
FT CUSTER NATIONAL CEMETERY

Construct 2,500 Niche Columbarium

BID SET STRUCTURAL DESIGN CALCULATIONS

Project 923 CM 3026

Structural Engineer:

KCI Technologies, Inc

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KCI Project No: 28133363.09



March 4, 2015

Table of Contents
Structural Calculations

<u>Item</u>	<u>Page</u>
Design Criteria	1
Foundation Design	7
Concrete Pier Design	20



Design Criteria per IBC

PROJECT Ft Custer - Columbarium
 KCI # 28133363.09
 DESIGNED BY LC DATE 10-27-2014
 CHECKED BY _____ DATE _____

SCOPE:

Determine wind, snow, seismic, and live loads applicable to the project.

REFERENCE CODES/TECHNICAL DATA:

Per International Building Code (IBC 2012) with reference to ASCE 7 (10) "Design Loads for Buildings and Other Structures".

SITE/BUILDING INFORMATION:

OCCUPANCY CATEGORY: I (per code) **USE: II (increased design)**
 WIND EXPOSURE: C
 LOCATION: Augusta, MI 49012

SNOW LOADS:

ASCE 7 (Figure 7-1): Augusta, MI	$P_g := 35 \text{psf}$	Ground snow load
ASCE 7 (Table 7-4): Occupancy II	$I_s := 1.0$	Snow importance factor
ASCE 7 (Table 7-2): Exp C, Fully Exposed	$C_e := 0.9$	Exposure factor
ASCE 7 (Table 7-3): Unheated	$C_t := 1.2$	Thermal factor
ASCE 7 Section 7.3.4:	$P_{f2} := 20 \text{psf}$	Minimum flat roof load

$$P_{f1} := 0.7 P_g \cdot C_e \cdot C_t \cdot I_s = 26.46 \cdot \text{psf} \quad \text{Calculated flat roof snow load}$$

$$P_f := \max(P_{f1}, P_{f2}) = 26.46 \cdot \text{psf} \quad \text{FLAT ROOF SNOW LOAD}$$

WIND LOAD CRITERIA:

ASCE 7 (Figure 26.5-1 A): Augusta, MI	BASIC_WIND_SPEED := 115mph
Importance Factor: Occupancy Category II	
Site Parameters:	EXPOSURE = C

DESIGN PARAMETERS:

$$V_{ww} := \frac{\text{BASIC_WIND_SPEED}}{\text{mph}} = 115$$

$$\text{height_above_grade} := 8.25\text{ft}$$

$$h := \frac{\text{height_above_grade}}{\text{ft}} = 8.25$$

$$\beta := 0.01 \quad (\text{dampening ratio})$$

$$z_- := \frac{\text{height_above_grade}}{\text{ft}}$$

Table 29.3-1 Parameters:

$$\alpha := 9.5$$

$$b_- := 0.65$$

$$z_g := 900$$

$$c_{ww} := 0.20$$

$$a_{\Delta} := \frac{1}{9.5}$$

$$l_{ww} := 500$$

$$b_{\Delta} := 1.00$$

$$\epsilon_- := \frac{1}{5.0}$$

$$\alpha_- := \frac{1}{6.5}$$

$$z_{\min} := 15\text{ft}$$

CHAPTER 29: Design Wind Load on "Other Structures"1. VELOCITY PRESSURE (q_z):

$$q_z = 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \quad (\text{Eq 29.3-1})$$

$$\text{velocity pressure coefficient } K_z: \quad z := \text{if}(h \leq 15, 15, h) = 15$$

$$K_z := 2.01 \cdot \left(\frac{z}{z_g} \right)^{\frac{2}{\alpha}} = 0.849$$

$$\text{topographic coefficient } K_{zt}: \quad K_{zt} := 1.0 \quad (\text{no hills or escarpments})$$

$$\text{directionality coefficient } K_d: \quad K_d := 0.85 \quad (\text{Table 26.6-1})$$

$$q_z := 0.00256 \text{psf} \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 = 24.429 \cdot \text{psf}$$

2. GUST EFFECT FACTOR (G/G_f):Note: If not flexible: $G_f = G = 0.85$ for both directions!!

6.5.8.2 - Flexible or dynamically sensitive structure per Section 26.9.4 is building or other structure with natural frequency less than 1 Hz. STRUCTURE IS NOT FLEXIBLE.

$$G_f := 0.85$$

3. FORCE COEFFICIENT (C_f):

Solid Freestanding Walls

(Figure 29.4-1)

$$\text{swall} := 8.25\text{ft}$$

$$B_{\text{wall}} := 61.25\text{ft}$$

$$A_s := \text{swall} \cdot B_{\text{wall}} = 505.31 \text{ft}^2$$

$$\text{clearance_ratio} := \frac{\text{swall}}{\text{height_above_grade}} = 1$$

$$\text{aspect_ratio} := \frac{B_{\text{wall}}}{\text{swall}} = 7.424$$

CASE A:

$$C_f_CASEA := 1.325$$

4. DESIGN WIND LOADS (F):

$$F = q_h \cdot G \cdot C_f \cdot A_s \quad (\text{Eq 29.4-1})$$

CASE A: resultant force acts normal to the face of the sign a distance above the geometric center equal to 0.05 times the average height of the sign.

$$F_A := q_z \cdot G_f \cdot C_f_CASEA \cdot A_s = 13903 \text{ lbf}$$

CHAPTER 29 WIND LOADS ON OTHER STRUCTURES AND BUILDING ATTACHMENTS—MWFRS

Velocity Pressure Exposure Coefficients, K_z , and K_{zt}

Table 29.3-1

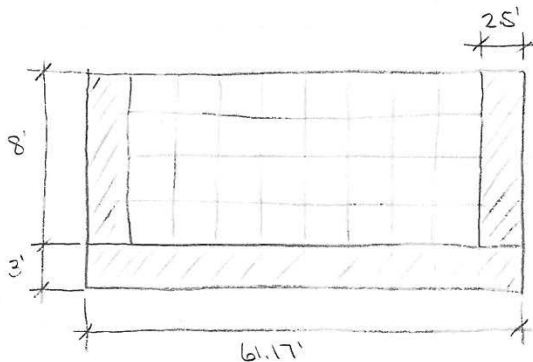
Height above ground level, z		Exposure		
		B	C	D
H	(m)			
0-15	(0-4.6)	0.57	0.85	1.03
20	(6.1)	0.62	0.90	1.08
25	(7.6)	0.66	0.94	1.12
30	(9.1)	0.70	0.98	1.16
40	(12.2)	0.76	1.04	1.22
50	(15.2)	0.81	1.09	1.27
60	(18)	0.85	1.13	1.31
70	(21.3)	0.89	1.17	1.34
80	(24.4)	0.93	1.21	1.38
90	(27.4)	0.96	1.24	1.40
100	(30.5)	0.99	1.26	1.43
120	(36.6)	1.04	1.31	1.48
140	(42.7)	1.09	1.36	1.52
160	(48.8)	1.13	1.39	1.55
180	(54.9)	1.17	1.43	1.58
200	(61.0)	1.20	1.46	1.61
250	(76.2)	1.28	1.53	1.68
300	(91.4)	1.35	1.59	1.73
350	(106.7)	1.41	1.64	1.78
400	(121.9)	1.47	1.69	1.82
450	(137.2)	1.52	1.73	1.86
500	(152.4)	1.56	1.77	1.89

Notes:

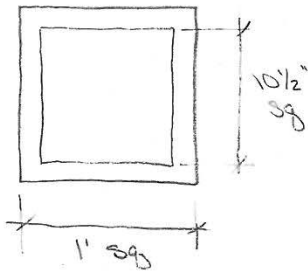
- The velocity pressure exposure coefficient K_z may be determined from the following formula:
 For $15 \text{ ft.} \leq z \leq z_g$ For $z < 15 \text{ ft.}$
 $K_z = 2.01 (z/z_g)^{2.6}$ $K_z = 2.01 (15/z_g)^{2.6}$
- α and z_g are tabulated in Table 26.9.1.
- Linear interpolation for intermediate values of height z is acceptable.
- Exposure categories are defined in Section 26.7.

