Structural Design

Verticrete Precast Wall Panels, Columns, and Piers

Type 1 Soils

for

Hawk Construction

5002 U.S. Highway 380

Princeton, Texas

By

Carr Consulting, Inc.

153 Wandering Drive
Forney, TX 75126
Registration No. F-2331

A. Wind Load Calculations

1. Basic Wind Speed:
    a. From Figure 6-1: 90 mph is shown on Fig. 6-1. Use 110 mph to be conservative.
        i. V = 110 mph

2. Wind Direction:
    a. From Table 6-4: Solid Signs & Walls,
        i. K_d = 0.85

3. Importance Factor:
    a. From Table 6-1: Category I structure, non-life threatening.
        i. I = 0.87

4. Exposure Category:
    a. Surface Roughness, Category C, open terrain.
    b. From Table 6-2:
        i. a=9.5,
        ii. z_g(ft)=900,
        iii. â=1/9.5,
        iv. b=1.00,
        v. a_{avg}=1/6.5,
        vi. b_{avg}=0.65,
        vii. c=0.20,
        viii. l(ft)=500,
        ix. $\in_{avg}$=1/5.0,
        x. z_{min}(ft)=15

5. Velocity Pressure Exposure Coefficient:
    a. From Table 6-3: Exposure C, <15 ft
        i. K_h and Kz=0.85

6. Topographic Factor:
    a. From 6.5.7.2:
        i. $K_{zt} = (1+K_1K_2K_3)^2$
        ii. K_{zt} = 1, Does not meet conditions for inclusion.

7. Gust Effect Factor:
    a. Use default value:
        i. G = 0.85

8. Enclosure Classification:
    a. Open; generates no internal pressure.

9. Internal Pressure Coefficient:

a. $GC_{pi} = 0.0$

10. External Pressure Coefficient:
    a. From Figure 6-20: B/s>45, s/h=1
        i. $C_f = 1.30$

11. Velocity Pressure:
    a. Section 6.5.14
        i. $q_h = q_z$
        ii. $q_z = 0.00256K_zK_{zt}K_dV^2I$ (psf)
        iii. $q_z = 0.00256*0.85*1*0.85*110^2*0.87$
        iv. $q_z = 19.47$ psf

12. Design Wind Load:
    a. From Section 6.5.14, Eqn (6-27)
    b. $F = q_hGC_fA_s$ (lbs)
        i. $F = 19.47*0.85*1.30*A_f$
        ii. $F = 21.51A_f$ lbs

13. Area Normal to Wind on Panel
    a. For 8 ft Panel:
        i. $A_f = HW$
        ii. H = 8 ft
        iii. W= 12 ft
        iv. $A_f = 8*12\ ft^2$
        v. $A_f = 96\ ft^2$

14. Area Normal to Wind on Column
        i. $A_f = HW$
        ii. H = 8 ft
        iii. W= 1.67 ft
        iv. $A_f = 1.67*8\ ft^2$
        v. $A_f = 13.36\ ft^2$

15. Total Force on Panel
    a. $F_{panel} = 21.51A_f$ lbs
    b. $F_{panel} = 21.51*96$
    c. $F_{panel} = 2,065$ lbs

16. Force on Column
    a. $F_{col} = 21.51A_f$ lbs
    b. $F_{col} = 21.51*13.36$
    c. $F_{col} = 287.4$ lbs

17. Total Force on Column
    a. $F_{ctot} = F_{panel} + F_{col}$
    b. $F_{ctot} = 2,065 + 287$
    c. $F_{ctot} = 2,352$ lbs

18. Height of Force
    a. From Figure 6-20:
        i. $h_F = 0.55h$
        ii. $h_F = 0.55*8$ ft
        iii. $h_F = 4.4$ ft

B. Structural Calculations

1. Material Properties
    a. Steel
        i. Ultimate Tensile Strength: $f_y = 60{,}000$ psi
        ii. Welded Wire Reinforcement: 8x8 W2.9xW2.9
            1. Area of Steel Vertical = Area of Steel Horizontal
                a. $A_{sv} = A_{sh}$
                b. $A_{sv} = 0.029$ in^2 x 12 in/8 in
                    i. $A_{sv} = 0.0435$ in^2/ft
                c. % = 0.435 in^2 / (4.5 in x 12 in) = 0.087%
    b. Concrete
        i. 28 day Compressive Strength: $f_c = 4{,}000$ psi
        ii. Shear Strength: $V_c = 2\text{sqrt}(f'c)b_wd$
            1. $V_c = 126.5b_wd$
        iii. Unit Weight: $w_c = 150$ lbs/ft^3
        iv. Modulus of Elasticity:
            1. $E_c =$
2. Dimensions
    a. Panel
        i. Length: l = 12 ft
        ii. Height: h= 8 ft
        iii. Thickness: t = 5 in
        iv. Distance to Steel = t/2 = 2.5 in
        v. Area: = lh = 96 ft^2
    b. Column
        i. Length: l = 20 in
        ii. Height: h = 8 ft
        iii. Thickness: t = 20 in
        iv. Distance to Steel Centroid: 10 in
3. Weights
    a. Panel
        i. Thickness (avg): tavg = 5.5 in
        ii. $W_{panel} = lht_{avg}w$
            1. W_{panel} = 12 ft x 8 ft x 5.5 in/12in/ft x 150 lbs/ft^3
            2. W_{panel} = 6,600 lbs

