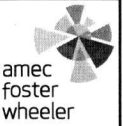


JOB NO. 62521508.01 SHEET 1 OF 15
 PHASE SCULPTURE TASK FOUNDATION DES
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Risk Category: I ASCE 7-10, Table 1.5-1
 Design Wind Speed: 105 mph ASCE 7-10

$$q_z = 0.00256 K_z K_{zt} K_d V^2 \quad \text{Eq 29.3-1, ASCE 7-10}$$

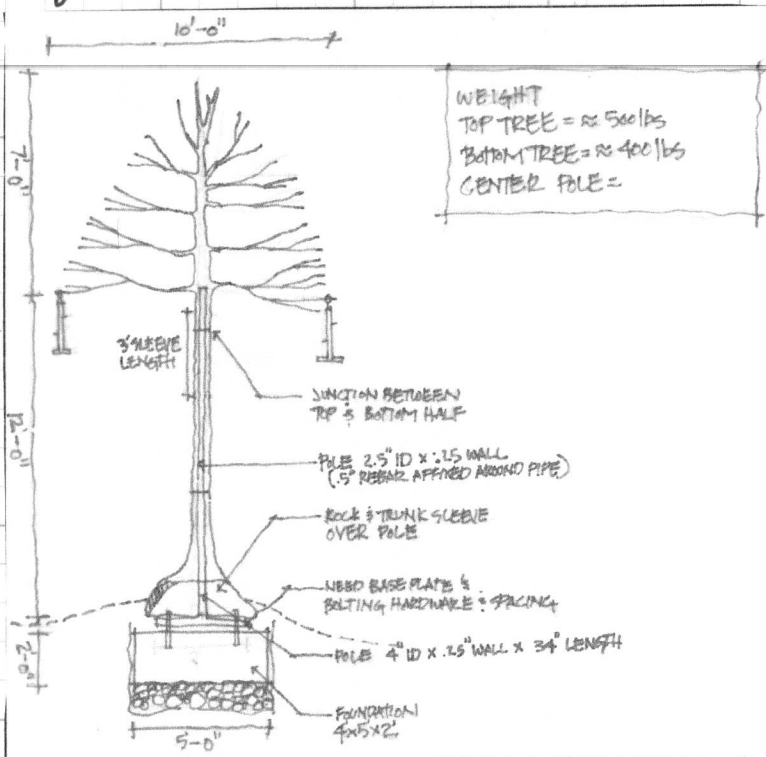
Height $\approx 20'$

$K_d = 0.85$ TABLE 26.6-1, "Open Signs and Lattice Framework", ASCE 7-10

$K_z = 0.90$ TABLE 29.3-1, $H = 20'$, Exposure C, ASCE 7-10

$K_{zt} = 1.0$ (Assumed) ASCE 7-10, 26.8.2

$$\rightarrow q_z = 0.00256 (0.85)(0.90)(1.0)(105)^2 = 21.6 \text{ psf}$$



SKETCH BY
 CLIENT

Wind Force

$$F = q_z G C_f A_f \quad (\text{Eq 29.5-1})$$

Assume $G = 1.0$ (CONSERVATIVE)

Assume Avg Width of 8" from the base to bottom branch (12')

Assume void ratio, $\epsilon = 0.3$ for upper portion (7')

$$A_{f1} \text{ of TRUNK} = (8/12)(12') = 8 \text{ ft}^2$$

A_{f2} of upper portion of sculpture:

$$A_{f2} = \frac{1}{2}(\epsilon)(b)(h) = \frac{1}{2}(0.3)(10')(7')$$

$$A_{f2} = 10.5 \text{ ft}^2$$

$$\text{USE } D = 0.67'$$

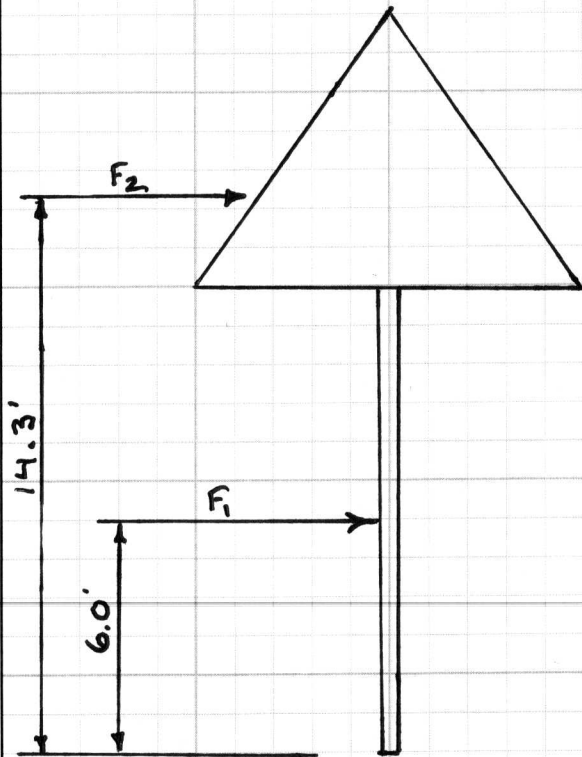
$$D\sqrt{q_z} = 0.67\sqrt{21.6} = 3.11$$

$$C_f \approx 1.1 \quad (\text{Fig. 29.5-2})$$

JOB NO. 62521508.01 SHEET 2 OF 15
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$$F_1 = 21.6 \text{ psf} (1.0) (1.1) (8 \text{ ft}^2) = 190 \#$$

$$\approx 200 \#$$

$$M_1 = (200 \#) (6') = 1200 \# \text{ lbs ft}$$

$$F_2 = 21.6 \text{ psf} (1.0) (1.1) (10.5 \text{ ft}^2) = 249.5 \#$$

$$\approx 250 \#$$

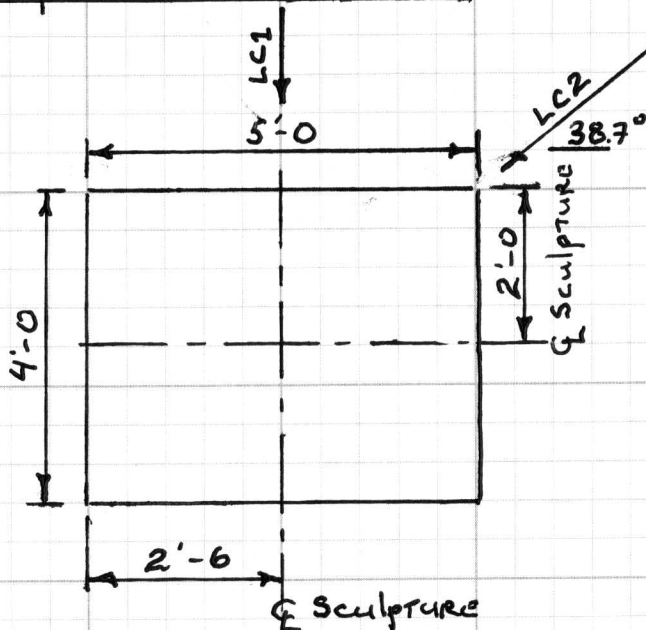
$$M_2 = (250 \#) (14.3') = 3575 \# \text{ ft}$$

$$\sum F_w = F_1 + F_2 = 200 \# + 250 \# = 450 \#$$

$$\sum M_w = M_1 + M_2 = 1200 \text{ ft} \cdot \text{lbs} + 3575 \text{ ft} \cdot \text{lbs}$$

$$= 4775 \text{ ft} \cdot \text{lbs}$$

Proposed Footing Size:



Assume Allowable Bearing Pressure = 2000 psf

Use Allowable Strength Design

By observation, ASCE 7-10, SECT 2.4, Load Combination 7 controls

$$0.6D + 0.6W$$

Where:

D = Dead Load of Sculpture

W = Wind Load on Sculpture

JOB NO. 62521508.01 SHEET 3 OF 15
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By observation, Two load cases to be considered:

- 1) Wind Transverse to Longitudinal Axis (LC1)
- 2) Wind diagonal with opposing corners (LC2)

Dead Load of Sculpture ≈ 900 lbs (FROM CLIENT, SEE PAGE 2)

Assume Foundation Thickness $\approx 2'-0$

Assume Coefficient of Static Friction (μ_s) between Concrete + Soil = 0.35

Neglect Passive Pressure of Soil (CONSERVATIVE)

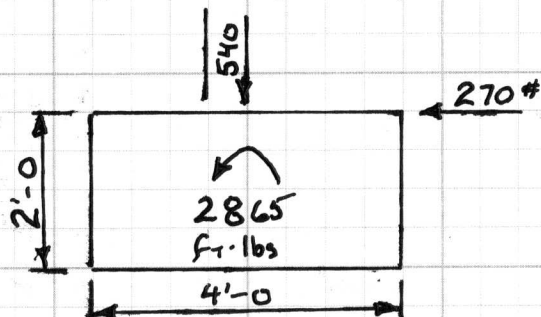
Load Case 1

$$0.6(900 \text{ lbs}) + 0.6(F_w) + 0.6(M_w)$$

$$0.6(900) = 540 \text{ lbs}$$

$$0.6(F_w) = 0.6(450) = 270 \text{ lbs}$$

$$0.6(M_w) = 0.6(4775) = 2865 \text{ ft}\cdot\text{lbs}$$



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FROM ALEX TOMANOVICH'S "FOOTINGS" CALCULATION SPREADSHEET.

Input Data:

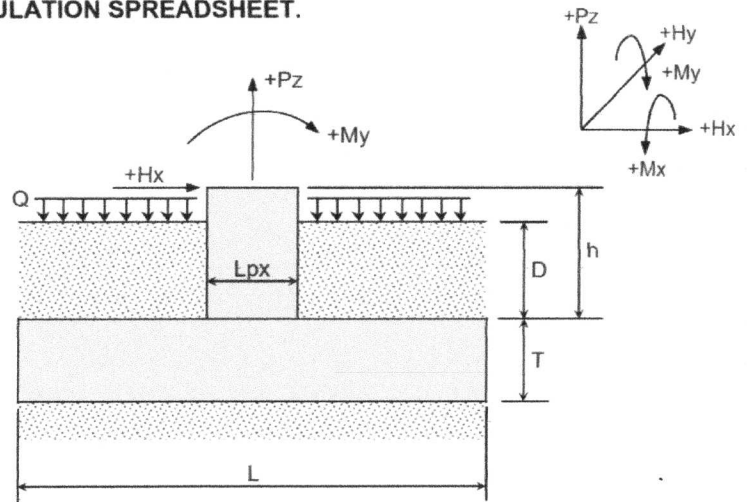
LOAD CASE 1:

Footing Data:

Footing Length, L = 5.000 ft.
 Footing Width, B = 4.000 ft.
 Footing Thickness, T = 2.000 ft.
 Concrete Unit Wt., γ_c = 0.150 kcf
 Soil Depth, D = 0.000 ft.
 Soil Unit Wt., γ_s = 0.000 kcf
 Pass. Press. Coef., Kp = 3.000
 Coef. of Base Friction, μ = 0.350
 Uniform Surcharge, Q = 0.000 ksf

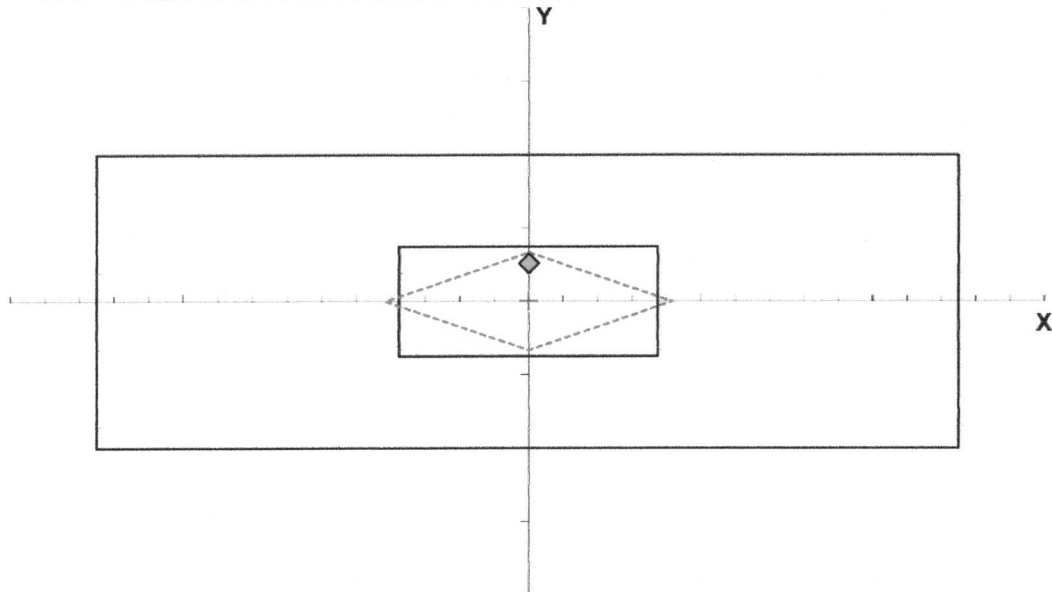
Pier/Loding Data:

Number of Piers = 1



Nomenclature

	Pier #1								
Xp (ft.) =	0.000								
Yp (ft.) =	0.000								
Lpx (ft.) =	1.500								
Lpy (ft.) =	1.500								
h (ft.) =	0.000								
Pz (k) =	-0.54								
Hx (k) =	0.00								
Hy (k) =	0.27								
Mx (ft-k) =	-2.87								
My (ft-k) =	0.00								



FOOTING PLAN

JOB NO. 62521508.01 SHEET 5 OF 15
 PHASE SCULPTURE TASK FOUNDATION DESIGN
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Results:

Total Resultant Load and Eccentricities:

$\Sigma P_z = -6.54$ kips
 $e_x = 0.00$
 $e_y = 0.52$ ft. ($\leq B/6$)

Overturning Check:

$\Sigma M_{rx} = 13.08$ ft-kips
 $\Sigma M_{oy} = -3.41$ ft-kips
 $FS(ot)x = 3.841$ (≥ 1.5)
 $\Sigma M_{ry} = N.A.$ ft-kips
 $\Sigma M_{ox} = N.A.$ ft-kips
 $FS(ot)y = N.A.$

Sliding Check:

Pass(x) = 0.00 kips
 Frict(x) = 2.29 kips
 $FS(slid)x = N.A.$
 Passive(y) = 0.00 kips
 Frict(y) = 2.29 kips
 $FS(slid)y = 8.478$ (≥ 1.5)

Uplift Check:

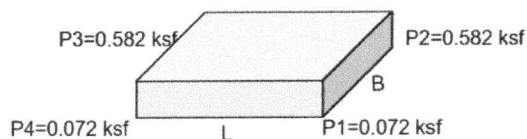
$\Sigma P_z(down) = -6.54$ kips
 $\Sigma P_z(uplift) = 0.00$ kips
 $FS(uplift) = N.A.$

Bearing Length and % Bearing Area:

Dist. x = N.A. ft.
 Dist. y = N.A. ft.
 Brg. Lx = 5.000 ft.
 Brg. Ly = 4.000 ft.
 %Brg. Area = 100.00 %
 Biaxial Case = N.A.

Gross Soil Bearing Corner Pressures:

$P_1 = 0.072$ ksf
 $P_2 = 0.582$ ksf
 $P_3 = 0.582$ ksf
 $P_4 = 0.072$ ksf



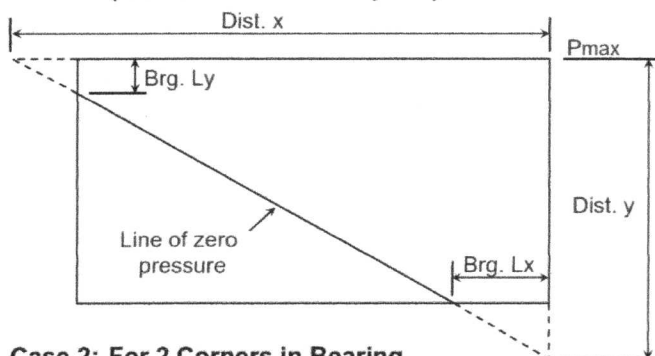
CORNER PRESSURES

Maximum Net Soil Pressure:

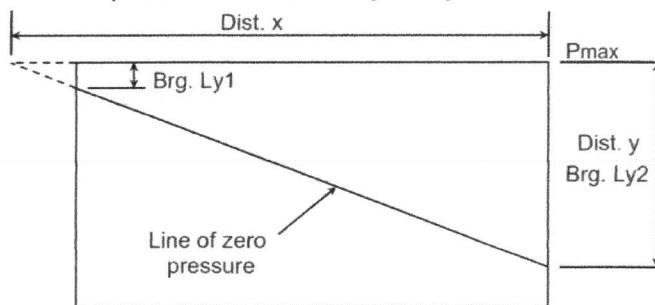
$P_{max(net)} = P_{max(gross)} - (D+T) \cdot \gamma_s$
 $P_{max(net)} = 0.582$ ksf

Nomenclature for Biaxial Eccentricity:

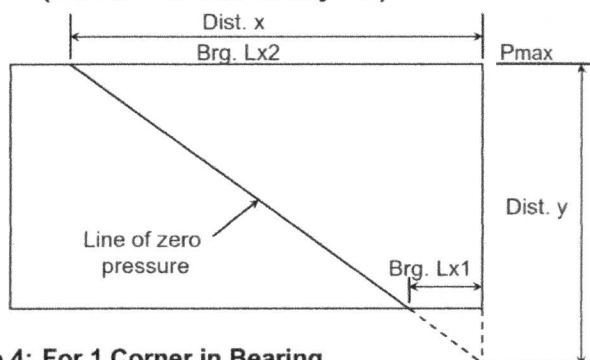
Case 1: For 3 Corners in Bearing (Dist. x > L and Dist. y > B)



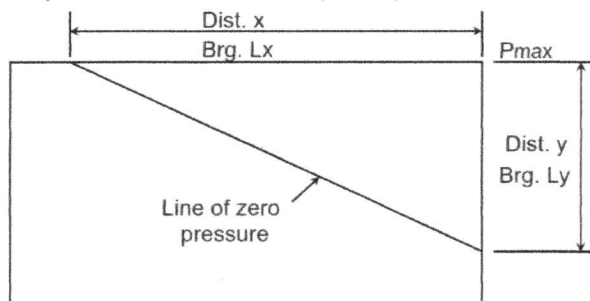
Case 2: For 2 Corners in Bearing (Dist. x > L and Dist. y \leq B)



Case 3: For 2 Corners in Bearing (Dist. x \leq L and Dist. y > B)



Case 4: For 1 Corner in Bearing (Dist. x \leq L and Dist. y \leq B)

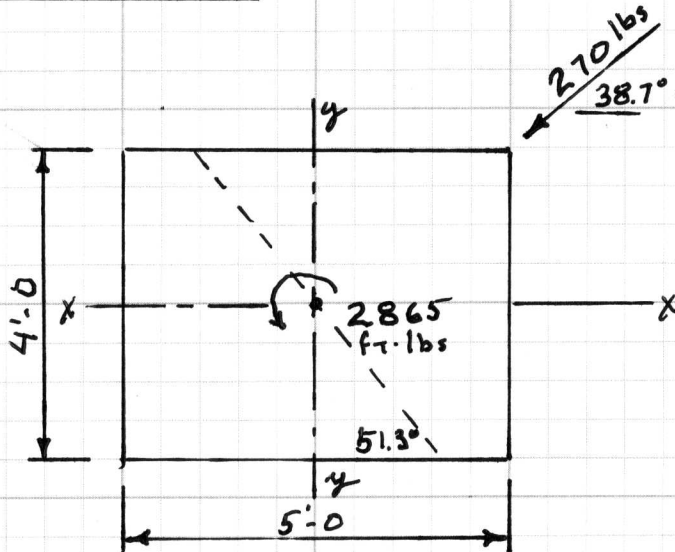


JOB NO. 62521508.01 SHEET 6 OF 15
 PHASE Sculpture TASK Fdn Design
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LOAD CASE 2