SUBJECT FT CLUSTER COLUMBARIUMJOB NUMBER 28133263.09 SHEET 1 OF 3DESIGN LC DATE 10.27.14CHECK HDB DATE 10/28/14

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DESIGN DEAD LOADS

$$\begin{aligned}
 \text{DL Concrete} &= (3' \times 61.17' \times 3.83') + 2(8' \times 2.5' \times 3.83') \\
 &= 702.84 + 2(76.6) \\
 &= 856.04 \text{ ft}^3 \times 150 \text{ pcf} \\
 &= 128,406 \text{ lbs}
 \end{aligned}$$



$$\begin{aligned}
 \text{DL Precast Cubes} &= (1' \times 1' \times 3.83') - (10.5'' \times 10.5'' - 3.67') \\
 &= 3.83 - 2.81 \\
 &= 1.03 \text{ pcf} \times (8' \times 61.17') \times 150 \text{ pcf} \\
 &= 69,426 \text{ lbs}
 \end{aligned}$$

$$\begin{aligned}
 \text{DL Contents} &= (8' \times 61.17') \times 250 \text{ pcf} \\
 &= 112,340 \text{ lbs}
 \end{aligned}$$

$$\begin{aligned}
 \text{DL FULL} &= 128.4 \text{ k} + 112.3 \text{ k} \\
 &= 240.7 \text{ kips} \\
 &= \frac{240.7 \text{ kips}}{61.17 \text{ ft}} = 3.93 \text{ k/ft} \checkmark
 \end{aligned}$$

$$\begin{aligned}
 \text{DL EMPTY} &= 128.4 \text{ k} + 69.4 \text{ k} \\
 &= 197.8 \text{ kips} \\
 &= \frac{197.8 \text{ kips}}{61.17 \text{ ft}} = 3.23 \text{ k/ft} \checkmark
 \end{aligned}$$

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↳ TEDS DESIGN Output



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Section

Job Ref. Page 7 of 21
28133363.09

Sheet no./rev.

1

Calc. by
LC

Date
12/18/2014

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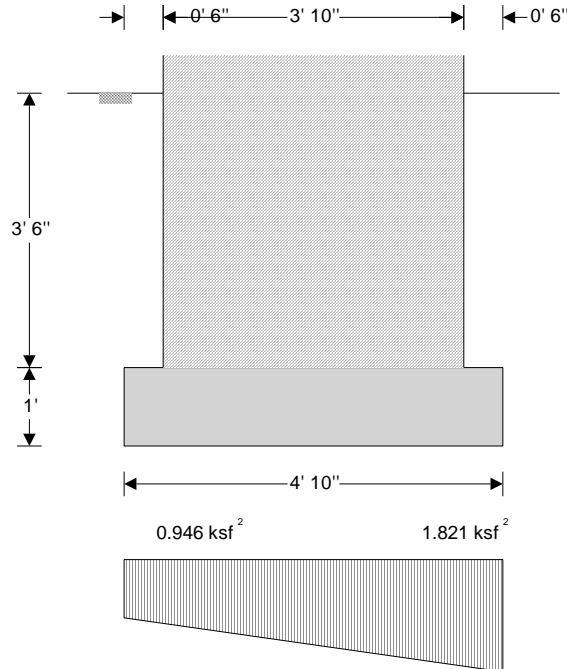
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STRIP FOOTING ANALYSIS AND DESIGN (ACI318-11)

TEDDS calculation version 2.0.05.00



Strip footing details

Width of strip footing

$B = 4.830$ ft

Depth of strip footing

$h = 12.000$ in

Depth of soil over strip footing

$h_{\text{soil}} = 42.000$ in

Density of concrete

$\rho_{\text{conc}} = 150.0$ lb/ft³

Load details

Load width

$b = 46.000$ in

Load eccentricity

$e_P = 0.000$ in

Soil details

Density of soil

$\rho_{\text{soil}} = 120.0$ lb/ft³

Angle of internal friction

$\phi' = 25.0$ deg

Design base friction angle

$\delta = 19.3$ deg

Coefficient of base friction

$\tan(\delta) = 0.350$

Allowable bearing pressure

$P_{\text{bearing}} = 3.000$ ksf

Axial loading on strip footing

Dead axial load

$P_G = 3.930$ kips/ft

Live axial load

$P_Q = 0.000$ kips/ft

Wind axial load

$P_W = 0.000$ kips/ft

Total axial load

$P = 3.930$ kips/ft

Foundation loads

Dead surcharge load

$F_{G_{\text{sur}}} = 0.000$ ksf



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Project Ft. Custer Columbarium				Job Ref. Page 8 of 21 28133363.09	
Section				Sheet no./rev. 2	
Calc. by LC	Date 12/18/2014	Chk'd by	Date	App'd by	Date

Live surcharge load

$$F_{Qsur} = 0.000 \text{ ksf}$$

Strip footing self weight

$$F_{swt} = h \times \rho_{conc} = 0.150 \text{ ksf}$$

Soil self weight

$$F_{soil} = h_{soil} \times \rho_{soil} = 0.420 \text{ ksf}$$

Total foundation load

$$F = B \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil}) = 2.753 \text{ kips/ft}$$

Moment on strip footing

Dead moment

$$M_G = 0.000 \text{ kip_ft/ft}$$

Live moment

$$M_Q = 0.000 \text{ kip_ft/ft}$$

Wind moment

$$M_W = 1.700 \text{ kip_ft/ft}$$

Total moment

$$M = 1.700 \text{ kip_ft/ft}$$

Check stability against overturning

Total overturning moment

$$M_{OT} = M + H \times h = 1.700 \text{ kip_ft/ft}$$

Restoring moment

Foundation loading

$$M_{sur} = B^2 \times (F_{Gsur} + F_{swt} + F_{soil}) / 2 = 6.649 \text{ kip_ft/ft}$$

Axial loading on column

$$M_{axial} = (P_G) \times (B / 2 - e_P) = 9.491 \text{ kip_ft/ft}$$

Total restoring moment

$$M_{res} = M_{sur} + M_{axial} = 16.140 \text{ kip_ft/ft}$$

PASS - Restoring moment is greater than overturning moment

Calculate base reaction

Total base reaction

$$T = F + P = 6.683 \text{ kips/ft}$$

Eccentricity of base reaction in x

$$e_T = (P \times e_P + M + H \times h) / T = 3.052 \text{ in}$$

Base reaction acts within middle third of base

Calculate base pressures

$$q_1 = (T / B) \times (1 - 6 \times e_T / B) = 0.946 \text{ ksf}$$

$$q_2 = (T / B) \times (1 + 6 \times e_T / B) = 1.821 \text{ ksf}$$

Minimum base pressure

$$q_{min} = \min(q_1, q_2) = 0.946 \text{ ksf}$$

Maximum base pressure

$$q_{max} = \max(q_1, q_2) = 1.821 \text{ ksf}$$

PASS - Maximum base pressure is less than allowable bearing pressure

Partial safety factors for loads

Partial safety factor for dead loads

$$\gamma_{FG} = 0.90$$

Partial safety factor for imposed loads

$$\gamma_{FQ} = 0.00$$

Partial safety factor for wind loads

$$\gamma_{FW} = 1.00$$

Strength reduction factors

Flexural strength reduction factor

$$\phi_f = 0.90$$

Shear strength reduction factor

$$\phi_s = 0.75$$

Ultimate axial loading

Ultimate axial loading

$$P_u = P_G \times \gamma_{FG} + P_Q \times \gamma_{FQ} + P_W \times \gamma_{FW} = 3.537 \text{ kips/ft}$$

Ultimate foundation loading

Ultimate foundation loading

$$F_u = B \times [(F_{Gsur} + F_{swt} + F_{soil}) \times \gamma_{FG} + F_{Qsur} \times \gamma_{FQ}] = 2.478 \text{ kips/ft}$$

