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Purpose: Design pile supported foundation for 3-story hotel structure.

- Sources:
1. ASCE 7-10
 2. IBC 2012
 3. ACI 318
 4. Drawings by Dammon Engineering dated January, 2016
 5. Soils Analysis by Daniel J. Holder, PE dated September 18, 2015

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WIND ANALYSIS



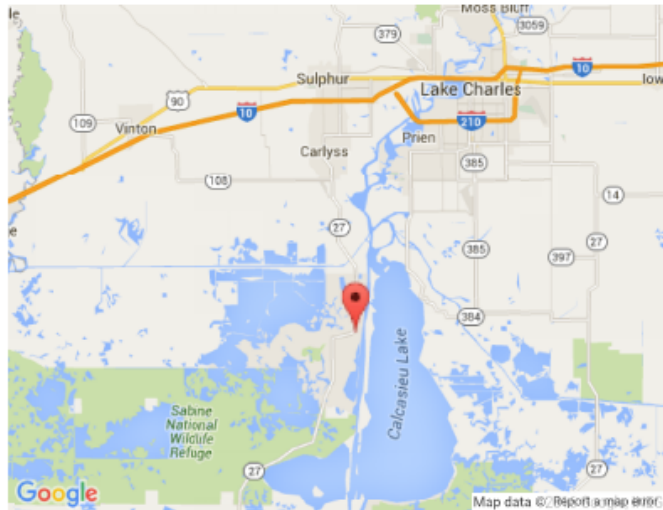
Search Results

Query Date: Fri Jan 29 2016
 Latitude: 29.9960
 Longitude: -93.3421

ASCE 7-10 Windspeeds
 (3-sec peak gust in mph*):

Risk Category I: 127
 Risk Category II: 138
 Risk Category III-IV: 148
 MRI** 10-Year: 76
 MRI** 25-Year: 91
 MRI** 50-Year: 101
 MRI** 100-Year: 112

ASCE 7-05 Windspeed:
 116 (3-sec peak gust in mph)
 ASCE 7-93 Windspeed:
 94 (fastest mile in mph)

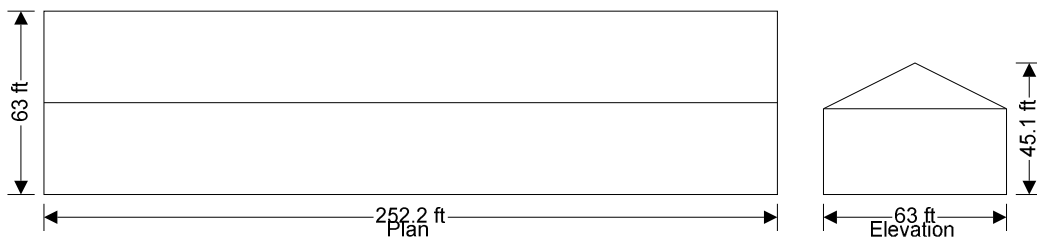


Design wind speed per ASCE7-10 is 138mph.

WIND LOADING (ASCE7-10)

Using the directional design method

TEDDS calculation version 2.0.11



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Building data

Type of roof Gable
Length of building $b = 252.23$ ft
Width of building $d = 63.00$ ft
Height to eaves $H = 29.44$ ft
Pitch of roof $\alpha_0 = 26.4$ deg
Mean height $h = 37.27$ ft

General wind load requirements

Basic wind speed $V = 138.0$ mph
Risk category II
Velocity pressure exponent coeff (Table 26.6-1) $K_d = 0.85$
Exposure category (cl.26.7.3) C
Enclosure classification (cl.26.10) Enclosed buildings
Internal pressure coef +ve (Table 26.11-1) $GC_{pi,p} = 0.18$
Internal pressure coef -ve (Table 26.11-1) $GC_{pi,n} = -0.18$
Gust effect factor $G_f = 0.85$

Topography

Topography factor not significant $K_{zt} = 1.0$
Velocity pressure equation $q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1 \text{psf}/\text{mph}^2$

Velocity pressures table

z (ft)	K_z (Table 27.3-1)	q_z (psf)
15.00	0.85	35.22
20.00	0.90	37.30
29.44	0.98	40.43
30.00	0.98	40.61
37.27	1.02	42.42
45.10	1.07	44.15

