$ Nn
Steel Strength Nsa	0.75	35.1	0.00
Rebars Strength Nrg	0.75	N.A.	N.A.
Conc. Breakout Ncbg	0.70	25.0	0.00
Pullout Strength Npn	0.70	50.4	0.00 ✓
Side Blowout Nsbg	0.70	N.A.	N.A.
Nu / $\phi$ Nn Tension Design Ratio ....		0.00	OK

**Shear Analysis (kip)**

**Shear Taken by Shear Lug + Friction**

Total Shear Force V ..... 40.0 kip  
 Compression Force C ..... 22.5 kip  
 Friction Coefficient ..... 0.20  
 Friction Strength  $\phi$ Fr ..... 3.4 kip  
 V /  $\phi$ Fr Shear Friction Ratio ..... 1.00 OK  
 Shear Force in Lug ..... 36.6 kip  
 Shear Lug Width WI ..... 12.0 in  
 Shear Lug Height HI ..... 3.0 in  
 Shear Lug Thickness tl ..... 1.0 in

Failure Mode	$\phi$	Vn	Vu / $\phi$ Vn
Conc. Bearing Vcbr	0.65	93.6	0.60
Conc. Breakout Vcb	0.75	57.8	0.84 ✓
V / $\phi$ Vn Shear Design Ratio .....		0.84	OK

**DESIGN CODES**

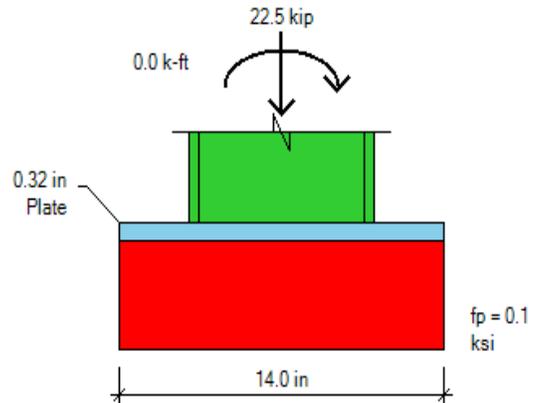
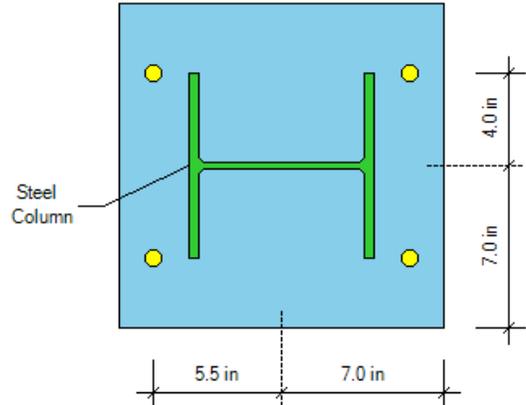
Steel design ..... AISC 360-10 (14th Ed.)  
 Base plate design .... AISC Design Series # 1  
 Anchorage design ... ACI 318-11 Appendix D

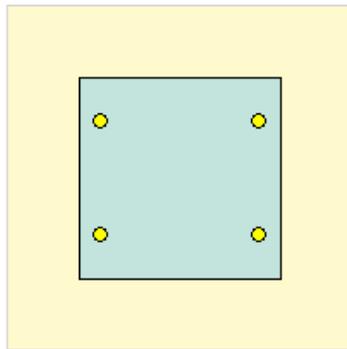
**SUMMARY OF RESULTS**

Design Moment @ Plate ... 0.8 k-in/in  
 Plate Thickness tp ..... 0.32 in  
 Max. Bearing Stress fp ..... 0.11 ksi  
 Bearing Strength  $\phi$ Fp ..... 2.84 ksi  
 fp /  $\phi$ Fp Design Ratio ..... 0.04 OK

**DESIGN IS DUCTILE**

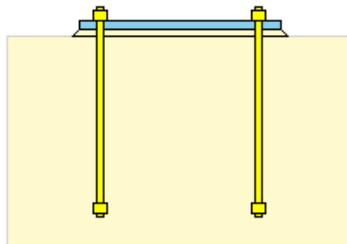
(4) 1" diam. x 12.0 in emb.  
 F1554-36 Rods





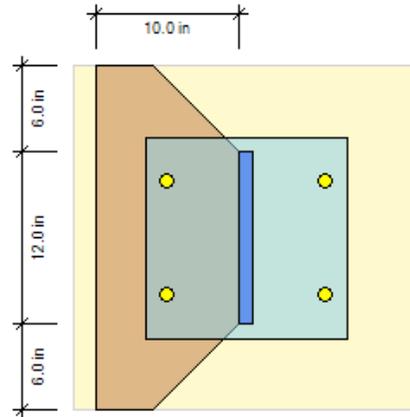
Area Anc = 0 (No anchors in tension)