Ultimate horizontal loading

Ultimate horizontal loading

$$H_u = H_G \times \gamma_{FG} + H_Q \times \gamma_{FQ} + H_W \times \gamma_{FW} = 0.000 \text{ kips/ft}$$

Ultimate moment on foundation

Ultimate moment

$$M_u = M_G \times \gamma_{FG} + M_Q \times \gamma_{FQ} + M_W \times \gamma_{FW} = 1.700 \text{ kip_ft/ft}$$



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Project Ft. Custer Columbarium				Job Ref. Page 9 of 21 28133363.09	
Section				Sheet no./rev. 3	
Calc. by LC	Date 12/18/2014	Chk'd by	Date	App'd by	Date

Calculate ultimate base reaction

Ultimate base reaction

$$T_u = F_u + P_u = \mathbf{6.015 \text{ kips/ft}}$$

Eccentricity of ultimate base reaction

$$e_{Tu} = (P_u \times e_P + M_u + H_u \times h) / T_u = \mathbf{3.392 \text{ in}}$$

Calculate ultimate pad base pressures

$$q_{1u} = (T_u / B) \times (1 - 6 \times e_{Tu} / B) = \mathbf{0.808 \text{ ksf}}$$

$$q_{2u} = (T_u / B) \times (1 + 6 \times e_{Tu} / B) = \mathbf{1.683 \text{ ksf}}$$

Minimum ultimate base pressure

$$q_{\min u} = \min(q_{1u}, q_{2u}) = \mathbf{0.808 \text{ ksf}}$$

Maximum ultimate base pressure

$$q_{\max u} = \max(q_{1u}, q_{2u}) = \mathbf{1.683 \text{ ksf}}$$

Calculate base lengths

Left hand length

$$B_L = B / 2 + e_P = \mathbf{28.980 \text{ in}}$$

Right hand length

$$B_R = B / 2 - e_P = \mathbf{28.980 \text{ in}}$$

Calculate rate of change of base pressure

Length of base reaction

$$B_x = B = \mathbf{57.960 \text{ in}}$$

Rate of change of base pressure

$$C_x = (q_{2u} - q_{1u}) / B_x = \mathbf{0.181 \text{ ksf/ft}}$$

Calculate ultimate moment

Ultimate moment

$$M_x = (q_{1u} - F_u / B) \times B_L^2 / 2 + C_x \times B_L^3 / 6 + M_u = \mathbf{2.985 \text{ kip_ft/ft}}$$

Material details

Compressive strength of concrete

$$f'_c = \mathbf{3000 \text{ psi}}$$

Yield strength of reinforcement

$$f_y = \mathbf{60000 \text{ psi}}$$

Cover to reinforcement

$$C_{nom} = \mathbf{1.500 \text{ in}}$$

Concrete type

$$\text{Normal weight}$$

Concrete modification factor

$$\lambda = \mathbf{1.00}$$

Moment design in x direction

Tension reinforcement bar designation

$$\mathbf{5}$$

Diameter of tension reinforcement

$$\phi_B = \mathbf{0.625 \text{ in}}$$

Spacing of tension bars

$$S_B = \mathbf{12.000 \text{ in}}$$

Tension reinforcement provided

$$\mathbf{\text{No. 5 bars @ 12.000 in centres}}$$

Depth of tension reinforcement

$$d = h - C_{nom} - \phi_B / 2 = \mathbf{10.187 \text{ in}}$$

Area of tension reinforcement provided

$$A_{s_B_prov} = \pi \times \phi_B^2 / (4 \times S_B) = \mathbf{0.307 \text{ in}^2/\text{ft}}$$

Minimum area of tension reinforcement

$$A_{s_x_min} = 0.0018 \times h = \mathbf{0.259 \text{ in}^2/\text{ft}}$$

Maximum spacing of reinforcement

$$S_{\max} = \min(3 \times h, 18\text{in}) = \mathbf{18.000 \text{ in}}$$

PASS - Tension reinforcement provided exceeds minimum tension reinforcement required

Depth of compression block

$$a_x = A_{s_B_prov} \times f_y / (0.85 \times f'_c) = \mathbf{0.60 \text{ in}}$$

Neutral axis factor

$$\beta_1 = \mathbf{0.85}$$

Depth to the neutral axis

$$C_{na_x} = a_x / \beta_1 = \mathbf{0.71 \text{ in}}$$

Strain in reinforcement

$$\epsilon_{t_x} = 0.003 \times (d - C_{na_x}) / C_{na_x} = \mathbf{0.04018}$$

PASS - The section has adequate ductility (cl. 10.3.5)

Nominal moment strength

$$M_{nx} = M_x / \phi_f = \mathbf{3.317 \text{ kip_ft/ft}}$$

Moment capacity of base

$$M_{capx} = A_{s_B_prov} \times f_y \times [d - (A_{s_B_prov} \times f_y / (1.7 \times f'_c))] = \mathbf{15.166 \text{ kip_ft/ft}}$$

PASS - Moment capacity of base exceeds nominal moment strength required

Calculate ultimate shear force at face of load

Ultimate shear force at face of load

$$V_{su} = (q_{2u} - F_u / B) \times (B_R - b / 2 - d) - C_x \times (B_R - b / 2 - d)^2 / 2 = \mathbf{-0.421 \text{ kips/ft}}$$



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Project
Ft. Custer Columbarium

Section

Calc. by
LC

Date
12/18/2014

Chk'd by

Date

Job Ref. Page 10 of 21
28133363.09

Sheet no./rev.
4

App'd by

Date

Shear design at distance d from face of load

Strength reduction factor in shear

$$\phi_s = 0.75$$

Nominal shear strength

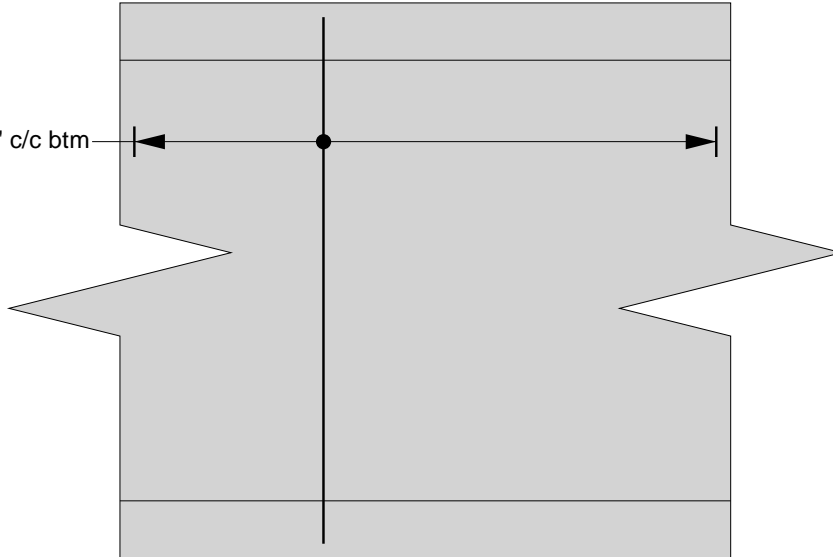
$$V_{nsu} = V_{su} / \phi_s = -0.562 \text{ kips/ft}$$

Concrete shear strength

$$V_c = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times d = 13.392 \text{ kips/ft}$$

