

**STRUCTURAL ANALYSIS OF  
SIX FOOT HIGH  
PRECAST CONCRETE FENCE**

**PREPARED FOR  
O-WELL PRECAST LLC.**

**May 15, 2004**

**PREPARED BY  
David O. Keith  
AMERICAN FORK, UTAH**

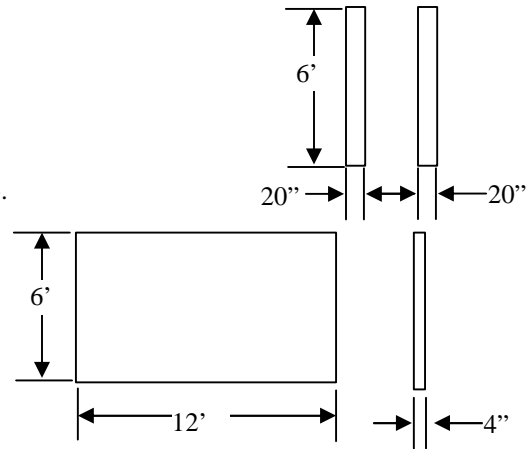
Six Foot High Precast Concrete Fence

**Posts**

Height: 6 ft.  
 Breadth: 20 in.  
 Width: 20 in.  
 Weight =  $6 \times 20/12 \times 20/12 \times 150 = 2,500$  lb.  
 Spacing: 13' - 0"

**Panels**

Height: 6 ft.  
 Breadth: 12 ft.  
 Thickness: 4 in.  
 Weight =  $6 \times 12 \times 4/12 \times 150 = 3,600$  lb.



**Lateral Forces (2003 IBC) and Anchor Loads**

Wind:

$V = 90$  mph, Exposure C  
 $Z = 6$  ft  
 Structure Category = I  
 $K_z = 2.01(15/z_g)^{2.7} = 0.85$   
 Where,  $\rho = 9.5$  and  $z_g = 900$  ft.  
 $K_{zt} = 1$ , flat terrain  
 $K_d = 0.85$   
 $I = 0.87$   
 $q_z = 0.00256K_zK_{zt}K_dV^2I = 13$  psf  
 $F = q_zGC A$   
 $C = 1.2$ ,  $G = 0.85$   
 $F = 13 \times 0.85 \times 1.2 \times 13 \times 6 = 1,034$  lb.

Overturning Moment =  $1034 \times 3 = 3,103$  ft-lb.  
 Resisting Moment =  $0.67(3600) 2/12 + 0.67(2500) 10/12 = 1799$  ft-lb.  
 Required hold down force,  $F_h$   
 $3103 - 1799 = F_h (10/12)$   
 $F_h = 1,566$  lb  
 Apply safety factors:  
 $V_u = 1,264 \times 1.6 = 2,022$  lb.  
 $N_u = 1,566 \times 1.6 = 2,506$  lb.

Seismic:

Short period spectral acceleration,  $S_s = 1.5$ ; 1-Second spectral acceleration,  $S_1 = 0.50$   
 Site Class D assumed.  $F_a = 1.0$ ,  $F_v = 1.5$ ,  
 $F_p = 0.4a_pS_{DS}W_p / (R_p/I_p) (1 + 2(z/h)) = 0.4W_p$   
 $a_p = 2.5$   
 $S_{DS} = 2/3F_a S_s = 1.0$   
 $R_p = 2.5$   
 $I_p = 1.0$   
 $z = 0$  ft.,  $h = 6$  ft.  
 $W_p = 1/8$  panel weight + post weight =  $450 + 2500 = 2950$  lb.  
 This assumes that the fence panel is supported by the soil between posts and remains within 1 ft. of vertical. Then the force applied at the top of the panel by the post would be:  $F = WX/(H)$ ,  $W$  is the panel weight,  $X$  is the horizontal distance, measured from vertical to the top edge of the fence panel, and  $H$  is the fence height.  
 $F_p = 180 + 1000 = 1180$  lb.