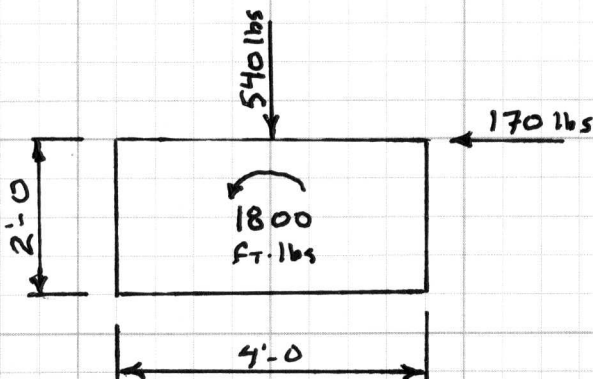
$$P \approx 540 \text{ lbs}$$

X-X Axis:

$$\begin{aligned} F_{\text{wind}(x)} &= F_w (\sin 38.7^\circ) \\ &= 270 (0.625) \\ &= 168.8 \text{ lbs} \\ &\approx 170 \text{ lbs} \end{aligned}$$

$$\begin{aligned} M_{\text{wind}(x)} &= M_w (\sin 38.7^\circ) \\ &= 2865 (0.625) \\ &= 1791 \text{ ft-lbs} \approx 1800 \text{ ft-lbs} \end{aligned}$$

X-X Axis

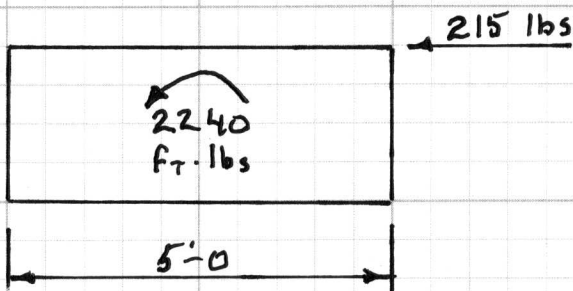


Y-Y Axis:

$$\begin{aligned} F_{\text{wind}(y)} &= F_w (\cos 38.7^\circ) \\ &= 270 (0.78) \\ &= 210.72 \approx 215 \text{ lbs} \end{aligned}$$

$$\begin{aligned} M_{\text{wind}(y)} &= M_w (\cos 38.7^\circ) \\ &= 2865 (0.78) \\ &= 2235.93 \approx 2240 \text{ ft-lbs} \end{aligned}$$

Y-Y Axis



Footing Weight:

$$\gamma_{\text{conc}} = 0.150 \text{ kcf}$$

$$(5' \times 4' \times 2') (0.15) = 6 \text{ K}$$

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 PHASE SCULPTURE TASK FOUNDATION DESIGN
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 BY QCB DATE 4/13/2017
 CHECKED BY DATE



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FROM ALEX TOMANOVICH'S "FOOTINGS" CALCULATION SPREADSHEET.

Input Data:

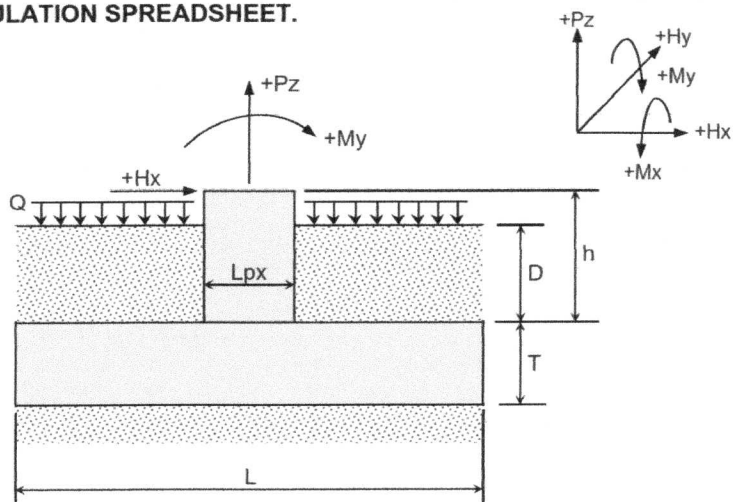
LOAD CASE 2:

Footing Data:

Footing Length, L = 5.000 ft.
 Footing Width, B = 4.000 ft.
 Footing Thickness, T = 2.000 ft.
 Concrete Unit Wt., γ_c = 0.150 kcf
 Soil Depth, D = 0.000 ft.
 Soil Unit Wt., γ_s = 0.000 kcf
 Pass. Press. Coef., K_p = 3.000
 Coef. of Base Friction, μ = 0.350
 Uniform Surcharge, Q = 0.000 ksf

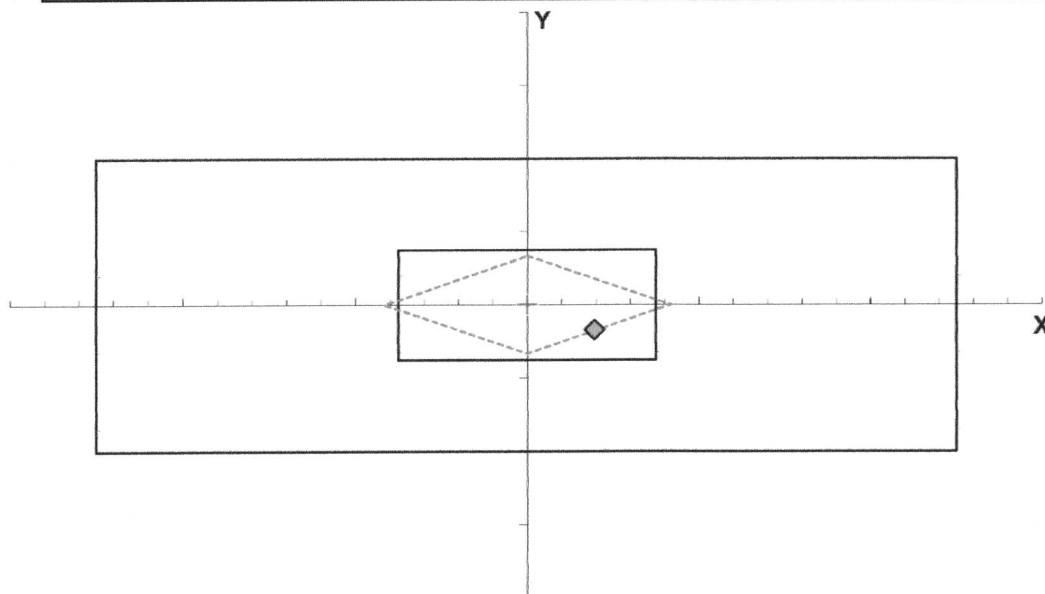
Pier/Loding Data:

Number of Piers = 1



Nomenclature

	Pier #1							
Xp (ft.) =	0.000							
Yp (ft.) =	0.000							
Lpx (ft.) =	1.500							
Lpy (ft.) =	1.500							
h (ft.) =	0.000							
Pz (k) =	-0.54							
Hx (k) =	0.17							
Hy (k) =	-0.22							
Mx (ft-k) =	1.80							
My (ft-k) =	2.24							



FOOTING PLAN

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 PHASE SCULPTURE TASK FOUNDATION DESIGN
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Results:

Total Resultant Load and Eccentricities:

$\Sigma Pz = -6.54$ kips
 $ex = 0.39$ ft. ($\leq L/6$)
 $ey = -0.34$ ft. ($\leq B/6$)

Overturning Check:

$\Sigma Mrx = 13.08$ ft-kips
 $\Sigma Mox = 2.23$ ft-kips
 $FS(ot)x = 5.865$ (≥ 1.5)
 $\Sigma Mry = 16.35$ ft-kips
 $\Sigma Moy = 2.58$ ft-kips
 $FS(ot)y = 6.337$ (≥ 1.5)

Sliding Check:

Pass(x) = 0.00 kips
 Frict(x) = 2.29 kips
 $FS(slid)x = 13.465$ (≥ 1.5)
 Passive(y) = 0.00 kips
 Frict(y) = 2.29 kips
 $FS(slid)y = 10.647$ (≥ 1.5)

Uplift Check:

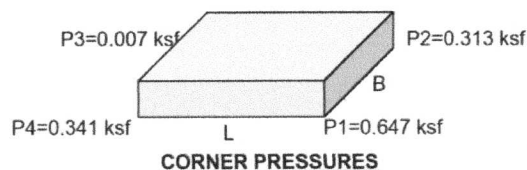
$\Sigma Pz(down) = -6.54$ kips
 $\Sigma Pz(uplift) = 0.00$ kips
 $FS(uplift) = N.A.$

Bearing Length and % Bearing Area:

Dist. x = N.A. ft.
 Dist. y = N.A. ft.
 Brg. Lx = 5.000 ft.
 Brg. Ly = 4.000 ft.
 %Brg. Area = 100.00 %
 Biaxial Case = N.A. $6*ex/L + 6*ey/B = 0.978$

Gross Soil Bearing Corner Pressures:

$P1 = 0.647$ ksf
 $P2 = 0.313$ ksf
 $P3 = 0.007$ ksf
 $P4 = 0.341$ ksf

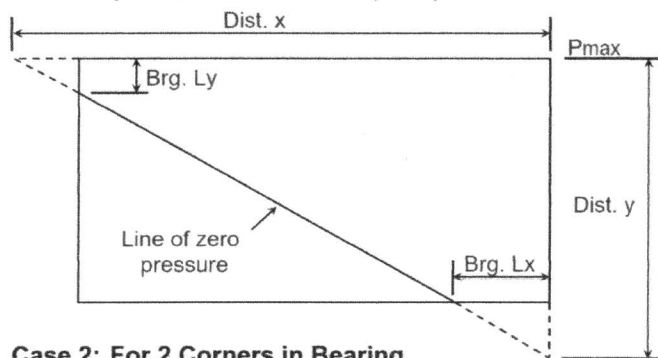


Maximum Net Soil Pressure:

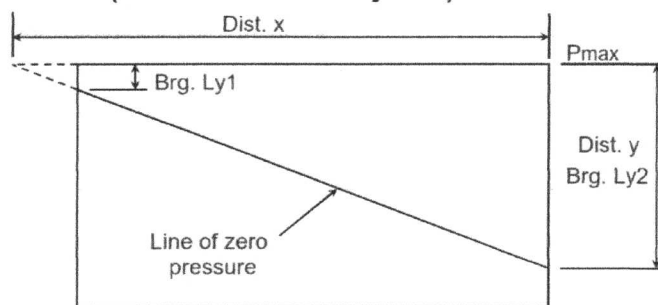
$P_{max(net)} = P_{max(gross)} - (D+T)*\gamma_s$
 $P_{max(net)} = 0.647$ ksf

Nomenclature for Biaxial Eccentricity:

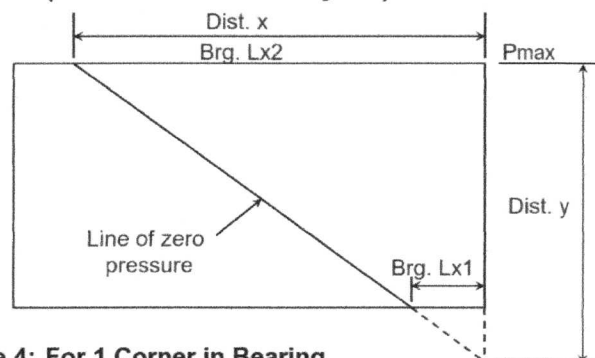
Case 1: For 3 Corners in Bearing (Dist. x > L and Dist. y > B)



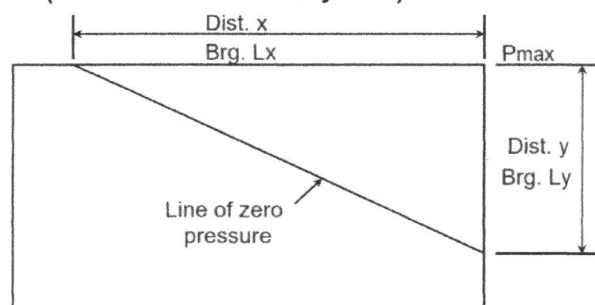
Case 2: For 2 Corners in Bearing (Dist. x > L and Dist. y ≤ B)



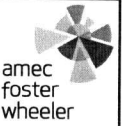
Case 3: For 2 Corners in Bearing (Dist. x ≤ L and Dist. y > B)



Case 4: For 1 Corner in Bearing (Dist. x ≤ L and Dist. y ≤ B)



JOB NO. 62521508.0 SHEET 9 OF 15
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LOAD CASE 2 IS WORST CASE.