PASS - Nominal shear strength is less than concrete shear strength

No. 5 bars @ 12" c/c btm





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Ft. Custer Columbarium

Section
Foundation Wall Design

Calc. by
LC

Date
12/17/2014

Chk'd by

Date

Job Ref. Page 11 of 21
28133363.09

Sheet no./rev.
1

App'd by

Date

RETAINING WALL ANALYSIS

In accordance with IBC 2012

Tedds calculation version 2.5.00

Retaining wall details

Stem type	Cantilever
Stem height	$h_{\text{stem}} = 3.083 \text{ ft}$
Stem thickness	$t_{\text{stem}} = 46 \text{ in}$
Angle to rear face of stem	$\alpha = 90 \text{ deg}$
Stem density	$\gamma_{\text{stem}} = 150 \text{ pcf}$
Toe length	$l_{\text{toe}} = 0.5 \text{ ft}$
Heel length	$l_{\text{heel}} = 0.5 \text{ ft}$
Base thickness	$t_{\text{base}} = 12 \text{ in}$
Base density	$\gamma_{\text{base}} = 150 \text{ pcf}$
Height of retained soil	$h_{\text{ret}} = 0.083 \text{ ft}$
Angle of soil surface	$\beta = 0 \text{ deg}$
Depth of cover	$d_{\text{cover}} = 3 \text{ ft}$

Retained soil properties

Soil type	Medium dense well graded sand
Moist density	$\gamma_{\text{mr}} = 125 \text{ pcf}$
Saturated density	$\gamma_{\text{sr}} = 137 \text{ pcf}$
Effective angle of internal resistance	$\phi_r = 30 \text{ deg}$
Effective wall friction angle	$\delta_r = 15 \text{ deg}$

Base soil properties

Soil type	Medium dense well graded sand
Moist density	$\gamma_{\text{mb}} = 115 \text{ pcf}$
Cohesion	$c_b = 0 \text{ psf}$
Effective angle of internal resistance	$\phi_b = 30 \text{ deg}$
Effective wall friction angle	$\delta_b = 15 \text{ deg}$
Effective base friction angle	$\delta_{bb} = 30 \text{ deg}$
Allowable bearing pressure	$P_{\text{bearing}} = 2000 \text{ psf}$

Loading details

Vertical line load at 2.42 ft	$P_{D1} = 3230 \text{ plf}$
Horizontal line load at 7.4 ft	$P_{L2} = 227 \text{ plf}$



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Ft. Custer Columbarium

Section
Foundation Wall Design

Calc. by
LC

Date
12/17/2014

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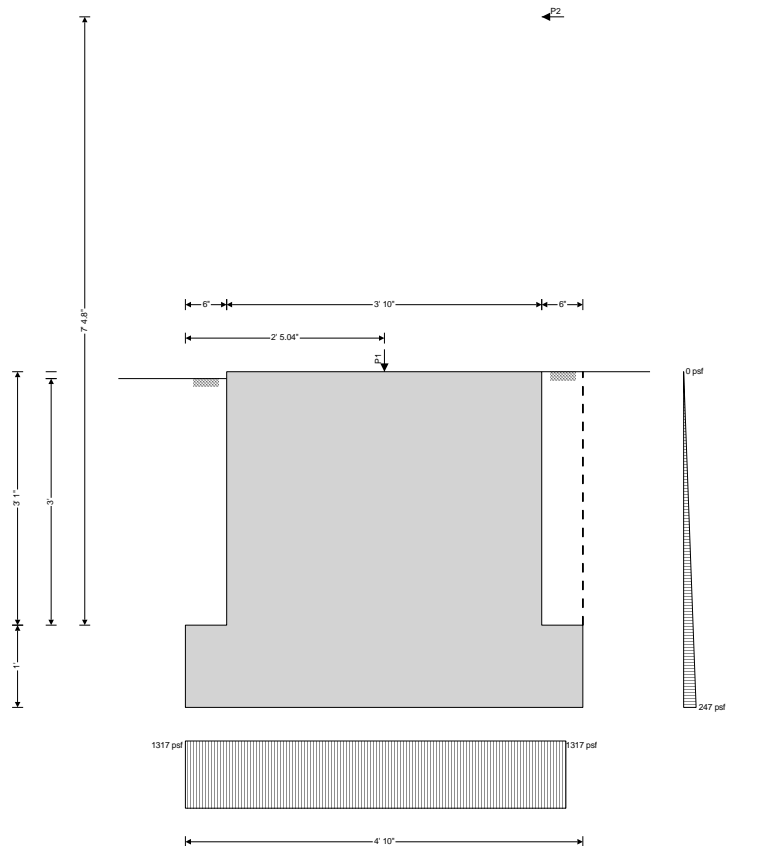
Date

Job Ref. Page 12 of 21
28133363.09

Sheet no./rev.
2

App'd by

Date



Calculate retaining wall geometry

Base length

$$l_{base} = l_{toe} + t_{stem} + l_{heel} = 4.833 \text{ ft}$$

Moist soil height

$$h_{moist} = h_{soil} = 3.083 \text{ ft}$$

Retained surface length

$$l_{sur} = l_{heel} = 0.5 \text{ ft}$$

Effective height of wall

$$h_{eff} = h_{base} + d_{cover} + h_{ret} = 4.083 \text{ ft}$$

Area of wall stem

$$A_{stem} = h_{stem} \times t_{stem} = 11.819 \text{ ft}^2$$

- Distance to vertical component

$$x_{stem} = l_{toe} + t_{stem} / 2 = 2.417 \text{ ft}$$

Area of wall base

$$A_{base} = l_{base} \times t_{base} = 4.833 \text{ ft}^2$$

- Distance to vertical component

$$x_{base} = l_{base} / 2 = 2.417 \text{ ft}$$

Area of moist soil

$$A_{moist} = h_{moist} \times l_{heel} = 1.542 \text{ ft}^2$$

- Distance to vertical component

$$x_{moist_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist} = 4.583 \text{ ft}$$

- Distance to horizontal component

$$x_{moist_h} = h_{eff} / 3 = 1.361 \text{ ft}$$

Area of base soil

$$A_{pass} = d_{cover} \times l_{toe} = 1.5 \text{ ft}^2$$

- Distance to vertical component

$$x_{pass_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass} = 0.25 \text{ ft}$$

- Distance to horizontal component

$$x_{pass_h} = (d_{cover} + h_{base}) / 3 = 1.333 \text{ ft}$$

Area of excavated base soil

$$A_{exc} = h_{pass} \times l_{toe} = 1.5 \text{ ft}^2$$

- Distance to vertical component

$$x_{exc_v} = l_{base} - (h_{pass} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{exc} = 0.25 \text{ ft}$$

- Distance to horizontal component

$$x_{exc_h} = (h_{pass} + h_{base}) / 3 = 1.333 \text{ ft}$$

Using Coulomb theory

At rest pressure coefficient

$$K_0 = 1 - \sin(\phi_r) = 0.500$$



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Project Ft. Custer Columbarium				Job RefPage 13 of 21 28133363.09	
Section Foundation Wall Design				Sheet no./rev. 3	
Calc. by LC	Date 12/17/2014	Chk'd by	Date	App'd by	Date

Passive pressure coefficient

$$K_P = \sin(90 - \phi_b)^2 / (\sin(90 + \delta_b) \times [1 - \sqrt{(\sin(\phi_b + \delta_b) \times \sin(\phi_b) / (\sin(90 + \delta_b)))^2}] = \mathbf{4.977}$$

From IBC 2012 cl.1807.2.3 Safety factor

Load combination 1

$$1.0 \times \text{Dead} + 1.0 \times \text{Live} + 1.0 \times \text{Lateral earth}$$

Sliding check

Vertical forces on wall

Wall stem

$$F_{\text{stem}} = A_{\text{stem}} \times \gamma_{\text{stem}} = \mathbf{1773 \text{ plf}}$$

Wall base

$$F_{\text{base}} = A_{\text{base}} \times \gamma_{\text{base}} = \mathbf{725 \text{ plf}}$$

Line loads

$$F_{P_V} = P_{D1} = \mathbf{3230 \text{ plf}}$$

Moist retained soil

$$F_{\text{moist}_V} = A_{\text{moist}} \times \gamma_{\text{mr}} = \mathbf{193 \text{ plf}}$$

Base soil

$$F_{\text{exc}_V} = A_{\text{exc}} \times \gamma_{\text{mb}} = \mathbf{173 \text{ plf}}$$