Overturning Moment =  $1180 \times 3 = 3540$  ft-lb.  
 Resisting Moment =  $0.85(3600) 2/12 + 0.85(2500) 10/12 = 2281$  ft-lb.  
 Required hold down force,  $F_h$   
 $3540 - 2281 = F_h (10/12)$

$$F_h = 1,511 \text{ lb.}$$

Apply safety factors:

$$V_u = 1,180 \times 1.3 = 1,534 \text{ lb.}$$

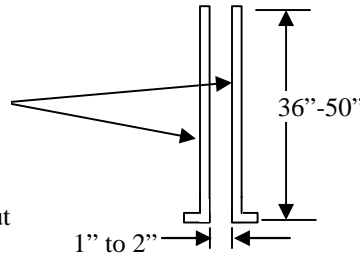
$$N_u = 1,511 \times 1.3 = 1,964 \text{ lb.}$$

### Post Hold Down

Description:

(2) #4 bars embedded into footing and post.

#4 L-Bars, ASTM A615 Grade 60, L-ends or standard hook embedded 12" min. into footing or deformed bars embedded a minimum of 25" into footing and center of post core. The core is 8" diameter at the top and 6" diameter at the bottom. The grout shall have an  $f'_c$  4,000 psi.



Anchors shall have edge reinforcement of a No. 4 bar or greater between anchor and edge and with the edge reinforcement enclosed within stirrups spaced at not more than 4 inches.

Anchor Strength in Post

1. Steel Strength of Anchor in Tension

$$N_s = nA_{se}f_y = 2 \times 0.196 \times 60\text{ksi} = 23.52\text{k per two bars}$$

2. Steel Strength of Anchor in Shear

$$V_s = nA_{se}f_y = 2 \times 0.196 \times 60\text{ksi} = 23.52\text{k per two bars}$$

3. Concrete Breakout Strength of Anchor in Tension

$$N_b = k \cdot f'_c \cdot h_{ef}^{1.5} = 24 \times 4000 \times 15.2^{1.5} = 90 \text{ k}$$

$$N_{cbg} = A_v/A_{v_o} \cdot \lambda \cdot \psi_1 \cdot \psi_2 \cdot \psi_3 \cdot N_b = 400/2079 \times 1 \times 0.82 \times 1 \times 90 = \underline{14.16 \text{ k}}$$

4. Concrete Breakout Strength of Anchor in Shear

$$V_b = 7 \cdot (l/d_o)^{0.2} \cdot d_o \cdot f'_c \cdot c_1^{1.5} = 7 \cdot (4/0.5)^{0.2} \times 0.5 \times 4000 \times 9^{1.5} = 14.716 \text{ k}$$

$$V_{cbg} = A_v/A_{v_o} \cdot \lambda \cdot \psi_5 \cdot \psi_6 \cdot \psi_7 \cdot V_b = 270/365 \times 1 \times 0.9 \times 1.4 \times 14.716 = \underline{13.735 \text{ k}}$$

5. Pullout Strength of Anchor in Tension

Rebar embedded minimum of 15.2 inches to develop full strength.

$$N_{pn} = 2 \times 0.196 \times 60\text{ksi} = 23.52\text{k per two bars}$$

6. Concrete Pryout Strength of Anchor in Shear

$$V_{cp} = k_{cp} \cdot N_{cb} = 2 \times 14.16 = 28.3 \text{ k}$$

7. Concrete Pryout Strength of Anchor in Shear

$$V_{cp} = k_{cp} \cdot N_{cb} = 2 \times 13.2 = 26.4 \text{ k}$$

### Allowable Loads

#### Wind

$$\phi N_n = N_u: \underline{0.75 \times 14,160 = 10,620 > 2,506 \text{ OK}}$$

$$\phi V_n = V_u: \underline{0.75 \times 13,735 = 10,301 > 2,022 \text{ OK}}$$

If,  $V_u > 0.2 \phi V_n$  and  $N_u > 0.2 \phi N_n$ , then:  $N_u / \phi N_n + V_u / \phi V_n = 1.2$

$$2506/10620 + 2022/10301 = \underline{0.236 + 0.196 = 0.432 \text{ OK}}$$

#### Seismic

$$0.75 \phi N_n = N_u: \underline{0.75 \times 0.75 \times 14,160 = 7,965 > 1,964 \text{ OK}}$$

$$0.75 \phi V_n = V_u: \underline{0.75 \times 0.75 \times 13,735 = 7,726 > 1,534 \text{ OK}}$$

$$\text{If, } V_u > 0.2 \phi V_n \text{ and } N_u > 0.2 \phi N_n, \text{ then: } N_u / \phi N_n + V_u / \phi V_n = 1.2 \\ 1964/7965 + 1534/7726 = \underline{0.247 + 0.199 = 0.446 \text{ OK}}$$