**PLAN**



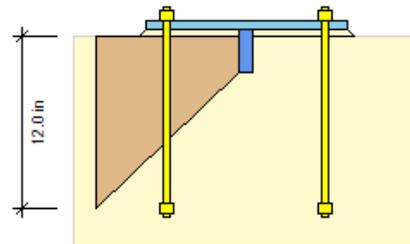
**SECTION**

Tension Breakout



Area Avc = 264.0 in<sup>2</sup>

**PLAN**



**SECTION**

Shear Breakout

GEOMETRY					
Column Section	W8X31				
	Width	Length			
Column	8.0	8.0	in		
Plate	14.0	14.0	in	OK	
Concrete Support	Wp1	Lp1	12.0	in	OK
	Wp2	Lp2	12.0	in	OK
Rod Offset	4.0	5.5	in	OK	
Thickness of Grout			1.0	in	

FACTORED LOADS (LRFD)		
Vertical Load P	22.5	kip
Bending Moment M	0.0	k-ft
Horizontal Load V	40.0	kip
Design Eccentricity e	0.0	in
Design Eccentricity Is < L/6		

MATERIALS		
Plate Steel Strength Fy	36.0	ksi
Pier Concrete Strength fc	3.0	ksi

**AXIALLY LOADED PLATES**

Bearing stress  $f_p = P / (W * L) = 22.5 / (14.0 * 14.0) = 0.1$  ksi

Bearing strength  $F_p = 0.85 * f_c * \sqrt{\frac{A_2}{A_1}} = 0.85 * 3.0 * \sqrt{\frac{576}{196}} = 4.4$  ksi ACI 10.14.1

Under-strength factor  $\phi = 0.65$  ACI 9.3.2.4

Bearing strength ratio  $= \frac{f_p}{\phi F_p} = \frac{0.1}{0.65 * 4.4} = 0.04 < 1.0$  OK

Critical section  $m = 0.5 * (L - 0.95 * d) = 0.5 * (14.0 - 0.95 * 8.0) = 3.2$  in AISC-DG#1 3.1.2

Critical section  $n = 0.5 * (W - 0.80 * bf) = 0.5 * (14.0 - 0.80 * 8.0) = 3.8$  in

$X = \left[ \frac{4 * d * bf}{(d + bf)^2} \right] * \text{Bearing ratio} = \left[ \frac{4 * 8.0 * 8.0}{(8.0 + 8.0)^2} \right] * 0.04 = 0.04$  AISC-DG#1 3.1.2

$\lambda = \frac{2 * \sqrt{X}}{(1 + \sqrt{1 - X})} = \frac{2 * \sqrt{0.04}}{1 + \sqrt{1 - 0.04}} = 0.20$

$n' = 0.25 * \sqrt{d * bf} = 0.25 * \sqrt{8 * 8} = 2.0$  in

Controlling section  $k = \text{Max}(m, n, \lambda n') = \text{Max}(3.2, 3.8, 0.20 * 2.0) = 3.8$  in

Plate moment  $M = f_p * k^2 / 2 = 0.1 * 3.8^2 / 2 = 0.8$  k-in/in

Plate thickness  $t = k * \sqrt{\frac{2 * f_p}{(\phi * f_y)}} = 3.8 * \sqrt{\frac{2 * 0.1}{0.9 * 36}} = 0.32$  in AISC-DG#1 3.1.2

**BASE PLATES WITH MOMENT**

**- Blodgett Method**

Eccentricity  $e = M/P = 0.0 * 12 / 22.5 = 0.0 \text{ in} < L/6 = 14.0 / 6 = 2.3 \text{ in}$

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

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

Bearing at critical section  $fp1 = fp - m / L * (fp - fpmin) = 0.1 - 3.2 / 14.0 * (0.1 - 0.1) = 0.1 \text{ ksi}$

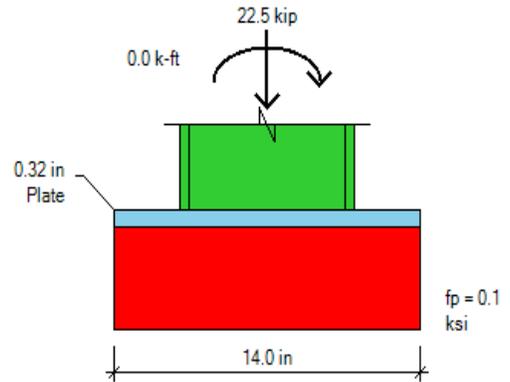
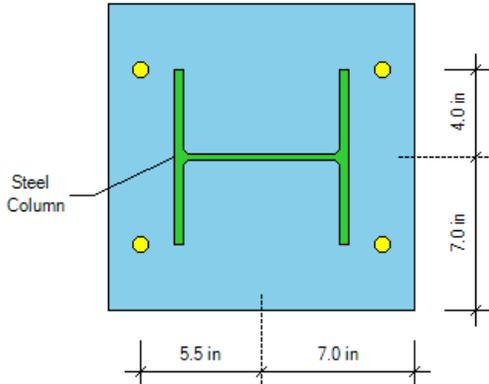
Moment due to bearing  $Mb = fp1 * m^2 / 2 * (fp - fp1) * m^2 / 3$