Total

$$F_{\text{total}_V} = F_{\text{stem}} + F_{\text{base}} + F_{\text{moist}_V} + F_{\text{exc}_V} + F_{P_V} = \mathbf{6093 \text{ plf}}$$

Horizontal forces on wall

Line loads

$$F_{P_h} = P_{L2} = \mathbf{227 \text{ plf}}$$

Moist retained soil

$$F_{\text{moist}_h} = K_0 \times \cos(\delta_r) \times \gamma_{\text{mr}} \times h_{\text{eff}}^2 / 2 = \mathbf{503 \text{ plf}}$$

Total

$$F_{\text{total}_h} = F_{\text{moist}_h} + F_{P_h} = \mathbf{730 \text{ plf}}$$

Check stability against sliding

Base soil resistance

$$F_{\text{exc}_h} = K_P \times \cos(\delta_b) \times \gamma_{\text{mb}} \times (h_{\text{pass}} + h_{\text{base}})^2 / 2 = \mathbf{4422 \text{ plf}}$$

Base friction

$$F_{\text{friction}} = F_{\text{total}_V} \times \tan(\delta_{bb}) = \mathbf{3518 \text{ plf}}$$

Resistance to sliding

$$F_{\text{rest}} = F_{\text{exc}_h} + F_{\text{friction}} = \mathbf{7940 \text{ plf}}$$

Factor of safety

$$F_{\text{OSl}} = F_{\text{rest}} / F_{\text{total}_h} = \mathbf{10.873} > 1.5$$

PASS - Factor of safety against sliding is adequate

Overturning check

Vertical forces on wall

Wall stem

$$F_{\text{stem}} = A_{\text{stem}} \times \gamma_{\text{stem}} = \mathbf{1773 \text{ plf}}$$

Wall base

$$F_{\text{base}} = A_{\text{base}} \times \gamma_{\text{base}} = \mathbf{725 \text{ plf}}$$

Line loads

$$F_{P_V} = P_{D1} = \mathbf{3230 \text{ plf}}$$

Moist retained soil

$$F_{\text{moist}_V} = A_{\text{moist}} \times \gamma_{\text{mr}} = \mathbf{193 \text{ plf}}$$

Base soil

$$F_{\text{exc}_V} = A_{\text{exc}} \times \gamma_{\text{mb}} = \mathbf{173 \text{ plf}}$$

Total

$$F_{\text{total}_V} = F_{\text{stem}} + F_{\text{base}} + F_{\text{moist}_V} + F_{\text{exc}_V} + F_{P_V} = \mathbf{6093 \text{ plf}}$$

Horizontal forces on wall

Line loads

$$F_{P_h} = P_{L2} = \mathbf{227 \text{ plf}}$$

Moist retained soil

$$F_{\text{moist}_h} = K_0 \times \cos(\delta_r) \times \gamma_{\text{mr}} \times h_{\text{eff}}^2 / 2 = \mathbf{503 \text{ plf}}$$

Base soil

$$F_{\text{exc}_h} = \max(-K_P \times \cos(\delta_b) \times \gamma_{\text{mb}} \times (h_{\text{pass}} + h_{\text{base}})^2 / 2, \min(-F_{\text{moist}_h} - F_{P_h}, 0 \text{ plf})) = \mathbf{-730 \text{ plf}}$$

Total

$$F_{\text{total}_h} = \max(F_{\text{moist}_h} + F_{\text{exc}_h} + F_{P_h}, 0 \text{ plf}) = \mathbf{0 \text{ plf}}$$

Overturning moments on wall

Line loads

$$M_{P_OT} = \text{abs}(P_{L2}) \times (p_2 + t_{\text{base}}) = \mathbf{1907 \text{ lb_ft/ft}}$$

Moist retained soil

$$M_{\text{moist}_OT} = F_{\text{moist}_h} \times X_{\text{moist}_h} = \mathbf{685 \text{ lb_ft/ft}}$$

Total

$$M_{\text{total}_OT} = M_{\text{moist}_OT} + M_{P_OT} = \mathbf{2592 \text{ lb_ft/ft}}$$



KCI Technologies
936 Ridgebrook Road
Sparks, MD 21152

Project Ft. Custer Columbarium				Job RefPage 14 of 21 28133363.09	
Section Foundation Wall Design				Sheet no./rev. 4	
Calc. by LC	Date 12/17/2014	Chk'd by	Date	App'd by	Date

Restoring moments on wall

Wall stem	$M_{stem_R} = F_{stem} \times X_{stem} = 4285 \text{ lb_ft/ft}$
Wall base	$M_{base_R} = F_{base} \times X_{base} = 1752 \text{ lb_ft/ft}$
Line loads	$M_{P_R} = \text{abs}(P_{D1}) \times p_1 = 7817 \text{ lb_ft/ft}$
Moist retained soil	$M_{moist_R} = F_{moist_v} \times X_{moist_v} = 883 \text{ lb_ft/ft}$
Base soil	$M_{exc_R} = F_{exc_v} \times X_{exc_v} - F_{exc_h} \times X_{exc_h} = 1017 \text{ lb_ft/ft}$
Total	$M_{total_R} = M_{stem_R} + M_{base_R} + M_{moist_R} + M_{exc_R} + M_{P_R} = 15753 \text{ lb_ft/ft}$

Check stability against overturning

Factor of safety	$FoS_{ot} = M_{total_R} / M_{total_OT} = 6.078 > 1.5$ PASS - Factor of safety against overturning is adequate
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Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} = 1773 \text{ plf}$
Wall base	$F_{base} = A_{base} \times \gamma_{base} = 725 \text{ plf}$
Line loads	$F_{P_v} = P_{D1} = 3230 \text{ plf}$
Moist retained soil	$F_{moist_v} = A_{moist} \times \gamma_{mr} = 193 \text{ plf}$
Base soil	$F_{pass_v} = A_{pass} \times \gamma_{mb} = 173 \text{ plf}$
Total	$F_{total_v} = F_{stem} + F_{base} + F_{moist_v} + F_{pass_v} + F_{P_v} = 6093 \text{ plf}$

Horizontal forces on wall

Line loads	$F_{P_h} = P_{L2} = 227 \text{ plf}$
Moist retained soil	$F_{moist_h} = K_0 \times \cos(\delta_r) \times \gamma_{mr} \times h_{eff}^2 / 2 = 503 \text{ plf}$
Total	$F_{total_h} = \max(F_{moist_h} + F_{pass_h} + F_{P_h} - F_{total_v} \times \tan(\delta_{bb}), 0 \text{ plf}) = 0 \text{ plf}$

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times X_{stem} = 4285 \text{ lb_ft/ft}$
Wall base	$M_{base} = F_{base} \times X_{base} = 1752 \text{ lb_ft/ft}$
Line loads	$M_P = P_{D1} \times p_1 = 7817 \text{ lb_ft/ft}$
Moist retained soil	$M_{moist} = F_{moist_v} \times X_{moist_v} - F_{moist_h} \times X_{moist_h} = 198 \text{ lb_ft/ft}$
Base soil	$M_{pass} = F_{pass_v} \times X_{pass_v} = 43 \text{ lb_ft/ft}$
Total	$M_{total} = M_{stem} + M_{base} + M_{moist} + M_{pass} + M_P = 14095 \text{ lb_ft/ft}$

Check bearing pressure

Distance to reaction	$\bar{x} = M_{total} / F_{total_v} = 2.313 \text{ ft}$
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 = -0.103 \text{ ft}$
Loaded length of base	$l_{load} = 2 \times \bar{x} = 4.626 \text{ ft}$
Bearing pressure at toe	$q_{toe} = F_{total_v} / l_{load} = 1317 \text{ psf}$
Bearing pressure at heel	$q_{heel} = 0 \text{ psf}$
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = 1.519$ PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with ACI 318-11