#### Anchor Strength in Footing

1. Steel Strength of Anchor in Tension  
 $N_s = n A_{s_e} f_y = 2 \times 0.196 \times 60 \text{ksi} = 23.52 \text{k}$  per two bars
2. Steel Strength of Anchor in Shear  
 $V_s = n A_{s_e} f_y = 2 \times 0.196 \times 60 \text{ksi} = 23.52 \text{k}$  per two bars
3. Concrete Breakout Strength of Anchor in Tension  
 $N_b = k \phi f'_c h_{ef}^{1.5} = 16 \times \phi 3000 \times 12^{1.5} = 36.4 \text{ k}$   
 $N_{cbg} = A_v / A_{v_o} \phi_1 \phi_2 \phi_3 N_b = 576/1296 \times 1 \times 0.88 \times 1.25 \times 36 = \underline{17.88 \text{ k}}$
4. Concrete Breakout Strength of Anchor in Shear  
 $V_b = 7(l/d_o)^{0.2} \phi d_o \phi f'_c c_1^{1.5} = 7(4/0.5)^{0.2} \times \phi 0.5 \times \phi 3000 \times 11^{1.5} = 17.220 \text{ k}$   
 $V_{cbg} = A_v / A_{v_o} \phi_5 \phi_6 \phi_7 V_b = 396/545 \times 1 \times 0.9 \times 1.4 \times 19.622 = \underline{15.78 \text{ k}}$
5. Pullout Strength of Anchor in Tension  
 Rebar embedded minimum of 11 inches hooked to develop full strength.  
 $N_{pn} = 2 \times 0.196 \times 60 \text{ksi} = 23.52 \text{k}$  per two bars
6. Concrete Pryout Strength of Anchor in Shear  
 $V_{cp} = k_{cp} N_{cb} = 2 \times 17.88 = 35.8 \text{ k}$

#### Allowable Loads

$$\phi N_n = N_u: \underline{0.75 \times 17,880 = 13,410 > 2,506 \text{ OK}}$$

$$\phi V_n = V_u: \underline{0.75 \times 15,780 = 11,830 > 2,022 \text{ OK}}$$

$$\text{If, } V_u > 0.2 \phi V_n \text{ and } N_u > 0.2 \phi N_n, \text{ then: } N_u / \phi N_n + V_u / \phi V_n = 1.2 \\ 2506/13410 + 2022/11830 = \underline{0.187 + 0.171 = 0.36 \text{ OK}}$$

#### Seismic

$$0.75 \phi N_n = N_u: \underline{0.75 \times 0.75 \times 17,880 = 10,000 > 1,964 \text{ OK}}$$

$$0.75 \phi V_n = V_u: \underline{0.75 \times 0.75 \times 15,780 = 8,870 > 1,534 \text{ OK}}$$

$$\text{If, } V_u > 0.2 \phi V_n \text{ and } N_u > 0.2 \phi N_n, \text{ then: } N_u / \phi N_n + V_u / \phi V_n = 1.2 \\ 1964/10000 + 1534/8870 = \underline{0.196 + 0.173 = 0.37 \text{ OK}}$$

#### Post Footings

Wind Overturning Moment =  $1034 \times 3 = 3,103 \text{ ft-lb}$ .

Gravity Load Resisting Moment =  $0.67(3600) \frac{2}{12} + 0.67(2500) \frac{10}{12} = 1799 \text{ ft-lb}$ .

Resultant wind load on fence panel,  $F_w$

$$3103 - 1799 = F_w (3)$$

$$F_w = 435 \text{ lb.}$$

$$\text{Seismic Overturning Moment} = 1180 \times 3 = 3540 \text{ ft-lb.}$$

$$\text{Gravity Load Resisting Moment} = 0.85(3600) \frac{2}{12} + 0.85(2500) \frac{10}{12} = 2281 \text{ ft-lb.}$$

Required hold down force,  $F_E$

$$3540 - 2281 = F_E (3)$$

$$F_E = 420 \text{ lb.}$$

For footing design use 1.3W or E:

$$1.3W = 1.3 \times 435 = \underline{566 \text{ lb.}}$$

$$E = 420 \text{ lb.}$$

For lateral bearing allowable use two times the tabular values per Section 1804.3.1

Footings are 24" in diameter for soil bearing allowable greater than or equal to 1500 psf, with  $f'_c = 3,000$  psi min. The depth for soil types GW and GP (sandy gravel and/or gravel) is 34 inches; for soil types SW, SP, SM, SC, GM and GC (sand, silty sand, clayey sand, silty gravel and clayey gravel) is 39 inches; and, for soil types CL, ML, MH, and CH (clay sandy clay, silty clay, clayey silt, silt and sandy silt) is 45 inches.