b. Column
   i. W_{column} = lhtw
      1. W_{column} =8 ft x 20 in x 20 in / 144in^2/ft^2 x 150 lbs/ft^3
      2. W_{column} = 3,333 lbs

c. Combined Weight
   i. $W_{combined} = W_{panel} + W_{column}$
      1. $W_{combined}$ = 6,600 lbs +3,333 lbs = 9,933 lbs (10,000 lbs, approx.)

4. Structural Properties

a. Panel
   i. Gross Moment of Inertia
      1. $I_g = bh^3/3$ = 12 in x (5 in)^3/3 = 125 in^4

   ii. Nominal Moment
      1. $M_n = A_s f_y(d-\lambda)$ = 0.0435 x 60000*(2.5-λ)
      2. $A_c = f_y A_s/(0.85f'_c)$ = 60000 x 0.0435/(0.85 x 4000) = 0.759 in^2
      3. $\lambda = A_c/2b$ =0.759/2/12 = 0.032 in (approx. = 0)
      4. M_n = 14,108 in-lbs
      5. M_n = 1,176 ft-lbs

   iii. Design Moment
      1. Φ = 0.85
      2. $M_D = \Phi M_n$
      3. M_D = 0.85 x 1,176 ft-lbs = 999 ft-lbs

   iv. Actual Moment
      1. $M_a = wl^2/8$
      2. M_a = 21.51 x 11.33^2 / 8 =345 ft-lbs

   v. Cracking Moment
      1. $M_{cr} = f_r I_g/y_t$
      2. f_r = 7.5 sqrt(f'_c)
      3. f_r = 7.5 x sqrt(4,000) = 474 psi
      4. y_t = d = 2.5
      5. M_{cr} = 474 psi x 125 in^4 / 2.5 in = 19,211 in-lbs
      6. M_{cr} = 1,601 ft-lbs

   vi. Moment Design Check
      1. $M_a < M_D$
      2. $M_a < M_{cr}$
      3. **Panels can withstand the moments created by a 110 mph wind.**

   vii. Shear Forces
      1. $V_n = V_c + V_s$
      2. V_c = 2sqrt(f'_c)b_wd

3. b_w = 4.5 in
4. d = 6 in
5. V_c = 2 x sqrt(4,000) x 4.5 x 6 = 3,415 lbs
6. Weight of panel at pedestal = W_v
7. $W_v = W_{panel}/2$
8. W_v = 6,600/2 =3,300 lbs
9. Design Check
    a. $W_p < V_c$
    b. **Panels can withstand vertical shear forces w/o reinforcement**

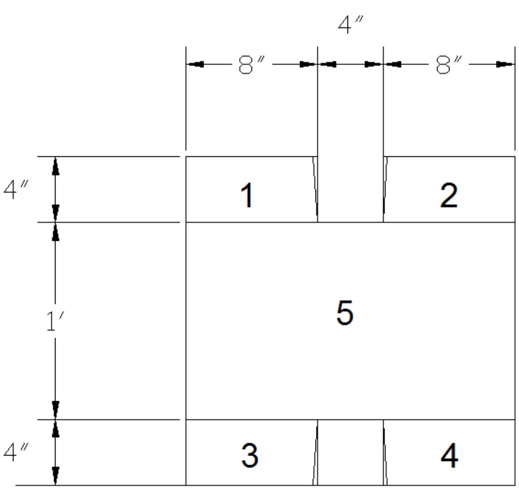
10. Horizontal Shear, at column face
    a. W_h = Horiz. Shear Force
    b. $W_h = F_{ctot}/2$
    c. W_h = 2,352/2 = 1,176 lbs
    d. $V_c = 2\text{sqrt}(f'_c)b_wd$
    e. b_w = 4.5 in
    f. d = 96 in
    g. V_c = 2 x sqrt(4,000) x 4.5 x 96 = 54,644 lbs
    h. Design Check
        i. $W_h < V_c$
        ii. **Panels can withstand horizontal shear forces w/o reinforcement.**