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) $q_i = 42.42$ psf

Pressures and forces

Net pressure $p = q \times G_f \times C_{pe} - q_i \times GC_{pi}$
Net force $F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, $GC_{pi} 0.18$, $-C_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
A (-ve)	37.27	-0.30	42.42	-18.54	8873.39	-164.56
B (-ve)	37.27	-0.60	42.42	-29.27	8873.39	-259.72

Total vertical net force $F_{w,v} = -379.90$ kips

Total horizontal net force $F_{w,h} = 42.37$ kips

Walls load case 1 - Wind 0, $GC_{pi} 0.18$, $-C_{pe}$

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Zone	Ref. height (ft)	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
A ₁	15.00	0.80	35.22	16.32	3783.45	61.73
A ₂	20.00	0.80	37.30	17.73	1261.15	22.35
A ₃	29.44	0.80	40.43	19.85	2381.05	47.27
B	37.27	-0.50	42.42	-25.66	7425.65	-190.57
C	37.27	-0.70	42.42	-32.87	2348.14	-77.19
D	37.27	-0.70	42.42	-32.87	2348.14	-77.19

Overall loading

Projected vertical plan area of wall

$$A_{vert_w_0} = b \times H = 7425.65 \text{ ft}^2$$

Projected vertical area of roof

$$A_{vert_r_0} = b \times d/2 \times \tan(\alpha_0) = 3950.97 \text{ ft}^2$$

Minimum overall horizontal loading

$$F_{w,total_min} = p_{min_w} \times A_{vert_w_0} + p_{min_r} \times A_{vert_r_0} = 150.4 \text{ kips}$$

Leeward net force

$$F_l = F_{w,wB} = -190.6 \text{ kips}$$

Windward net force

$$F_w = F_{w,wA_1} + F_{w,wA_2} + F_{w,wA_3} = 131.4 \text{ kips}$$

Overall horizontal loading

$$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total_min}) = 364.3 \text{ kips}$$

Roof load case 2 - Wind 0, $GC_{pi} -0.18$, $-0C_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
A (+ve)	37.27	0.17	42.42	13.91	8873.39	123.39
B (+ve)	37.27	-0.60	42.42	-14.00	8873.39	-124.21

Total vertical net force

$$F_{w,v} = -0.73 \text{ kips}$$

Total horizontal net force

$$F_{w,h} = 110.25 \text{ kips}$$

Walls load case 2 - Wind 0, $GC_{pi} -0.18$, $-0C_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
A ₁	15.00	0.80	35.22	31.59	3783.45	119.51
A ₂	20.00	0.80	37.30	33.00	1261.15	41.61
A ₃	29.44	0.80	40.43	35.12	2381.05	83.63
B	37.27	-0.50	42.42	-10.39	7425.65	-77.17
C	37.27	-0.70	42.42	-17.60	2348.14	-41.34
D	37.27	-0.70	42.42	-17.60	2348.14	-41.34

Overall loading

Projected vertical plan area of wall

$$A_{vert_w_0} = b \times H = 7425.65 \text{ ft}^2$$

Projected vertical area of roof

$$A_{vert_r_0} = b \times d/2 \times \tan(\alpha_0) = 3950.97 \text{ ft}^2$$

Minimum overall horizontal loading

$$F_{w,total_min} = p_{min_w} \times A_{vert_w_0} + p_{min_r} \times A_{vert_r_0} = 150.4 \text{ kips}$$

Leeward net force

$$F_l = F_{w,wB} = -77.2 \text{ kips}$$

Windward net force

$$F_w = F_{w,wA_1} + F_{w,wA_2} + F_{w,wA_3} = 244.8 \text{ kips}$$

Overall horizontal loading

$$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total_min}) = 432.2 \text{ kips}$$

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Roof load case 3 - Wind 90, GC_{pi} 0.18, -c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient C _{pe}	Peak velocity pressure q _p , (psf)	Net pressure p (psf)	Area A _{ref} (ft ²)	Net force F _w (kips)
A (-ve)	37.27	-0.90	42.42	-40.09	1311.22	-52.56
B (-ve)	37.27	-0.90	42.42	-40.09	1311.22	-52.56
C (-ve)	37.27	-0.50	42.42	-25.66	2622.45	-67.30
D (-ve)	37.27	-0.30	42.42	-18.45	12501.89	-230.69