Vertical load:

$$[2500 + (150 \times 34/12 \times p_1^2)]/p_1^2 = (2500+425)/3.1416 = 931 \text{ psf} < 3,000 \text{ OK}$$

$$[2500 + (150 \times 39/12 \times p_1^2)]/p_1^2 = (2500+488)/3.1416 = 951 \text{ psf} < 2,000 \text{ OK}$$

$$[2500 + (150 \times 45/12 \times p_1^2)]/p_1^2 = (2500+563)/3.1416 = 975 \text{ psf} < 1,500 \text{ OK}$$

Fence Panels

$$\text{Wind load} = q_z G C = 13 \times 0.85 \times 1.2 = 13.3 \text{ psf} \times 1.6 = \underline{21.3 \text{ psf}}$$

$$\text{Seismic load} = 0.4(150 \times 4/12) = 20.0 \text{ psf}$$

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 DESIGN OF FOOTINGS FOR CANTILEVERED POST  
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DESCRIPTION >> 6' High Fence with Posts at 13' O/C.  
 >> CL,ML,MH SW,SP,SM GW & GP

|                                    |      | ----- | ----- | ----- | ----- |
|------------------------------------|------|-------|-------|-------|-------|
| ALLOW PASSIVE                      | psf  | 200   | 300   | 400   | 0     |
| MAX PASSIVE                        | psf  | 3,000 | 4,500 | 6,000 | 0     |
| LOAD DURATION FACTOR               |      | 1.33  | 1.33  | 1.33  | 1.33  |
| POINT LOAD                         | lbs  | 566   | 566   | 566   | 0     |
| LOAD Ht.                           | ft   | 3.00  | 3.00  | 3.00  | 0.00  |
| DIST. LOAD                         | #/ft | 0     | 0     | 0     | 0     |
| START HT.                          | ft   | 0.00  | 0.00  | 0.00  | 0.00  |
| END HT.                            | ft   | 0.00  | 0.00  | 0.00  | 0.00  |
| WIDTH/DIAMETER                     | in   | 24    | 24    | 24    | 0     |
| CIRCULAR ?                         | y/n  | Y     | Y     | Y     | N     |
| RESTRAINED ?                       | y/n  | N     | N     | N     | N     |
| ----- SUMMARY -----                |      |       |       |       |       |
| Moments @ Surface:                 |      |       |       |       |       |
| Point Load                         | ft-# | 1,697 | 1,697 | 1,697 | 0     |
| Uniform Load                       | ft-# | 0     | 0     | 0     | 0     |
| ... Total Moment                   | ft-# | 1,697 | 1,697 | 1,697 | 0     |
| Total Lateral Load                 | #    | 566   | 566   | 566   | 0     |
| --- NON-RESTRAINED --- RESULTS --- |      |       |       |       |       |
| Min. Req'd Embedment               |      |       |       |       |       |
| $A(1+(1+4.36h/A)^{.5})/2$          |      |       |       |       |       |
| A=2.34P/(S 1 b)                    | ft   | 3.74  | 3.18  | 2.83  | 0.00  |
| Press @ 1/3 Embed.:                |      |       |       |       |       |
| Actual                             | psf  | 331   | 422   | 504   | 0     |
| Allowable                          | psf  | 332   | 423   | 502   | 0     |