KCI Technologies
936 Ridgebrook Road
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Project Ft. Custer Columbarium				Job Ref. Page 15 of 21 28133363.09	
Section Foundation Wall Design				Sheet no./rev. 5	
Calc. by LC	Date 12/17/2014	Chk'd by	Date	App'd by	Date

Concrete details

Compressive strength of concrete $f'_c = 3000$ psi
Concrete type Normal weight

Reinforcement details

Yield strength of reinforcement $f_y = 60000$ psi
Modulus of elasticity of reinforcement $E_s = 29000000$ psi

Cover to reinforcement

Front face of stem $C_{sf} = 1.5$ in
Rear face of stem $C_{sr} = 1.5$ in
Top face of base $C_{bt} = 2$ in
Bottom face of base $C_{bb} = 3$ in

From IBC 2012 cl.1605.2.1 Basic load combinations

Load combination no.1 $1.4 \times \text{Dead}$
Load combination no.2 $1.2 \times \text{Dead} + 1.6 \times \text{Live} + 1.6 \times \text{Lateral earth}$
Load combination no.3 $1.2 \times \text{Dead} + 1.0 \times \text{Earthquake} + 1.0 \times \text{Live} + 1.6 \times \text{Lateral earth}$
Load combination no.4 $0.9 \times \text{Dead} + 1.0 \times \text{Earthquake} + 1.6 \times \text{Lateral earth}$

Check stem design at base of stem

Depth of section $h = 46$ in

Rectangular section in flexure - Chapter 10

Design bending moment combination 2 $M = 3160$ lb_ft/ft
Depth of tension reinforcement $d = h - C_{sr} - \phi_{sr} / 2 = 44.188$ in
Compression reinforcement provided No.5 bars @ 18" c/c
Area of compression reinforcement provided $A_{sf,prov} = \pi \times \phi_{sf}^2 / (4 \times S_{sf}) = 0.205$ in²/ft
Tension reinforcement provided No.5 bars @ 18" c/c
Area of tension reinforcement provided $A_{sr,prov} = \pi \times \phi_{sr}^2 / (4 \times S_{sr}) = 0.205$ in²/ft
Maximum reinforcement spacing - cl.14.3.5 $S_{max} = \min(18 \text{ in}, 3 \times h) = 18$ in

PASS - Reinforcement is adequately spaced

Depth of compression block $a = A_{sr,prov} \times f_y / (0.85 \times f'_c) = 0.401$ in
Neutral axis factor - cl.10.2.7.3 $\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) = 0.85$
Depth to neutral axis $c = a / \beta_1 = 0.472$ in
Strain in reinforcement $\epsilon_t = 0.003 \times (d - c) / c = 0.277964$

Section is in the tension controlled zone

Strength reduction factor $\phi_f = \min(\max(0.65 + (\epsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = 0.9$
Nominal flexural strength $M_n = A_{sr,prov} \times f_y \times (d - a / 2) = 44983$ lb_ft/ft
Design flexural strength $\phi M_n = \phi_f \times M_n = 40485$ lb_ft/ft
 $M / \phi M_n = 0.078$

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis $A_{sr,des} = 0.016$ in²/ft
Minimum area of reinforcement - cl.10.5.3 $A_{sr,mod} = 4 \times A_{sr,des} / 3 = 0.021$ in²/ft

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Chapter 11

Design shear force $V = 822$ lb/ft



KCI Technologies
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Project Ft. Custer Columbarium				Job Ref. Page 16 of 21 28133363.09	
Section Foundation Wall Design				Sheet no./rev. 6	
Calc. by LC	Date 12/17/2014	Chk'd by	Date	App'd by	Date

Concrete modification factor - cl.8.6.1

$$\lambda = 1$$

Nominal concrete shear strength - exp.11-3

$$V_c = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times d = 58086 \text{ lb/ft}$$

Strength reduction factor

$$\phi_s = 0.75$$

Design concrete shear strength - cl.11.4.6.1

$$\phi V_c = \phi_s \times V_c = 43564 \text{ lb/ft}$$

$$V / \phi V_c = 0.019$$

PASS - No shear reinforcement is required

Horizontal reinforcement parallel to face of stem

Minimum area of reinforcement - cl.14.3.3

$$A_{sx,req} = 0.0025 \times t_{stem} = 1.38 \text{ in}^2/\text{ft}$$

Transverse reinforcement provided

No.6 bars @ 7" c/c each face

Area of transverse reinforcement provided

$$A_{sx,prov} = 2 \times \pi \times \phi_{sx}^2 / (4 \times s_{sx}) = 1.515 \text{ in}^2/\text{ft}$$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section

$$h = 12 \text{ in}$$

Rectangular section in flexure - Chapter 10

Design bending moment combination 3

$$M = 146 \text{ lb}_\text{ft}/\text{ft}$$

Depth of tension reinforcement

$$d = h - C_{bb} - \phi_{bb} / 2 = 8.625 \text{ in}$$

Compression reinforcement provided

No.3 bars @ 12" c/c

Area of compression reinforcement provided

$$A_{bt,prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt}) = 0.11 \text{ in}^2/\text{ft}$$

Tension reinforcement provided

No.6 bars @ 12" c/c

Area of tension reinforcement provided

$$A_{bb,prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb}) = 0.442 \text{ in}^2/\text{ft}$$

Maximum reinforcement spacing - cl.10.5.4

$$s_{max} = \min(18 \text{ in}, 3 \times h) = 18 \text{ in}$$

PASS - Reinforcement is adequately spaced

Depth of compression block

$$a = A_{bb,prov} \times f_y / (0.85 \times f'_c) = 0.866 \text{ in}$$

Neutral axis factor - cl.10.2.7.3

$$\beta_1 = \min(\max(0.85 - 0.05 \times (f'_c - 4 \text{ ksi}) / 1 \text{ ksi}, 0.65), 0.85) = 0.85$$

Depth to neutral axis

$$c = a / \beta_1 = 1.019 \text{ in}$$

Strain in reinforcement

$$\epsilon_t = 0.003 \times (d - c) / c = 0.02239$$

Section is in the tension controlled zone

Strength reduction factor

$$\phi_f = \min(\max(0.65 + (\epsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = 0.9$$

Nominal flexural strength

$$M_n = A_{bb,prov} \times f_y \times (d - a / 2) = 18095 \text{ lb}_\text{ft}/\text{ft}$$

Design flexural strength

$$\phi M_n = \phi_f \times M_n = 16286 \text{ lb}_\text{ft}/\text{ft}$$

$$M / \phi M_n = 0.009$$

PASS - Design flexural strength exceeds factored bending moment

By iteration, reinforcement required by analysis

$$A_{bb,des} = 0.004 \text{ in}^2/\text{ft}$$

Minimum area of reinforcement - cl.7.12.2.1

$$A_{bb,min} = 0.0018 \times h = 0.259 \text{ in}^2/\text{ft}$$

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear - Chapter 11

Design shear force

$$V = 578 \text{ lb/ft}$$

Concrete modification factor - cl.8.6.1

$$\lambda = 1$$

Nominal concrete shear strength - exp.11-3

$$V_c = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times d = 11338 \text{ lb/ft}$$

Strength reduction factor

$$\phi_s = 0.75$$

Design concrete shear strength - cl.11.4.6.1

$$\phi V_c = \phi_s \times V_c = 8503 \text{ lb/ft}$$

$$V / \phi V_c = 0.068$$



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Project
Ft. Custer Columbarium

Section
Foundation Wall Design

Calc. by
LC

Date
12/17/2014

Chk'd by

Date

Job Ref. Page 17 of 21
28133363.09

Sheet no./rev.
7

App'd by

Date

PASS - No shear reinforcement is required

Rectangular section in shear - Chapter 11

Design shear force

$$V = 516 \text{ lb/ft}$$

Concrete modification factor - cl.8.6.1

$$\lambda = 1$$

Nominal concrete shear strength - exp.11-3

$$V_c = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times d = 11338 \text{ lb/ft}$$