Total vertical net force F_{w,v} = **-360.95** kips

Total horizontal net force F_{w,h} = **0.00** kips

Walls load case 3 - Wind 90, GC_{pi} 0.18, -c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient C _{pe}	Peak velocity pressure q _p , (psf)	Net pressure p (psf)	Area A _{ref} (ft ²)	Net force F _w (kips)
A ₁	15.00	0.80	35.22	16.32	945.00	15.42
A ₂	30.00	0.80	40.61	19.98	944.37	18.87
A ₃	45.10	0.80	44.15	22.39	458.77	10.27
B	37.27	-0.20	42.42	-14.85	2348.14	-34.86
C	37.27	-0.70	42.42	-32.87	7425.65	-244.12
D	37.27	-0.70	42.42	-32.87	7425.65	-244.12

Overall loading

Projected vertical plan area of wall

$$A_{\text{vert}_w_{90}} = d \times H + d^2 \times \tan(\alpha_0) / 4 = \mathbf{2348.14 \text{ ft}^2}$$

Projected vertical area of roof

$$A_{\text{vert}_r_{90}} = \mathbf{0.00 \text{ ft}^2}$$

Minimum overall horizontal loading

$$F_{w,\text{total_min}} = p_{\text{min}_w} \times A_{\text{vert}_w_{90}} + p_{\text{min}_r} \times A_{\text{vert}_r_{90}} = \mathbf{37.6 \text{ kips}}$$

Leeward net force

$$F_l = F_{w,wB} = \mathbf{-34.9 \text{ kips}}$$

Windward net force

$$F_w = F_{w,wA_1} + F_{w,wA_2} + F_{w,wA_3} = \mathbf{44.6 \text{ kips}}$$

Overall horizontal loading

$$F_{w,\text{total}} = \max(F_w - F_l + F_{w,h}, F_{w,\text{total_min}}) = \mathbf{79.4 \text{ kips}}$$

Roof load case 4 - Wind 90, GC_{pi} -0.18, +c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient C _{pe}	Peak velocity pressure q _p , (psf)	Net pressure p (psf)	Area A _{ref} (ft ²)	Net force F _w (kips)
A (+ve)	37.27	-0.18	42.42	1.15	1311.22	1.50
B (+ve)	37.27	-0.18	42.42	1.15	1311.22	1.50
C (+ve)	37.27	-0.18	42.42	1.15	2622.45	3.00
D (+ve)	37.27	-0.18	42.42	1.15	12501.89	14.32

Total vertical net force F_{w,v} = **18.20** kips

Total horizontal net force F_{w,h} = **0.00** kips

Walls load case 4 - Wind 90, GC_{pi} -0.18, +c_{pe}

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Zone	Ref. height (ft)	Ext pressure coefficient C_{pe}	Peak velocity pressure q_p , (psf)	Net pressure p (psf)	Area A_{ref} (ft ²)	Net force F_w (kips)
A ₁	15.00	0.80	35.22	31.59	945.00	29.85
A ₂	30.00	0.80	40.61	35.25	944.37	33.29
A ₃	45.10	0.80	44.15	37.66	458.77	17.28
B	37.27	-0.20	42.42	0.42	2348.14	1.00
C	37.27	-0.70	42.42	-17.60	7425.65	-130.72
D	37.27	-0.70	42.42	-17.60	7425.65	-130.72

Overall loading

Projected vertical plan area of wall

$$A_{vert_w_90} = d \times H + d^2 \times \tan(\alpha_0) / 4 = 2348.14 \text{ ft}^2$$

Projected vertical area of roof

$$A_{vert_r_90} = 0.00 \text{ ft}^2$$

Minimum overall horizontal loading

$$F_{w_total_min} = p_{min_w} \times A_{vert_w_90} + p_{min_r} \times A_{vert_r_90} = 37.6 \text{ kips}$$