USE MAXIMUM CORNER PRESSURES ACROSS FOOTING WIDTH (CONS.)



$$\text{AREA OF } F_{Tg} = 20 \text{ ft}^2$$

$$S = \frac{bh^2}{6} = \frac{4(5)^2}{6} = 16.67 \text{ ft}^3$$

Equivalent Beam Loading:

$$\text{Footing Wt} : (4'W)(2'd)(0.15 \text{ Kcf}) = 1.2 \text{ K/ft}$$

$$\text{MAX CORNER} : (4'W)(0.647 \text{ Ksf}) = 2.588 \text{ K/ft}$$

$$\text{MIN CORNER} : (4'W)(0.313 \text{ Ksf}) = 1.252 \text{ K/ft}$$

$$P_{\text{Tot}} = \frac{1}{2}(0.647 + 0.313)(4'W)(5'L) = 9.6 \text{ K}$$

$$P' = P_{\text{Tot}} - F_{Tg} \text{ Wt} = 9.6 \text{ K} - 6.0 \text{ K} = 3.6 \text{ K}$$

$$\text{MAX PRESSURE} = P/A + M/S$$

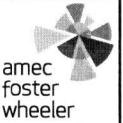
$$\text{MIN PRESSURE} = P/A - M/S$$

$$P/A = \frac{9.6 \text{ K}}{20 \text{ ft}^2} = 0.48 \text{ Ksf}$$

$$P/A + M/S = 0.647 \Rightarrow 0.48 \text{ Ksf} + \frac{M}{16.67} = 0.647 \text{ Ksf}$$

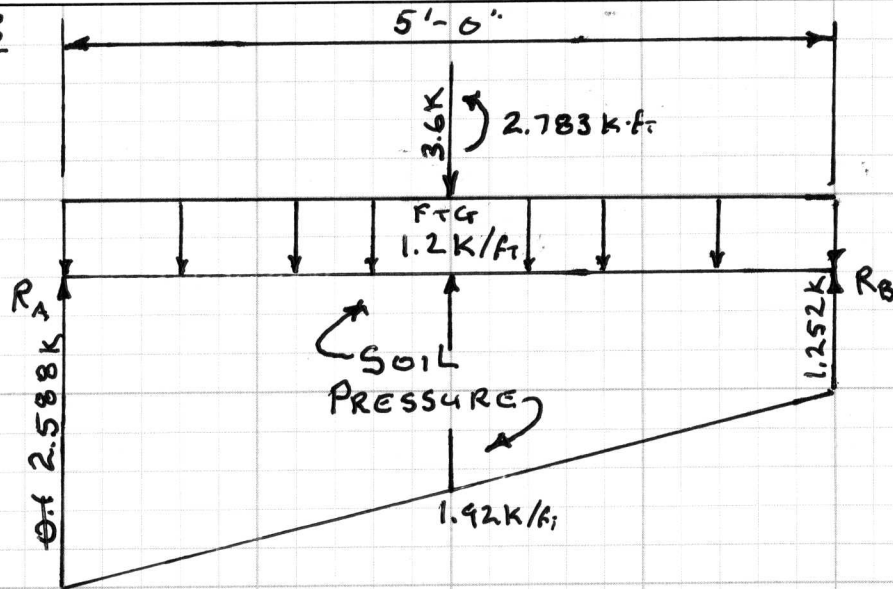
$$\frac{M}{S} = 0.167 \Rightarrow \frac{M}{16.67} = 0.167 \Rightarrow M = 2.783 \text{ K}\cdot\text{ft}$$

JOB NO. 62521508.01 SHEET 10 OF 15
 PHASE SCULPTURE TASK Fdn Design
 JOB NAME NC ARBORETUM
 BY ACB DATE 4/13/2017
 CHECKED BY _____ DATE _____

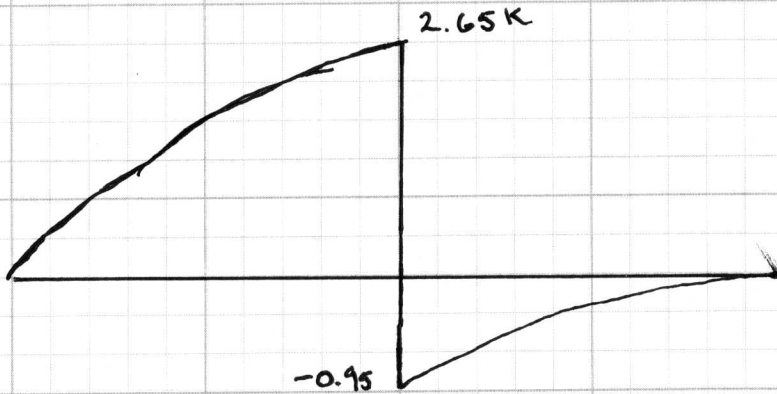


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GROSS:



SHEAR



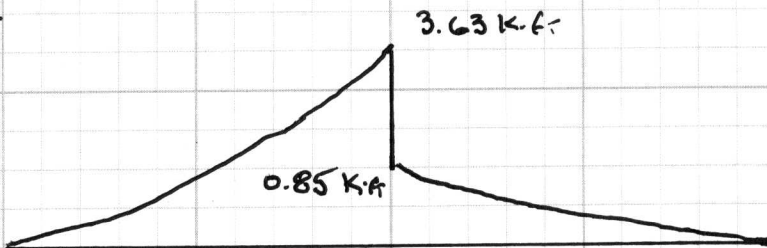
$$1.92k - 1.2k = 0.72k$$

$$0.72 (2.5) = 1.8k$$

$$\frac{1}{2} (2.5') (0.668) = 0.85$$

$$V@2.5' = 1.8 + 0.85 = 2.65k$$

MOMENT:



SHEAR Eq FROM X=0-2.5'

$$V = 1.338x - 0.1336x^2$$

MOMENT = AREA SHEAR
DIAGRAM

$$\int_0^{2.5} 1.338x - 0.1336x^2$$

$$= \frac{1.338x^2}{2} - \frac{0.1336x^3}{3}$$

$$= 0.694x^2 - 0.0453x^3 \Big|_0^{2.5}$$

JOB NO. 62521508.01 SHEET 11 OF 15
 PHASE Sculpture TASK Fdm Design
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MOMENT (CON'T)

$$M = 0.694(2.5)^2 - 0.0453(2.5)^3 - \cancel{(0.694(0)^2 - 0.0453(0)^2)}^{50}$$

$$= 4.338 - 0.708$$

$$= 3.63 \text{ K}\cdot\text{ft}$$

By OBSERVATION, 3.63 K·ft is MAX. MOMENT

Multiply by 1.5 to bring ASD MOMENT TO STRENGTH MOMENT

$$1.5 M = M_u$$

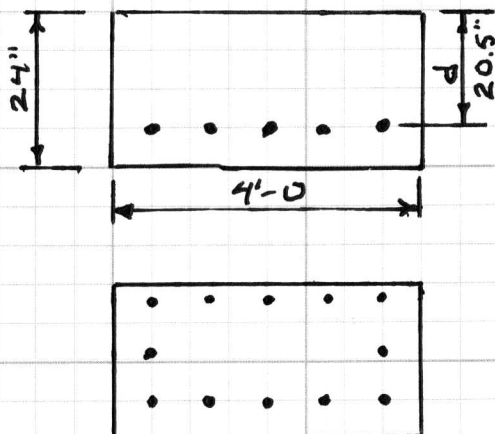
$$1.5 (3.63) = 5.45 \text{ K}\cdot\text{ft}$$

$$1.5 V = V_u$$

$$1.5 (2.65) = 4 \text{ K}$$

INPUT INTO ALEX TOMANAVICH'S "RECT BEAM" Spreadsheet

Try (5)-#5 BARS EQ SPACED IN SHORT DIRECTION



$$d = 24" - (3" \text{ CLR}) - \frac{1}{2} \left(\frac{5"}{8} \right) = 20.6875"$$