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 CONCRETE FENCE PANEL DESIGN & ANALYSIS  
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| DESCRIPTION                  |      | >> Fence Panels--#3 bars @ 12" O/C.<br>>> Maximum Design |           |           |           |
|------------------------------|------|--|-----------|-----------|-----------|
| ----- GENERAL -----          |      | --Span 1-  | --Span 2- | --Span 3- | --Span 4- |
| ALL SPANS SIMPLE SUPPORT ?   | y    | y/n  |           |           |           |
| SPAN LENGTH                  | ft   | 12   | 12        | 0         | 0         |
| END FIXITY.....              | Lft  | 1  | 1         | 1         | 1         |
| Fix,Pin,Free = 2,1,0         | Rt   | 1  | 1         | 1         | 1         |
| BEAM WIDTH                   | in   | 12   | 12        | 0         | 0         |
| BEAM DEPTH                   | in   | 4  | 4         | 0         | 0         |
| ---- CALCULATED VALUES ----  |      | -----  | -----     | -----     | -----     |
| Left: Max Mu                 | ft-k | 0.0  | 0.0       | 0.0       | 0.0       |
| Mn * Phi                     | ft-k | 1.0  | 1.0       | 0.0       | 0.0       |
| Center: Max Mu               | ft-k | 1.0  | 0.6       | 0.0       | 0.0       |
| Mn * Phi                     | ft-k | 1.0  | 1.0       | 0.0       | 0.0       |
| ..Dist. to Mu                | ft   | 6.00   | 6.00      | 0.00      | 0.00      |
| Right: Max Mu                | ft-k | 0.0  | 0.0       | 0.0       | 0.0       |
| Mn * Phi                     | ft-k | 1.0  | 1.0       | 0.0       | 0.0       |
| Max Vu: Left                 | k    | 0.34   | 0.20      | 0.00      | 0.00      |
| Rt                           | k    | 0.34   | 0.20      | 0.00      | 0.00      |
| Reactions: Left: Dead        | k    | 0.00   | 0.00      | 0.00      | 0.00      |
| ..(service) Live             | k    | 0.21   | 0.13      | 0.00      | 0.00      |
| Total                        | k    | 0.21   | 0.13      | 0.00      | 0.00      |
| Right: Dead                  | k    | 0.00   | 0.00      | 0.00      | 0.00      |
| Live                         | k    | 0.21   | 0.13      | 0.00      | 0.00      |
| Total                        | k    | 0.21   | 0.13      | 0.00      | 0.00      |
| Max. Defl @ Mid Span         | in   | -0.071   | -0.043    | 0.000     | 0.000     |
| X-Dist                       | ft   | 6.00   | 6.00      | 0.00      | 0.00      |
| ----- BEAM DESIGN DATA ----- |      | -----  | -----     | -----     | -----     |
| f'c                          | psi  | 4,000  | 4,000     | 3,000     | 3,000     |
| Fy                           | psi  | 60,000   | 60,000    | 60,000    | 60,000    |
| LEFT: As-TOP                 | in^2 | 0.12   | 0.12      | 0         | 0         |
| 'd' TO BARS                  | in   | 2  | 2         | 0         | 0         |
| CENTER: As-BOTTOM            | in^2 | 0.12   | 0.12      | 0         | 0         |
| 'd' TO BARS                  | in   | 2  | 2         | 0         | 0         |
| RIGHT: As-TOP                | in^2 | 0.12   | 0.12      | 0         | 0         |
| 'd' TO BARS                  | in   | 2  | 2         | 0         | 0         |
| ----- APPLIED LOADS -----    |      | -----  | -----     | -----     | -----     |
| USE LL THIS SPAN?            | Y/N  | Y  | Y         | Y         | Y         |
| UNIFORM..... DL              | klf  | 0  | 0         | 0         | 0         |
| LL                           | klf  | 0.035  | 0.02128   | 0         | 0         |

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 RECTANGULAR CONCRETE COLUMN DESIGN  
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DESCRIPTION >> 8' high fence post  
 >>

| DESIGN DATA                    | COLUMN DATA              |
|--------------------------------|--------------------------|
| f'c = 4,000 psi                | DEPTH = 20 in            |
| Fy = 60,000 psi                | WIDTH = 20 in            |
| SEISMIC ZONE (0=wind): 0 0-4   | COLUMN HEIGHT = 8 ft     |
| COMBINE LL & ST ? Y y/n        | CALC P:M DIAGRAM ? N y/n |
| EFF. LENGTH FACTOR = 1         |                          |
|                                | REBARS: COUNT BAR # "d"  |
| IS COLUMN UNBRACED ? y y/n     | SET #1 : 2 5 4 in        |
| IF UNBRACED, Delta:S = 0       | SET #2 : 2 5 16 in       |
|                                | SET #3 : 0 0 0 in        |
| Total Reinf. Area = 1.24 in^2  | SET #4 : 0 0 0 in        |
| % Steel = 0.31 %               | SET #5 : 0 0 0 in        |
| Note: 1% Min < Actual < 8% Max |                          |

