



Fence & Pier Structural Calculations
West Loop Substation, Weatherford, TX
March 26, 2015

Fence Design

Wind Loads

1.0	Basic Wind Speed:	V =	130 mph	From Figure 6-1: 90 mph is shown on Fig. 6-1. Use 100 mph to be conservative.
2.0	Wind Direction:	K_d =	0.85	From Table 6-4: Solid Signs & Walls
3.0	Importance Factor:	I =	0.87	From Table 6-1: Category I structure, non-life threatening.
4.0	Exposure Category:	a =	9.5	Surface Roughness, Category C, open terrain. From Table 6-2:
		z_g (ft) =	900	
		\hat{a} =	0.11	
		b =	1	
		a_{avg} =	1.54	
		b_{avg} =	0.65	
		c =	0.2	
		l(ft) =	500	
	v_{avg} =	0.2		
	z_{min} (ft) =	15		
5.0	Velocity Pressure Exposure Coefficient:	K_h and K_z =	0.9	From Table 6-3: Exposure C, >15 ft
6.0	Topographic Factor:	K_{zt} =	1	Does not meet conditions for inclusion.
7.0	Gust Effect Factor:	G =	0.85	Use default value:
8.0	Enclosure Classification:	Open		Generates no internal pressure.
9.0	Internal Pressure Coefficient:	GC_{pi} =	0	
10.0	External Pressure Coefficient:	C_f =	1.3	From Figure 6-20: B/s>45, s/h=1
11.0	Velocity Pressure:	q_z =	28.79 psf	Section 6.5.14
12.0	Design Wind Load:	F_D =	31.82 lbs	From Section 6.5.14, Eqn (6-27)
13.0	Area Normal to Wind on Bottom Panel	H_{pb} =	10 ft	Panel Height
		W_{pb} =	11.5 ft	Panel Width
		A_{tbp} =	115 ft ²	Panel Area
14.0	Area Normal to Wind on Top Panel (if stacked panels are used)	H_{pt} =	0 ft	Panel Height
		W_{pt} =	11.5 ft	Panel Width
		A_{tpt} =	0 ft ²	Panel Area
14.0	Area Normal to Wind on Top Panel (if stacked panels are used)	H_{pt} =	0 ft	Panel Height
		W_{pt} =	11.5 ft	Panel Width
		A_{tpt} =	0 ft ²	Panel Area
15.0	Area Normal to Wind on Column	H_c =	10 ft	
		W_c =	1.67 ft	
		A_{fc} =	16.67 ft ²	
16.0	Total Force on Panel(s)	F_{panel} =	3,659.30 lbs	
17.0	Force on Column	F_{col} =	530.33 lbs	
18.0	Total Force on Column	F_{ctot} =	4,189.63 lbs	
19.0	Height of Force	h_F =	5.5 ft	From Figure 6-20:

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Structural Calculations			
20.0	Material Properties, Steel	$f_y =$	60,000 psi Ultimate Tensile Strength:
21.0	Panel Steel Reinforcement	$A_{sv} =$	0.029 in ² Area of vert single wire 8x8 W2.9xW2.9
		$d_v =$	0.50 ft Vert spacing
		$d =$	1.00 ft
		$A_{sv} =$	0.0580 in ²
		$b_w =$	4.5 in
		$\rho_s =$	0.107%
22.0	Material Properties, Concrete	$f_c =$	4,700 psi 28 day Compressive Strength
		$V_c =$	3,702.05 lbs Shear Strength:
		$w_c =$	150.0 lbs/ft ³ Unit Weight:
23.0	Panel Dimensions	$l_p =$	11.5 ft Length:
		$h_{pn} =$	10 ft Height
		$t_p =$	4.5 in Thickness
		$t/2 =$	2.25 in Distance to steel
		$l_h =$	115 Area
24.0	Column Dimensions	$l_c =$	20 in Length:
		$h_c =$	10 ft Height
		$t_c =$	20 in Thickness
		$d_{c1} =$	10 in Distance to steel centroid
25.0	Panel Weight	$t_{avg} =$	5 in Thickness (avg):
		$W_{panel} =$	7,188 lbs Weight of both panels
26.0	Column Weight	$W_{column} =$	4,167 lbs
27.0	Combined Weight	$W_{combined} =$	11,354 lbs
28	Panel Gross Moment of Inertia	$I_g =$	500 in ⁴ 1 ft strip
29	Nominal Moment	$A_c =$	0.871 in ² Compression Area
		$l =$	0.036 in (approx. = 0)
		$M_n =$	7,704 in-lbs Nominal Moment
		$M_n =$	641.97 ft-lbs
30	Design Moment	$\phi =$	0.85 Factor
		$M_D =$	545.68 ft-lbs Design Moment
31	Actual Moment	$M_a =$	495.97 ft-lbs Actual Moment
32	Cracking Moment	$f_r =$	514 psi
		$y_t = d$	
		$y_t =$	2.25
		$M_{cr} =$	9,522 ft-lbs Cracking Moment
33	Moment Design Check	$M_a < M_D$	Passed
		$M_a < M_{cr}$	Passed
34	Shear Forces	$V_c =$	3,702.05
35	Weight of Panel at Pier	$W_v =$	3,593.75
36	Design Check	$W_v < V_c$	No shear reinforcement required
37	Additional Reinforcement	$A_{vp} =$	(0.003) in ² required
			1 #3 Required
38	Horizontal Shear, at column face	$W_h =$	1,829.65 lbs
		$d =$	96 in
		$b_w =$	4.5 in
		$V_c =$	59,233 lbs
39	Design Check		Pass, no steel required along column face
40	Panel Design Check		Passed Design Moment
			Passed Cracking Moment
		Shear at Base	No shear reinforcement required, 1#3
	Shear along Column Face		Pass, no steel required along column face