USE $d = 20.5"$ (slightly CONSERVATIVE)

$$A_s = (5)(0.31 \text{ in}^2) = 1.55 \text{ in}^2$$

$$\phi M_N = 140 \text{ K}\cdot\text{ft} > 5.5 \text{ K}\cdot\text{ft} \therefore \text{OK (PAGE 12)}$$

Provide (5)-#5s, T+B + (12)#5 EA FACE (12 TOTAL)

$$A_s (\text{TEMP}) = 0.0018 (48") (24") = 2.074 \text{ in}^2 \text{ REQ'd}$$

$$A_s (\text{PROVIDED}) = (12)(0.31 \text{ in}^2) = 3.72 \text{ in}^2 > 2.074 \text{ in}^2$$

$\therefore \text{OK}$

JOB NO. 62521508.01 SHEET 12 OF 15
 PHASE SCULPTURE TASK FOUNDATION DESIGN
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 BY ACB DATE 4/13/2017
 CHECKED BY DATE

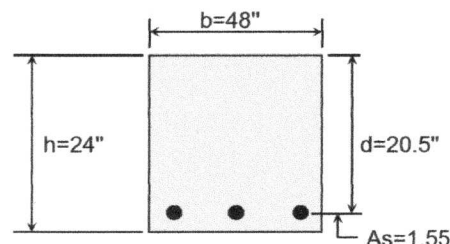


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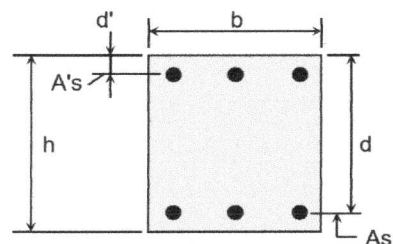
FROM ALEX TOMANOVICH'S "RECTBEAM (318-11)" CALCULATION SPREADSHEET.

Input Data:

Beam or Slab Section?	Slab	
Exterior or Interior Exposure?	Exterior	
Reinforcing Yield Strength, f_y =	60	ksi
Concrete Comp. Strength, f'_c =	3	ksi
Slab Section Width, b =	48.000	in.
Depth to Tension Reinforcing, d =	20.500	in.
Total Slab Section Depth, h =	24.000	in.
Tension Reinforcing, A_s =	1.550	in. ²
No. of Tension Bars in Slab, N_b =	5.000	
Tension Reinf. Bar Spacing, s_1 =	12.000	in.
Clear Cover to Tension Reinf., C_c =	3.000	in.
Depth to Compression Reinf., d' =	0.000	in.
Compression Reinforcing, A'_s =	0.000	in. ²
Working Stress Moment, M_a =	3.60	ft-kips
Ultimate Design Moment, M_u =	5.50	ft-kips
Ultimate Design Shear, V_u =	4.00	kips
Total Stirrup Area, $A_v(\text{stirrup})$ =	0.000	in. ²
Tie/Stirrup Spacing, s_2 =	0.0000	in.



Singly Reinforced Section



Doubly Reinforced Section

Results:

Moment Capacity Check for Slab-Type Section:

β_1 =	0.85	
c =	0.894	in.
a =	0.760	in.
ρ_b =	0.02138	
ρ =	0.00158	
$\rho(\text{min})$ =	N.A.	
$A_s(\text{min})$ =	N.A.	in. ²
$\rho(\text{temp})$ =	0.00180	(total for section)
$A_s(\text{temp})$ =	2.074	in. ² (total)
$\rho(\text{max})$ =	0.01548	
$A_s(\text{max})$ =	15.234	in. ² $\geq A_s = 1.55$ in. ² , O.K.
ϵ'_s =	N.A.	
f'_s =	N.A.	ksi
ϵ_t =	0.06580	≥ 0.005 , Tension-controlled
ϕ =	0.900	
ϕM_n =	140.34	ft-k $\geq M_u = 5.5$ ft-k, O.K.

Shear Capacity Check for Slab-Type Section:

ϕV_c =	80.84	kips $\geq V_u = 4$ kips, O.K.
ϕV_s =	N.A.	kips, $= \phi * f_y * d * A_v / s \leq \phi V_s(\text{max})$
ϕV_n =	N.A.	kips
$\phi V_s(\text{max})$ =	N.A.	kips
$A_v(\text{prov})$ =	N.A.	in. ² = $A_v(\text{stirrup})$
$A_v(\text{req'd})$ =	N.A.	in. ²
$A_v(\text{min})$ =	N.A.	in. ²
$s_2(\text{max})$ =	N.A.	in.

Crack Control (Distribution of Reinf.):

Per ACI 318-05 Code:

E_s =	29000	ksi
E_c =	3122	ksi
n =	9.29	
f_s =	1.43	ksi
$f_s(\text{used})$ =	1.43	ksi
$s_1(\text{max})$ =	334.57	in. $\geq s_1 = 12$ in., O.K.

Per ACI 318-95 Code (for reference only):

d_c =	3.5000	in.
z =	8.86	k/in.
$z(\text{allow})$ =	129.00	k/in. $\geq z = 8.86$ k/in., O.K.

(Note: this worksheet excludes sections that qualify as "Deep Beams" per ACI 318-11 Code, Sections 10.7 & 11.7.)