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 APPLIED LOADS  
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|                     |        | MOMENTS:       |        |      |
|---------------------|--------|----------------|--------|------|
|                     |        | Top            | Bottom |      |
| AXIAL: DL = 0.111 k |        | DL = 0         | ft-k   | 0    |
| LL = 0.111 k        |        | LL = 0         | ft-k   | 0    |
| ST = 0 k            |        | SHORT = 0      | ft-k   | 0    |
| Ecc. = 0 in         |        |                |        |      |
|                     |        |                | SLoc   | ELoc |
| DISTRIBUTED :       | #1 = 0 | 0.286 klf @ x= | 0      | 8 ft |
|                     | #2 = 0 | 0 klf @ x=     | 0      | 0 ft |
| POINT :             | #1 = 0 | 0 k @ x=       | 0 ft   |      |
|                     | #2 = 0 | 0 k @ x=       | 0 ft   |      |

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 SUMMARY  
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 ACI Equation  
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|                                   |   | 9-1     | 9-2    | 9-3       |
|-----------------------------------|---|---------|--------|-----------|
| Pu : Max. Factored                | = | 0.3441  | 0.2580 | 0.0999 k  |
| Pn * Phi : Capacity @ Design Ecc. | = | 800.903 | 4.1902 | 1.7371 k  |
|                                   |   | OK      | OK     | OK        |
| Mc = Delta:B*M2b + Delta:S*M2S    | = | 0.03    | 2.92   | 2.97 kft  |
| Final Ecc.: Axial*Ecc + Moments   | = | 1.20    | 135.64 | 357.29 in |
| Magnification Factor              | = | 1.000   | 1.000  | 1.000 in  |
| Design Eccentricity               | = | 1.20    | 135.64 | 357.29 in |
| Po * .80 : (short column)         | = | 1144.14 | 1144.1 | 1144.1 k  |
| P : Balanced                      | = | 539.177 | 539.17 | 539.17 k  |
| Ecc: Balanced                     | = | 6.80    | 6.80   | 6.80 in   |