$$Mb = 0.1 * 3.2^2 / 2 * (0.1 - 0.1) * 3.2^2 / 3 = 0.6 \text{ k-in/in}$$

$$\text{Plate thickness } t = \sqrt{\frac{4 * Mb}{(\Phi * fy)}} = \sqrt{\frac{4 * 0.6}{0.9 * 36}} = 0.27 \text{ in}$$

AISC-DG#1 3.1.2

(4) 1" diam. x 12.0 in emb.  
 F1554-36 Rods



**ANCHORAGE DESIGN**

Rod Material Specification ..... F1554-36 , Use (4) Rods ,  $f_y = 36.0$  ksi,  $f_u = 58.0$  ksi

Anchor Rod Size .... 1" diam. x 12.0 in emb. ,  $A_{se} = 0.61$  in<sup>2</sup> ,  $A_{brg} = 1.50$  in<sup>2</sup>

**- Tension Analysis**

ACI D.5

Total tension force  $N_u = 0.0$  kip , # of tension rods = 0 , Tension force per rod  $N_{ui} = 0.0$  kip

**- Steel strength of anchors in tension**

ACI D.5.1

Steel strength  $N_{sa} = A_{se} * f_u = 0.606 * 58.0 = 35.1$  kip

ACI Eq. (D-2)

Under-strength factor  $\phi = 0.75$

ACI D.4.3

Steel strength ratio =  $\frac{N_{ui}}{\phi N_{sa}} = \frac{0.0}{0.75 * 35.1} = 0.00 < 1.0$  OK

ACI D.4.1.1

**- Concrete breakout strength of anchors in tension**

ACI D.5.2

**No Reinforcing Bars Provided**

Effective embedment  $h_{ef} = C_{a,max} / 1.5 = 17.50 / 1.5 = 11.67$  in

ACI D.5.2.3

Anchor group area  $A_{nc} = (C_{a1} + C_{b1}) * (C_{a2} + S_2 + C_{b2})$

$A_{nc} = (17.5 + 6.5) * (8.0 + 8.0 + 6.5) = 576.0$  in<sup>2</sup>

ACI D.5.2.1

Single anchor area  $A_{nc0} = 9 h_{ef}^2 = 9 * (11.7)^2 = 1225.0$  in<sup>2</sup>

Eq. (D-5)

Single anchor strength  $N_b = 24 \sqrt{f'_c} \cdot h_{ef}^{1.5} = 24 \sqrt{3000} \cdot 11.7^{1.5} = 52.4$  kip

Eq. (D-6)

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

ACI D.5.2.4

Edge effects factor  $\psi_{ed} = 0.7 + 0.3 \frac{C_{a,min}}{1.5 h_{ef}} = 0.7 + 0.3 \frac{6.5}{1.5 * 11.7} = 0.81$

ACI D.5.2.5

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

ACI D.5.2.6

Breakout strength  $N_{cbg} = \frac{A_{nc}}{A_{nc0}} \psi_{ec} \psi_{ed} \psi_{cn} N_b$

$N_{cbg} = \frac{576.0}{1225.0} 1.00 * 0.81 * 1.25 * 52.4 = 25.0$  kip

Eq. (D-4)

Under-strength factor  $\phi = 0.70$

ACI D.4.3

Breakout strength ratio =  $\frac{N_u}{\phi N_{cbg}} = \frac{0.0}{0.70 * 25.0} = 0.00 < 1.0$  OK

ACI D.4.1.1

Breakout strength ratio controls (0.00 < 0.00)

ACI D.5.2.9

**- Concrete pullout strength of anchors in tension**

ACI D.5.3

Single anchor strength  $N_p = 8 A_{brg} f_c = 8 * 1.50 * 3.0 = 36.0$  kip

ACI Eq. (D-14)

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

ACI D.5.3.6

Pullout strength  $N_{pn} = \psi_{cp} N_p = 1.25 * 36.0 = 50.4$  kip

ACI Eq. (D-13)

Under-strength factor  $\phi = 0.70$

ACI D.4.3

Pullout strength ratio =  $\frac{N_u}{\phi N_{pn}} = \frac{0.0}{0.70 * 50.4} = 0.00 < 1.0$  OK