Strength reduction factor

$$\phi_s = 0.75$$

Design concrete shear strength - cl.11.4.6.1

$$\phi V_c = \phi_s \times V_c = 8503 \text{ lb/ft}$$

$$V / \phi V_c = 0.061$$

PASS - No shear reinforcement is required

Transverse reinforcement parallel to base

Minimum area of reinforcement - cl.7.12.2.1

$$A_{bx,req} = 0.0018 \times t_{base} = 0.259 \text{ in}^2/\text{ft}$$

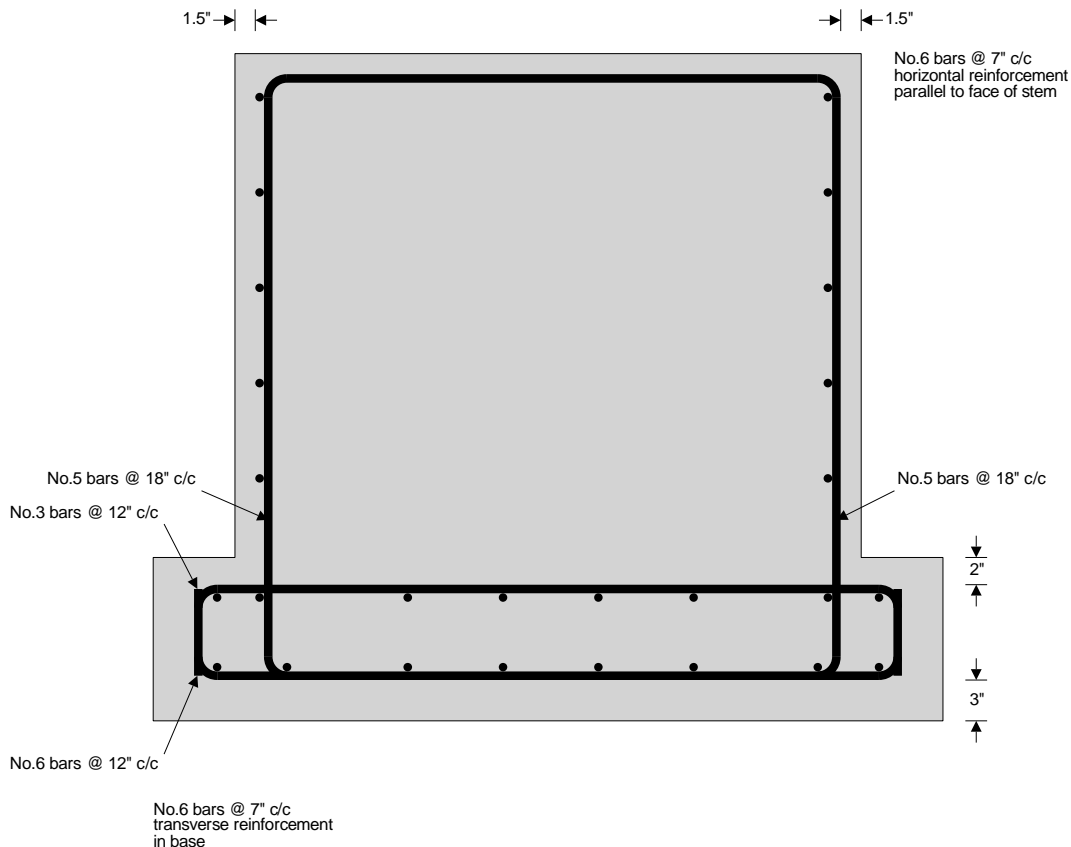
Transverse reinforcement provided

No.6 bars @ 7" c/c each face

Area of transverse reinforcement provided

$$A_{bx,prov} = 2 \times \pi \times \phi_{bx}^2 / (4 \times S_{bx}) = 1.515 \text{ in}^2/\text{ft}$$

PASS - Area of reinforcement provided is greater than area of reinforcement required





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Project
Ft. Custer Columbarium

Section
Concrete Pier Design

Calc. by
LC

Date
2/4/2015

Chk'd by

Date

Job Ref.
39133363.09

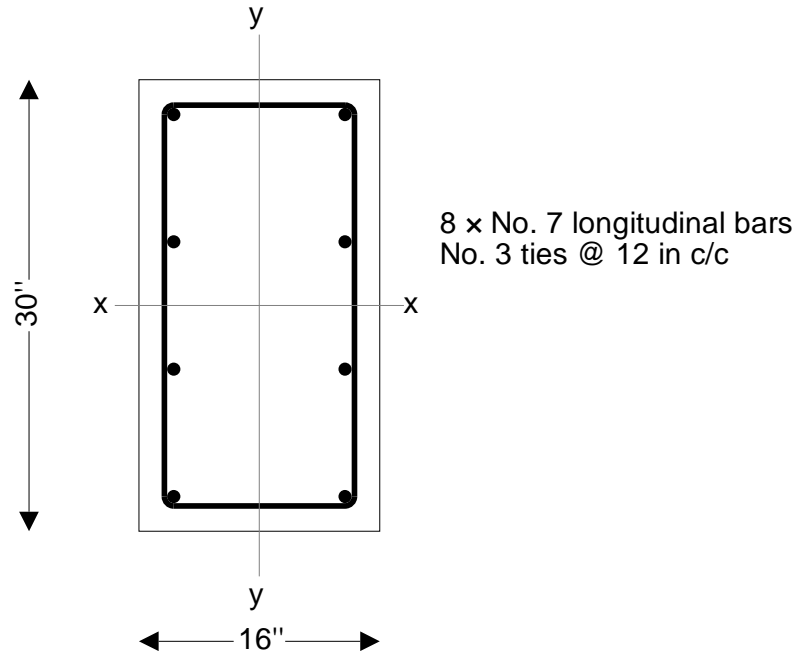
Sheet no./rev.
1

App'd by

Date

RC RECTANGULAR COLUMN DESIGN (ACI318-11)

TEDDS calculation version 2.1.05



Applied loads

Ultimate axial force acting on column

$P_{u_act} = 12$ kips

Ultimate moment about major (X) axis

$M_{ux_act} = 1.7$ kips_ft

Geometry of column

Depth of column (larger dimension of column)

$h = 30.0$ in

Width of column (smaller dimension of column)

$b = 16.0$ in

Clear cover to reinforcement (both sides)

$c_c = 1.5$ in

Unsupported height of column about x axis

$l_{ux} = 8.0$ ft

Effective height factor about x axis

$k_x = 1.00$

Column state about the x axis

Unbraced

Unsupported height of column about y axis

$l_{uy} = 8.0$ ft

Effective height factor about y axis

$k_y = 1.00$

Column state about the y axis

Braced

Check on overall column dimensions

Column dimensions are OK - $h < 4b$

Reinforcement of column

Numbers of bars of longitudinal steel

$N = 8$

Longitudinal steel bar diameter number

$D_{bar_num} = 7$

Diameter of longitudinal bar

$D_{long} = 0.875$ in

Stirrup bar diameter number


$D_{stir_num} = 3$

Diameter of stirrup bar

$D_{stir} = 0.375$ in

Specified yield strength of reinforcement

$f_y = 60000$ psi

 KCI Technologies 936 Ridgebrook Road Sparks, MD 21152	Project Ft. Custer Columbarium				Job Ref. 39133363.09	
	Section Concrete Pier Design				Sheet no./rev. 2	
	Calc. by LC	Date 2/4/2015	Chk'd by	Date	App'd by	Date

Specified compressive strength of concrete

$$f'_c = 3000 \text{ psi}$$

Modulus of elasticity of bar reinforcement

$$E_s = 29 \times 10^6 \text{ psi}$$

Modulus of elasticity of concrete

$$E_c = 57000 \times f'_c^{1/2} \times (1 \text{ psi})^{1/2} = 3122019 \text{ psi}$$