viii. Panel Design Check
1. **Moment passes**
2. **Shear passes**
3. **Serviceability passes**

b. Column
   i. Material Properties
      1. Steel
         a. Ultimate Tensile Strength: f_y = 60,000 psi
         b. Welded Wire Reinforcement: 8x8 W2.9xW2.9
            i. Area of Steel Vertical = Area of Steel Horizontal
               1. $A_{sv} = A_{sh}$
               2. $A_{sv} = 0.029\ in^2 \times 12\ in/8\ in$
               3. $A_{sv} = 0.0435\ in^2/ft$
               4. % = $0.435\ in^2$ / (4.5 in x 12 in) = 0.087%

c. Concrete
   i. 28 day Compressive Strength: f_c = 4,000 psi
   ii. Shear Strength: $V_c = 2sqrt(f'c)b_wd$
      1. $V_c = 126.5b_wd$
   iii. Unit Weight: $w_c = 150\ lbs/ft^3$

d. Dimensions



e. Structural Properties

   i. Gross Moment of Inertia
      1. $I_{c1} = bh^3/3 = 8\ in \times (4\ in)^3/3 = 171\ in^4$
      2. $I_{c2} = bh^3/3 = 8\ in \times (4\ in)^3/3 = 171\ in^4$
      3. $I_{c3} = bh^3/3 = 8\ in \times (4\ in)^3/3 = 171\ in^4$
      4. $I_{c4} = bh^3/3 = 8\ in \times (4\ in)^3/3 = 171\ in^4$
      5. $I_{c5} = bh^3/3 = 20\ in \times (12\ in)^3/3 = 11{,}520\ in^4$
      6. $I_c = I_{c3} + I_{c4} + A_5d_5^2 + A_1d_1^2 + A_2d_2^2$
      7. $I_c = 171 + 171 + 20x12 \times 10^2 + 4x8x18^2 + 4x8x18^2$
      8. $I_c = 45,078\ in^4$
         a. Note: I_g calculated from center of steel to edge

ii. Nominal Moment
1. Assume Center of Steel is the Edge of Column @ y=0
2. As = 2 x 0.31 in^2 = 0.62 in^2 (2-#5 bars)
3. β_1 = 0.85
4. $a_b = b_1(87{,}000/(87{,}000+f_y)d$
5. a_b = 0.85 x (87,000/(87,000+60,000)*10 in
6. a_b = 5.03 in
7. A_{cb} = 16 x 4 + 20x(5.03 – 4) = 84.6 in^2
8. $A_{sb} = 0.85f'_cA_{cb}/f_y$
9. A_{sb} = 0.85 x 4,000 x 84.6 / 60,000
10. A_{sb} = 4.79 in^2
11. $A_{smax} = 0.75A_{sb}$
12. A_{smax} = 3.60 in^2
13. $\lambda = A_{cb}/2b$ =84.6/2/20 = 2.12 in
14. $M_n = A_sf_y(d-\lambda)$ = 0.62 x 60000*(10-2.12)
15. M_n = 293,136 in-lbs
16. M_n = 24,428 ft-lbs

iii. Design Moment
1. Φ = 0.85
2. $M_D = \Phi M_n$
3. M_D = 0.85 x 24,428 ft-lbs = 20,763 ft-lbs

iv. Actual Moment
1. $M_a = wl^2/2$
2. w = 2352 lbs/8 ft = 294 plf
3. M_a = 294 x 8^2 / 2 = 9,408 ft-lbs

v. Cracking Moment
1. $M_{cr} = f_rI_g/y_t$
2. f_r = 7.5 sqrt(f'_c)
3. f_r = 7.5 x sqrt(4,000) = 474 psi
4. y_t = d = 10
5. M_{cr} = 474 psi x 45,078 in^4 / 10 in = 213,697 in-lbs
6. M_{cr} = 178,058 ft-lbs

vi. Design Check
1. $M_a < M_D$
2. $M_a < M_{cr}$
3. **Columns will not fail in moment at 110 mph.**

vii. Shear Strength
1. The size of the columns is large enough to make calculation unnecessary for shear since shear loads are small, and pass on the smaller cross-section of the panels.
2. **Columns pass on shear strength.**

viii. **Column Design Check**

1. **Column passes on moment**
2. **Column passes on shear**
3. **Column passes on serviceability**

5. Piers
   a. Horizontal Resistance; Use NAVFAC Figure 7.2-114 (Passive Pressure Distribution for Soldier Piles)

   b. End Bearing; Use Safety Factor = 3, for allowable end bearing pressures.