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41	Gross Moment of Inertia	$y' = 0$ $d_1 = 6.5$ $d_2 = 0.0$ $d_3 = -6.5$ $b_1 = 20.0$ $b_2 = 14.0$ $b_3 = 20.0$ $h_1 = 7.0$ $h_2 = 3.0$ $h_3 = 0.0$ $I_{cx1} = 2,287 \text{ in}^4$ $I_{cx2} = 126 \text{ in}^4$ $I_{cx3} = - \text{ in}^4$ $I_{xx} = 16,529 \text{ in}^4$	Dimensions
Note: I_x calculated y' from the X axis			
42	Nominal Moment	$A_s = 1.77 \text{ in}^2$ $\beta_1 = 0.85$ $d = 12.2 \text{ in}$ $a_b = 6.12 \text{ in}$ $A_{cb} = 77.53 \text{ in}^2$ $A_{sb} = 5.16 \text{ in}^2$ $A_{smax} = 0.75A_s b \text{ in}^2$ $A_{smax} = 3.87 \text{ in}^2$ $\lambda = 1.94 \text{ in}$ $M_n = 1,085,125 \text{ in-lbs}$ $M_n = 90,427 \text{ ft-lbs}$	4-#6 Assume Center of Steel is the center of Column @ $y'=0$ From the edge to the centroid of the far flange, ignore the web.
43	Design Moment	$\phi = 0.85$ $M_D = 76,863 \text{ ft-lbs}$	
44	Actual Moment	$w = 418.96 \text{ plf}$ $M_a = 20,948 \text{ ft-lbs}$ $f_r = 7.5 \text{ sqrt}(f'_c) \text{ psi}$	
45	Cracking Moment	$f_r = 514.2 \text{ psi}$ $y_t = 12.1725 \text{ in}$ $M_{cr} = 58,184 \text{ ft-lbs}$	
46	Moment Design Check	$M_a < M_D$ Passed $M_a < M_{cr}$ Passed	
47	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	
Column Design Check		1. Column passes on moment 2. Column passes on shear 3. Column passes on serviceability	

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Piers			
Horizontal Resistance; Use NAVFAC Figure 7.2-114 (Passive Pressure Distribution for Soldier Piles)			
End Bearing; Use Safety Factor = 3, for allowable end bearing pressures.			
Geotechnical Investigation: Rone Engineering, February 2013; Report No. 13-18024			
48	Safety Factor	$F_s =$	3 Safety Factor
50	Concrete Compressive Strength	$f_c =$	3,000 psi 28 day Compressive Strength
51	Dimensions	$r =$	9 in Pier Radius
		$d_1 =$	10 ft Depth 1, = height of column +3 ft, expansive soil
		$A_{cross} =$	1.77 ft ² Pier Cross Sectional Area
		$V_1 =$	17.67 ft ³ Volume 1
52	Pier Weight	$W_1 =$	2,651 lbs Weight 1
53	Loading and Capacities	$W_v =$	14,005 lbs Vertical Soil Loading
	Vertical Soil Capacity	$S_{uc} =$	1500 psf Soil compressive strength
	End Bearing Capacity	$P_{end} =$	2,651 lbs
	Side Friction	$l =$	10 ft
		$l_f =$	10 ft
	$A_f =$	47.1 ft	
	$S_f =$	950 psf	
$P_{side} =$	44,768 lbs		
49	Reinforcing Steel	Bar No. =	6 Vertical Steel in Piers
		No. Bars =	4
		Bar No. =	4
		No. Bars =	4
		$A_{actual} =$	2.56 in ²
		$A_s =$	0.93 in ²
50	Design Check, Steel Vertical Reinforcement	$A_{actual} > A_s$	Steel passes required strength design
51	Total Capacity	$P_{tot} =$	47,418 lbs
52	Design Check, Vertical Bearing Capacity	$P_{tot} > W_v$	Passes, will support vertical dead loads.
53	Horizontal Soil Capacity	$\phi =$	35 degrees Friction angle
		$K_p = \tan^2(45 + \phi/2)$	
		$K_p =$	3.69
		$b =$	3 ft Pivot depth
		$\gamma =$	125 pcf Soil density
		$\sigma_b =$	375 plf
		$\sigma_1 =$	1,250 plf
		$P_h =$	232,346 lbs
54	Horizontal Soil Load	$F_w =$	4,189.63 lbs
		$d_w =$	9 ft
		$M_{wind} =$	35,612 ft-lbs
		$W_{soil} =$	33,192
		$M_{soil} =$	542,142
55	Design Check	$M_{soil} > M_{wind}$	Pier lateral soil pressure will adequately resist the wind load.

John P. Carr