ACI D.4.1.1

- Concrete side-face blowout strength of anchors in tension

ACI D.5.4

Side-face blowout  $N_{sbg} = \text{N.A.}$  (Embed  $< 2.5 C_{a1}$ ,  $12.0 < 2.5 * 6.5 = 16.3$ )

ACI D.5.4.1

$$\text{Tension Design Ratio} = \frac{N_u}{\phi N_n} = 0.00 < 1.0 \text{ OK}$$

ACI D.4.1.1

- 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  $\phi Fr = \phi * C * \text{coeff} = 0.75 * 22.5 * 0.20 = 3.4 \text{ kip}$

$$\text{Friction strength ratio} = \frac{V_u}{\phi Fr} = \frac{40.0}{0.70 * 3.4} = 1.00 \leq 1.0 \text{ OK}$$

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  $M_{lug} = V_{lug} * (\text{grout} + (H_l - \text{grout}) / 2) = 36.6 * (1.0 + (3.0 - 1.0) / 2) = 73.3 \text{ k-in}$

Lug flexural strength  $M_n = W_l * f_y * t_l^2 / 4 = 12.0 * 36 * 1.0^2 / 4 = 108.0 \text{ k-in}$

AISC F.11

Under-strength factor  $\phi = 0.90$

AISC F.1

$$\text{Flexural strength ratio} = \frac{M_{lug}}{\phi M_n} = \frac{73.3}{0.90 * 108.0} = 0.75 < 1.0 \text{ OK}$$

AISC B3.4

- Steel strength of lug in shear

Shear force in lug  $V_{lug} = V_u - \phi Fr = 40.0 - 3.4 = 36.6 \text{ kip}$

Lug shear strength  $V_{sln} = 0.6 * f_y * W_l * t_l = 0.6 * 36 * 12.0 * 1.0 = 259.2 \text{ kip}$

AISC Eq. (G2.1)

Under-strength factor  $\phi = 0.90$

AISC G.1

$$\text{Shear strength ratio} = \frac{V_{lug}}{\phi V_{sln}} = \frac{36.6}{0.90 * 259.2} = 0.16 < 1.0 \text{ OK}$$

AISC B3.4

- Weld strength in shear lug

Shear lug fillet weld size  $a = 0.250 \text{ in}$  (Min size = 0.313 in) NG

AISC J2.2a

Shear per unit width  $f_v = V_{lug} / (2 * W_l) = 36.6 / (2 * 12.0) = 1.5 \text{ kip}$

Tension per unit width  $f_t = M_{lug} / [(t_l + 2a / 3) * W_l] = 73.3 / [(1.0 + 2 * 0.250 / 3) * 12.0] = 5.2 \text{ kip}$

Resultant per lug width  $R = \sqrt{f_v^2 + f_t^2} * 2 * W_l = \sqrt{1.5^2 + 5.2^2} * 2 * 12 = 130.8 \text{ kip}$

Weld stress  $F_w = 0.6 * F_{exx} (1 + 0.5 \text{Sin}(90)) = 0.6 * 70 * (1 + 0.5) = 63.0 \text{ ksi}$

AISC Eq. (J2.5)

Weld strength  $V_{wn} = F_w * 0.707 a * 2 W_l = 63.0 * 0.707 * 0.250 * 2 * 12.0 = 267.3 \text{ kip}$

Under-strength factor  $\phi = 0.75$

AISC J3b.4

$$\text{Weld strength ratio} = \frac{R}{\phi V_{wn}} = \frac{130.8}{0.75 * 267.3} = 0.65 < 1.0 \text{ OK}$$

AISC B3.3

- Concrete bearing strength of lug in shear

Lug bearing area  $A_p = (H_l - \text{grout}) * W_l = (3.0 - 1.0) * 12.00 = 24.0 \text{ in}^2$

Lug bearing strength  $V_{pn} = 1.3 * f_c * A_p = 1.3 * 3.0 * 24.0 = 93.6 \text{ kip}$

ACI 349 D.4.6

Under-strength factor  $\phi = 0.65$

ACI 9.3.2.4

$$\text{Bearing strength ratio} = \frac{V_{lug}}{\phi V_{pn}} = \frac{36.6}{0.65 * 93.6} = 0.60 < 1.0 \text{ OK}$$

ACI 4.1.1

- Concrete breakout strength of lug in shear

ACI 349 D.11.2

Lug breakout area  $A_{vc} = (C_{a1} + Hl - grout) * (C_{a2} + Wl + C_{b2}) - A_p$

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

$$\text{Breakout strength } V_{cb} = A_{vc} * 4\sqrt{f'_c} = 264.0 * 4\sqrt{3000} = 57.8 \text{ kip}$$

Under-strength factor  $\phi = 0.75$

ACI D.4.3

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

ACI D.4.1.1

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

ACI D.4.1.1

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