Moment of Inertia for Deflection:

f_r =	0.411	ksi
k_d =	3.2197	in.
I_g =	55296.00	in. ⁴
M_{cr} =	157.74	ft-k
I_{cr} =	4833.32	in. ⁴
I_e =	55296.00	in. ⁴ (for deflection)

JOB NO. 62521508.01 SHEET 13 OF 15
 PHASE Sculpture TASK Fdn Design
 JOB NAME NC ARBORETUM
 BY ACB DATE 4/18/2017
 CHECKED BY _____ DATE _____



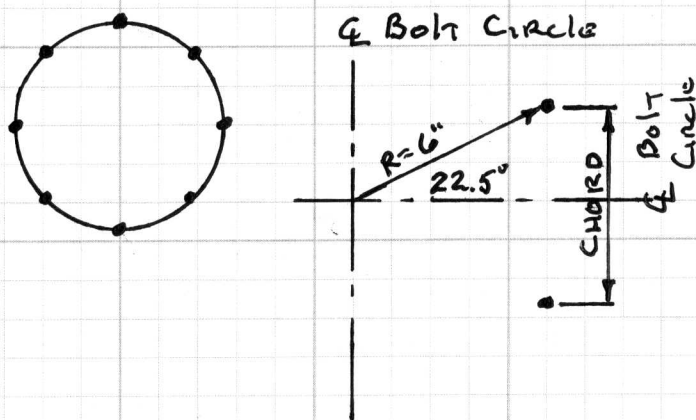
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Anchor Rod Design:

Assume 12" ϕ Bolt Circle Pattern

Assume 8 Anchor Rods \Rightarrow Angle between Rods = 45°

Min. Distance Between Bolts = 4" (Chord distance)



Chord Distance:

$$(2)(\sin 22.5^\circ) = \frac{x}{6}$$

$$x = (6)(2)(\sin 22.5^\circ)$$

$$= 4.59" > 4"$$

\therefore 12" ϕ Bolt Circle OK

Assume $3/4"$ ϕ Anchor Rods

Assume 8" Projection of Rods

$$\text{Moment of Inertia of Bolt Group } (I_b) = (2)(6")^2 + (4)((6)(\sin 22.5^\circ))^2 + 2(0)^2$$

$$I_b = 144 \text{ in}^2$$

$$M = 2.865 \text{ K}\cdot\text{ft} \text{ (Page 3)}$$

$$\text{Max Tension in Bolt} = \frac{M_c}{I_b} = \frac{2.865 \text{ K}\cdot\text{ft} \left(\frac{12"}{\text{ft}} \right) (6")}{144 \text{ in}^2} = 1.43 \text{ K}$$

$$V = 0.27 \text{ K}$$

$$V_b = V/\text{Bolt} = V/N_b \text{ Where } N_b = \text{Number of Bolts} \Rightarrow V_b = \frac{0.27}{8} = 0.034 \text{ K/Bolt}$$

$3/4"$ ϕ Anchor Rod OK By Engineering Judgment

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BASE PLATE Design:

MATERIAL = ASTM A36 (MIN.) ($F_y = 36 \text{ ksi}$)

BASE PLATE IS UNDERGROUND \therefore Apply CORROSION ALLOWANCE

CORROSION ALLOWANCE = $\frac{1}{16}$ " EACH FACE \therefore add $(2)(\frac{1}{16})$ OR $\frac{1}{8}$ " TO design

Typical Allowable STRESS for plate = $0.75 F_y$, HOWEVER, Loading will be cyclic, but NOT EXTREME cyclic \therefore Reduce by $\frac{1}{3}$

$$\begin{aligned}\sigma_{\text{ALLOW}} &= (\frac{2}{3})(0.75)(F_y) \\ &= (\frac{2}{3})(0.75)(36) \\ &= 18 \text{ ksi}\end{aligned}$$

Edge distance of Bolts = $1\frac{1}{2}$ "

$$\begin{aligned}\text{PLATE Width} &= (\text{Bolt Circle}) + (2)(1\frac{1}{2}) \\ &= (12") + (3") \\ &= 15"\end{aligned}$$

$$M = 2.865 \text{ K}\cdot\text{ft} \quad (\text{Page 3})$$

$$\sigma = M/S$$

$$S = \frac{b^3}{6} = \frac{15^3}{6} = 2.5 \text{ ft}^3$$

$$\frac{2.865 \text{ K}\cdot\text{ft} (\frac{12"}{\text{ft}})}{2.5 \text{ ft}^3} \leq 18 \text{ ksi}$$

$$t^3 \geq \frac{34.38 \text{ K}\cdot\text{in}}{2.5 \text{ in} (18 \text{ K/in}^2)}$$

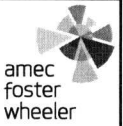
$$t^3 \geq 0.764 \text{ in}^3$$

$$t \geq \sqrt[3]{0.764 \text{ in}^3} = 0.914"$$

Add CORROSION ALLOWANCE: $0.914" + 0.125" = 1"$

USE 1" THICK PLATE

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Calculate Required Weld Size:

Sculpture Support Post: 4" STD Pipe (By ARBORETUM)
 $F_y = 35 \text{ KSI}$
 Outside diameter = 4 1/2"

PER Blodgett, "Design of Welded Structures":

$$I_w = \pi r^3$$

$$r = 2.25"$$

$$I_w = \pi (2.25")^3$$

$$= \pi (11.39 \text{ in}^3) = 35.785 \text{ in}^3$$

$$\text{CIRCUMFERENCE} = \pi d = \pi (4.5") = 14.14"$$

SHEAR STRESS ON WELD = $\frac{V}{L}$, WHERE V = TOTAL SHEAR + L = LENGTH WELD

$$V = 0.27 \text{ K (Page 3)}$$

$$\sigma_v = \frac{0.27 \text{ K}}{35.78} \frac{0.27 \text{ K}}{14.14 \text{ in}} = 0.02 \text{ K/in}$$

STRESS due TO MOMENT = $\frac{M_c}{I_w}$, WHERE M = TOTAL MOMENT, C = EXTREME DIST.
 + I_w = MOMENT OF INERTIA OF WELD

$$M = 2.865 \text{ K} \cdot \text{ft (Page 3)}$$

$$c = \frac{1}{2} d = \frac{1}{2} (4.5") = 2.25"$$

$$I_w = 35.785 \text{ in}^3 \text{ (SEE ABOVE)}$$

$$\sigma_m = \frac{2.865 \text{ K} \cdot \text{ft} \left(\frac{12"}{\text{ft}} \right) (2.25")}{35.785 \text{ in}^3} = 2.16 \text{ K/in}$$

$$\sigma_{\text{RES.}} = \sqrt{\sigma_v^2 + \sigma_m^2} = \sqrt{(0.02)^2 + (2.16)^2} \approx 2.16 \text{ K/in}$$

By observation, USE MIN. WELD SIZE OF 3/16" (WELD STRENGTH = 2.78 K/in)

CHECK STRENGTH OF POST: $(0.6)(F_y)(\text{THICKNESS}) = 0.6(35 \text{ KSI})(0.237") = 4.98 \text{ K/in}$

$4.98 \text{ K/in} > 2.78 \text{ K/in} \therefore \text{OK (WELD CONTROLS)}$