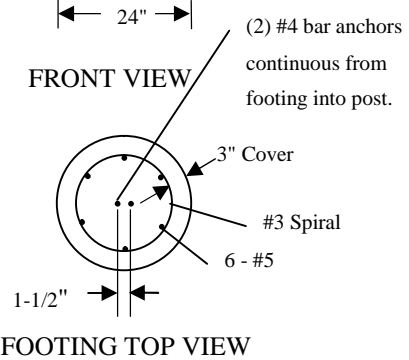
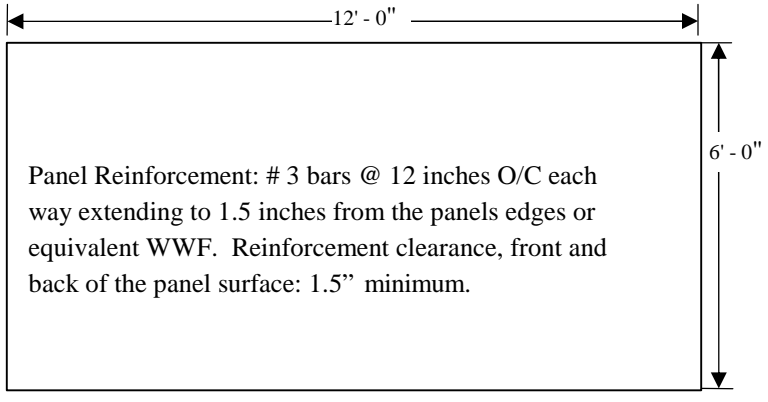
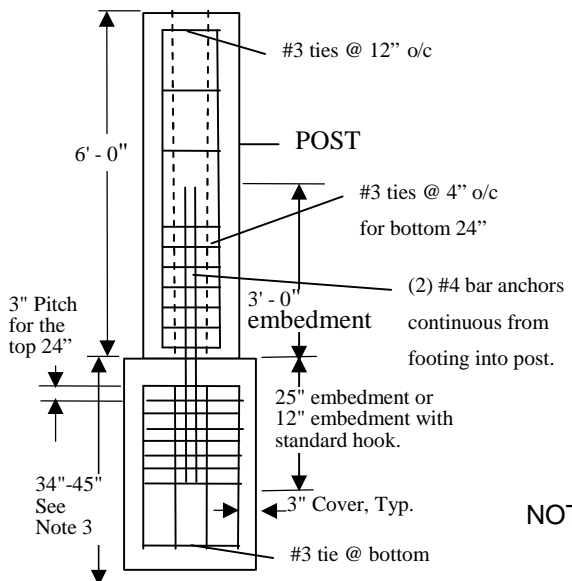
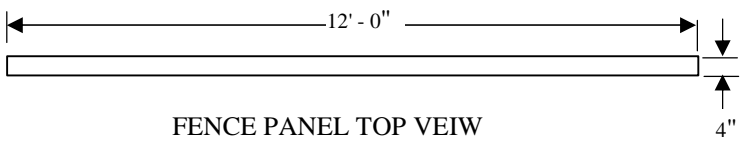
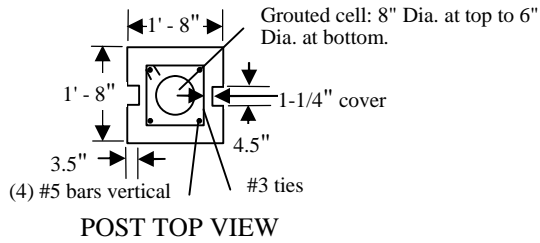
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 SLENDERNESS EFFECTS  
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|                                    |   | Elastic Modulus = 4E+06 psi |       |           |
|------------------------------------|---|-----------------------------|-------|-----------|
| Actual kLu/r = 16.00               |   |                             |       |           |
| Beta = 0.85                        |   |                             |       |           |
|                                    |   | 9-1                         | 9-2   | 9-3       |
| Neutral Axis Distance from Edge    | = | 20.90                       | 1.39  | 1.33 in   |
| Phi                                | = | 0.700                       | 0.705 | 0.702     |
| Max. kLu/r to neglect slenderness  | = | 22.0                        | 22.0  | 22.0      |
| Beta = Sustained / Maximum Moments | = | 0.452                       | 0.452 | 1.000     |
| Cm                                 | = | 1.000                       | 1.000 | 1.000     |
| EI = Max[(.2*Ec*Ig+Es*Is)/(1+Beta) |   |                             |       |           |
| -or- .4 Ec Ig/(1+Beta)]/1000000    | = | 0                           | 0     | 0         |
| Pc : Pi^2 * EI / (kLu)^2           | = | 0.0                         | 0.0   | 0.0       |
| Alpha = Max Pu / ( Phi * Pc )      | = | 0.000                       | 0.000 | 0.000     |
| Delta:b=Cm/(1-Alpha)               | = | 1.000                       | 1.000 | 1.000     |
| Delta:b * M2b                      | = | 0.00                        | 0.00  | 0.00 ft-k |
| Delta:s * M2s                      | = | 0.00                        | 2.92  | 2.97 ft-k |

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Eccentricity: Ecc. Loads + Moments =          1.20 135.64 357.29 in
Design Ecc. = Ecc * Delta          =          1.20 135.64 357.29 in
-----
ACI 9-1 & 9-2 DL =          1.40          ACI 9-2 Group Factor = 0.75
ACI 9-1 & 9-2 LL =          1.70          ACI 9-3 Dead Load Fact= 0.90
ACI 9-1 & 9-2 ST =          1.70          ACI 9-3 Short Term      = 1.30
.... Seismic = ST *          1.10

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**NOTES:**

1.  $F'_c = 4,000$  PSI MINIMUM FOR POST AND PANEL CONCRETE INCLUDING CELL GROUT.
2.  $F'_c = 3,000$  PSI MINIMUM FOR FOOTINGS.
3. FOOTINGS ARE 24" IN DIAMETER. FOOTING DEPTH FOR SOIL TYPES GW AND GP (SANDY GRAVEL AND/OR GRAVEL) IS 34 INCHES; FOR SOIL TYPES SW, SP, SM, SC, GM AND GC (SAND, SILTY SAND, CLAYEY SAND, SILTY GRAVEL AND CLAYEY GRAVEL), 39 INCHES; AND, FOR SOIL TYPES CL, ML, MH, AND CH (CLAY SANDY CLAY, SILTY CLAY, CLAYEY SILT, SILT AND SANDY SILT), 45 INCHES.
4. MINIMUM SOIL BEARING PRESSURE ALLOWABLE: 1500 PSF
5. WIND:  $V = 90$  MPH, EXPOSURE C.
6. SEISMIC: SHORT PERIOD SPECTRAL ACCELERATION,  $S_s = 1.5$  OR LESS; SITE CLASSES A, B, C, OR D ; IMPORTANCE FACTOR,  $I_p = 1.0$