Ultimate design strain

$$\epsilon_c = 0.003 \text{ in/in}$$

Check for minimum area of steel to CL.10.9

Gross area of column

$$A_g = h \times b = 480.000 \text{ in}^2$$

Area of steel

$$A_{st} = N \times (\pi \times D_{long}^2) / 4 = 4.811 \text{ in}^2$$

Minimum area of steel required

$$A_{st_min} = 0.01 \times A_g = 4.800 \text{ in}^2$$

$A_{st} > A_{st_min}$, PASS- Minimum steel check

Check for maximum area of steel to CL.10.9

Permissible maximum area of steel

$$A_{st_max} = 0.08 \times A_g = 38.400 \text{ in}^2$$

$A_{st} < A_{st_max}$, PASS - Maximum steel check

Slenderness check about x axis

Radius of gyration

$$r_x = 0.3 \times h = 9 \text{ in}$$

Actual slenderness ratio

$$S_{rx_act} = k_x \times l_{ux} / r_x = 10.67$$

Slenderness ratio is less than 22, slenderness effects may be neglected

Slenderness check about y axis

Radius of gyration

$$r_y = 0.3 \times b = 4.8 \text{ in}$$

Actual slenderness ratio

$$S_{ry_act} = k_y \times l_{uy} / r_y = 20$$

Slenderness ratio is less than 22, slenderness effects may be neglected

Uniaxially loaded column about major axis

Details of column cross-section

c/d_t ratio

$$r_{xb} = 0.450$$

Effective cover to reinforcement

$$d' = c_c + D_{stir} + (D_{long}/2) = 2.312 \text{ in}$$

Spacing between bars

$$s = ((h - (2 \times d')) / ((N/2) - 1)) = 8.458 \text{ in}$$

Depth of tension steel

$$d_t = h - d' = 27.688 \text{ in}$$

Depth of NA from extreme compression face

$$c_x = r_{xb} \times d_t = 12.459 \text{ in}$$

Factor of depth of compressive stress block

$$\beta_1 = 0.850$$

Depth of equivalent rectangular stress block

$$a_x = \min((\beta_1 \times c_x), h) = 10.590 \text{ in}$$

Yield strain in steel

$$\epsilon_{sx} = f_y / E_s = 0.002$$

Strength reduction factor

$$\phi_x = 0.789$$

Details of concrete block

Force carried by concrete

Forces carried by concrete

$$P_{xcon} = 0.85 \times f'_c \times b \times a_x = 432.091 \text{ kips}$$

Moment carried by concrete

Moment carried by concrete

$$M_{xcon} = P_{xcon} \times ((h/2) - (a_x/2)) = 349.445 \text{ kip_ft}$$

Details of steel layer 1

Depth of layer

$$x_{x1} = 2.312 \text{ in}$$

Strain of layer


$$\epsilon_{x1} = \epsilon_c \times (1 - x_{x1} / c_x) = 0.00244$$

Stress in layer

$$\sigma_{x1} = \min(f_y, E_s \times \epsilon_{x1}) = 60000.00 \text{ psi}$$

Force carried by layer

$$P_{x1} = N_x \times A_{bar} \times \sigma_{x1} = 72.158 \text{ kips}$$

 KCI Technologies 936 Ridgebrook Road Sparks, MD 21152	Project Ft. Custer Columbarium		Job Ref. 39133363.09	
	Section Concrete Pier Design		Sheet no./rev. 3	
	Calc. by LC	Date 2/4/2015	Chk'd by	Date
			App'd by	Date

Moment carried by steel layer

$$M_{x1} = P_{x1} \times ((h / 2) - x_{x1}) = \mathbf{76.293 \text{ kip_ft}}$$

Details of steel layer 2

Depth of layer

$$x_{x2} = \mathbf{10.771 \text{ in}}$$

Strain of layer

$$\epsilon_{x2} = \epsilon_c \times (1 - x_{x2} / C_x) = \mathbf{0.00041}$$

Stress in layer

$$\sigma_{x2} = \min(f_y, E_s \times \epsilon_{x2}) = \mathbf{11790.57 \text{ psi}}$$

Force carried by layer

$$P_{x2} = 2 \times A_{bar} \times \sigma_{x2} = \mathbf{14.180 \text{ kips}}$$

Moment carried by steel layer

$$M_{x2} = P_{x2} \times ((h / 2) - x_{x2}) = \mathbf{4.997 \text{ kip_ft}}$$

Details of steel layer 3

Depth of layer

$$x_{x3} = \mathbf{19.229 \text{ in}}$$

Strain of layer

$$\epsilon_{x3} = \epsilon_c \times (1 - x_{x3} / C_x) = \mathbf{-0.00163}$$

Stress in layer

$$\sigma_{x3} = \max(-1 \times f_y, E_s \times \epsilon_{x3}) = \mathbf{-47271.38 \text{ psi}}$$

Force carried by layer

$$P_{x3} = 2 \times A_{bar} \times \sigma_{x3} = \mathbf{-56.850 \text{ kips}}$$

Moment carried by steel layer

$$M_{x3} = P_{x3} \times ((h / 2) - x_{x3}) = \mathbf{20.036 \text{ kip_ft}}$$

Details of steel layer 4

Depth of layer

$$x_{x4} = \mathbf{27.688 \text{ in}}$$

Strain of layer

$$\epsilon_{x4} = \epsilon_c \times (1 - x_{x4} / C_x) = \mathbf{-0.00367}$$

Stress in layer

$$\sigma_{x4} = \max(-1 \times f_y, E_s \times \epsilon_{x4}) = \mathbf{-60000.00 \text{ psi}}$$

Force carried by layer

$$P_{x4} = N_x \times A_{bar} \times \sigma_{x4} = \mathbf{-72.158 \text{ kips}}$$

Moment carried by steel layer

$$M_{x4} = P_{x4} \times ((h / 2) - x_{x4}) = \mathbf{76.293 \text{ kip_ft}}$$

Force carried by steel

Sum of forces by steel

$$P_{xs} = \mathbf{-42.7 \text{ kips}}$$

Total force carried by column

Nominal axial load strength

$$P_{nx} = \mathbf{389.420 \text{ kips}}$$

Strength reduction factor

$$\phi_x = \mathbf{0.789}$$

Ultimate axial load carrying capacity of column

$$P_{ux} = \phi_x \times P_{nx} = \mathbf{307.209 \text{ kips}}$$

Total moment carried by column

Total moment carried by column

$$M_{ox} = \mathbf{527.064 \text{ kip_ft}}$$

Ultimate moment strength capacity of column

$$M_{ux} = \phi_x \times M_{ox} = \mathbf{415.795 \text{ kip_ft}}$$

Check load capacity for uniaxial loads about the x axis

Factored axial load

$$P_{u_act} = \mathbf{12 \text{ kips}}$$

Ultimate axial capacity

$$P_{ux} = \mathbf{307.21 \text{ kips}}$$

PASS - Ultimate axial capacity exceeds factored axial load

Factored moment about x axis

$$M_{ux_act} = \mathbf{1.7 \text{ kip_ft}}$$

Ultimate moment capacity about the x axis

$$M_{ux} = \mathbf{415.79 \text{ kip_ft}}$$

PASS - Ultimate moment capacity exceeds factored moment about x axis

Design of column ties to CL.7.10.5

Spacing of lateral ties

$$S_{v_ties} = \mathbf{12.000 \text{ in}}$$

16 times longitudinal bar diameter

$$S_{v1} = 16 \times D_{long} = \mathbf{14.000 \text{ in}}$$

48 times tie bar diameter

$$S_{v2} = 48 \times D_{stir} = \mathbf{18.000 \text{ in}}$$

Least column dimension

$$S_{v3} = \min(h, b) = \mathbf{16.000 \text{ in}}$$

Provide max tie spacing

$$s = \min(S_{v1}, S_{v2}, S_{v3}) = \mathbf{14.000 \text{ in}}$$

$S_{v_ties} < s$ PASS



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Project

Ft. Custer Columbarium

Job Ref.

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Section

Concrete Pier Design

Sheet no./rev.

4

Calc. by

LC

Date

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Date

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Date