Carr Consulting Inc.
Engineers - Scientists - Planners

Reg# F-2331

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.
		z_g(ft)=	900		From Table 6-2:
		â=	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 Kz=	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_{fbp} =	115	ft^2	Panel Area
14.0	Area Normal to Wind on Top Panel	H_{pt} =	0	ft	Panel Height
	(if stacked panels are used)	W_{pt} =	11.5	ft	Panel Width
		A_{fpt} =	0	ft^2	Panel Area
14.0	Area Normal to Wind on Top Panel	H_{pt} =	0	ft	Panel Height
	(if stacked panels are used)	W_{pt} =	11.5	ft	Panel Width
		A_{fpt} =	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:

Structural Calculations

20.0	Material Properties, Steel	f_y =	60,000	psi	Ultimate Tensile Strength:
21.0	Panel Steel Reinforcement	A_{vw} =	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	
		ρ_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
		lh =	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	ϕ =	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**		

41	Gross Moment of Inertia	y' =	0	Dimensions
		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 in^4	
		I_{cx2} =	126 in^4	
		I_{cx3} =	- in^4	
	Note: I_x calculated y' from the X axis	I_{xx} =	16,529 in^4	
42	Nominal Moment	A_s =	1.77 in^2	4-#6
		β_1 =	0.85	Assume Center of Steel is the center of Column @ y'=0
		d =	12.2 in	From the edge to the centroid of the far flange, ignore the web.
		a_b =	6.12 in	
		A_{cb} =	77.53 in^2	
		A_{sb} =	5.16 in^2	
		A_{smax} =	0.75Asb in^2	
		A_{smax} =	3.87 in^2	
		λ =	1.94 in	
		M_n =	1,085,125 in-lbs	
		M_n =	90,427 ft-lbs	
43	Design Moment	ϕ =	0.85	
		M_D =	76,863 ft-lbs	
44	Actual Moment	w =	418.96 plf	
		M_a =	20,948 ft-lbs	
		f_r =	7.5 sqrt(f'$_c$) psi	
45	Cracking Moment	f_r =	514.2 psi	
		y_t =	12.1725 in	
		M_{cr} =	58,184 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		

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	Φ =	35	degrees	Friction angle
		K_p =	$\tan^2(45+\Phi/2)$		
		K_p =	3.69		
		b =	3	ft	Pivot depth
		γ =	125	pcf	Soil density
		σ_b =	375	plf	
		σ_l =	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.**	

P.E.

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