   c. Materials
      i. Steel
         1. Ultimate Tensile Strength: f_y = 60,000 psi
         2. Steel used; 4 - #5 bars, A_{actual} = 1.23 in^2

      ii. Concrete
         1. 28 day Compressive Strength: f_c = 3,000 psi
         2. Unit Weight: w_c = 150 lbs/ft^3

   d. Dimensions
      i. Radius: r = 9 in
      ii. Depth 1: d_1 = 8 ft
      iii. Cross Sectional Area: $A_{cross} = \prod r^2 = 3.14 \times 0.75^2 = 1.77\ ft^2$
      iv. Volume 1: $V_1 = A_{cross}d_1 = 1.77\ ft^2 \times 8\ ft = 14.14\ ft^3$
      v. Weight 1: $W_1 = V_1 w = 14.14\ ft^3 \times 150\ lbs/ft^3 = 2{,}120$ lbs

   e. Loading and Capacities
      i. Vertical Soil Loading
         1. $W_v = W_{panel} + W_{column} + W_{peir}$
         2. W_v = 10,000 + 2,150 = 12,150 lbs

      ii. Vertical Soil Capacity
         1. S_{uc} = 2,500 psf
         2. End Bearing Capacity
            a. $P_{end} = A_{pier}S_{uc}$
            b. $P_{end} = \prod r^2 S_{uc}$
            c. $P_{end} = 3.14 \times 0.75^2 \times 2{,}500 = 4{,}418$ lbs

         3. Side Friction
            a. $l_f = l - 2ft$ (ignore the top 2 feet)
            b. l = 8 ft
            c. lf = 6 ft
            d. $A_f = 2\prod r l_f$
            e. $A_f = 2 \times 3.14 \times 0.75\ ft \times 6\ ft = 28.3\ ft^2$
            f. S_f = 500 psf
            g. $P_{side} = S_f A_f$
            h. $P_{side} = 500\ lb/ft^2 \times 28.3\ ft^2 = 14{,}137$ lbs

         4. Reinforcing Steel
            a. $A_s = (P_{side} - W_{panel} - W_{column})/f_y$

b. A_s = 14,137 lbs / 60,000 lb/in^2 = 0.23 in^2
c. A_{actual} = 1.23 in^2
d. Design Check,
   i. $A_{actual} > A_s$
   ii. 1.23 in^2 > 0.23 in^2
   iii. **Steel passes required strength design.**

5. Total Capacity
   a. $P_{tot} = P_{end} + P_{side}$
   b. P_{tot} = 4,418 lbs + 14,137 lbs = 18,555 lbs

6. Design Check, Vertical Bearing Capacity
   a. $P_{tot} > W_{combined}$
   b. 18,555 lbs > 10,000 lbs
   c. **Vertical soil bearing will adequately support the dead loads.**

iii. Horizontal Soil Capacity
1. $K_p = \tan^2(45+\Phi/2)$
2. $\Phi = 30^o$
3. $K_p = \tan^2(60^o) = 3$
4. b = 1.5 ft
5. γ = 125 pcf
6. $\sigma_{1.5b} = b\gamma$
7. σ_b = 1.5ft x 125 lbs/ft^2 = 188 psf
8. σ_D = 8ft x 125 lbs/ft^2 = 1,000 psf
9. $P_h = (\sigma_b + \sigma_D)/2$ x (D-b) x K_p^2 x b
10. P_h = 1188/2 x 6.5 x 9 x 1.5 = 52,124 lbs

iv. Horizontal Soil Load
1. Select pivot point at depth “b” below the soil.
2. F_w = 2,325 lbs
3. $d_w = b + 0.55h_{panel}$
4. d_w = 1.5 ft + 0.55(8 ft)
5. d_w = 5.9 ft
6. M_w = Fw*l
7. M_w = 5.9 ft x 2,350 lbs = 13,865 ft-lbs
8. $w_{soil} = P_h/(D-b)$
9. w_{soil} = 52,124/(8-1.5) = 8,019 plf
10. $M_{soil} = w_{soil}(D-b)^2/2$
11. M_{soil} = 8,019 *6.5^2/2 = 169,401 ft-lbs

v. Design Check
1. $M_{soil} > M_{wind}$
2. 169,401 > 13,865
3. **Pier lateral soil pressure will adequately resist the wind load.**

C. Structural Summary

a. Wind Loads
   i. 2,350 lbs force @ 110 mph 3 second gust.
   ii. Panel Passes
   iii. Column Passes
   iv. Pier Passes

b. Dead Loads
   i. Panel Weight – 6,600 lbs
   ii. Column Weight – 3,300 lbs
   iii. Pier Weight – 2,120 lbs
   iv. Total Dead Load – 12,150 lbs

c. Design passes on all counts.

If you have any questions or comments, feel free to contact me at 972-786-5401.

Sincerely,

John P. Carr, P.E.