Leeward net force

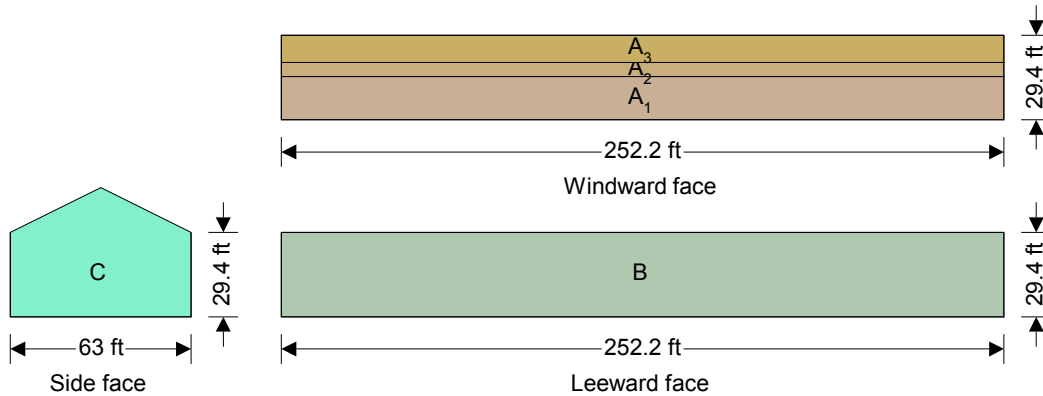
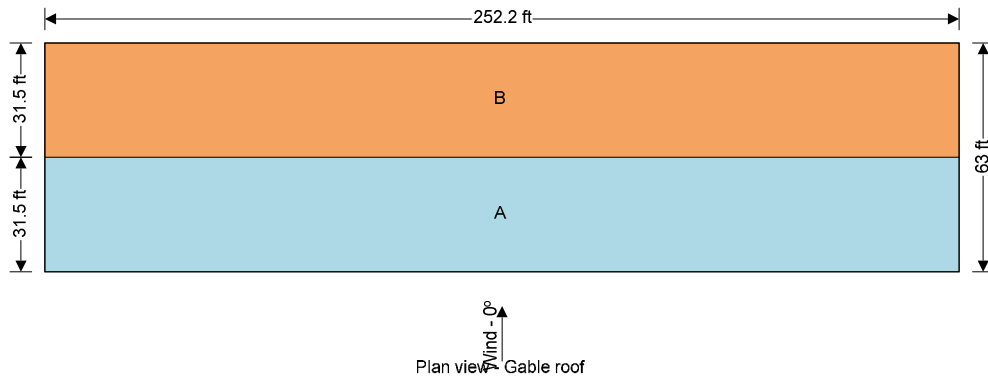
$$F_l = F_{w_WB} = 1.0 \text{ kips}$$

Windward net force

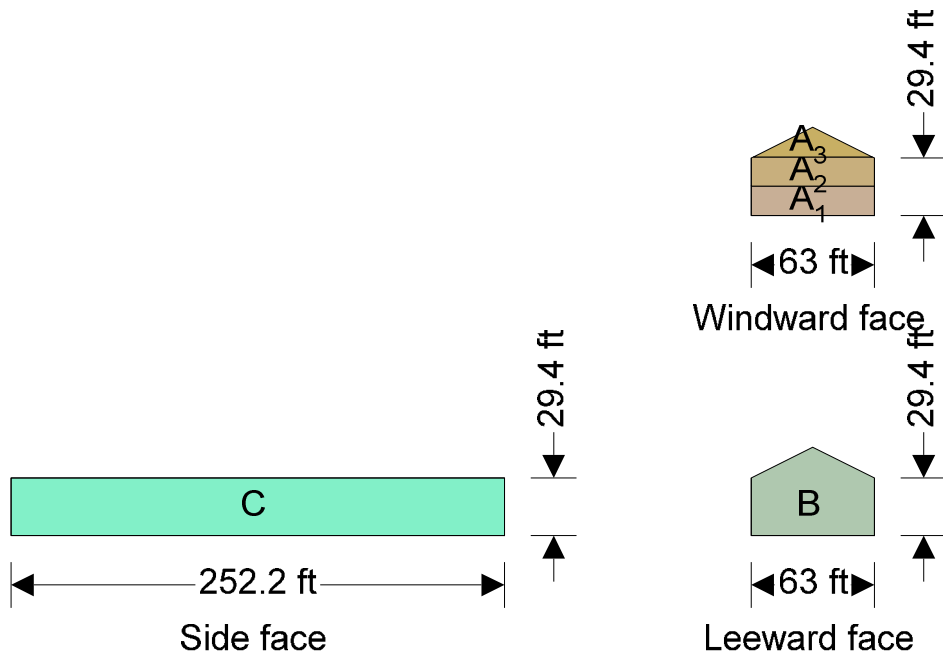
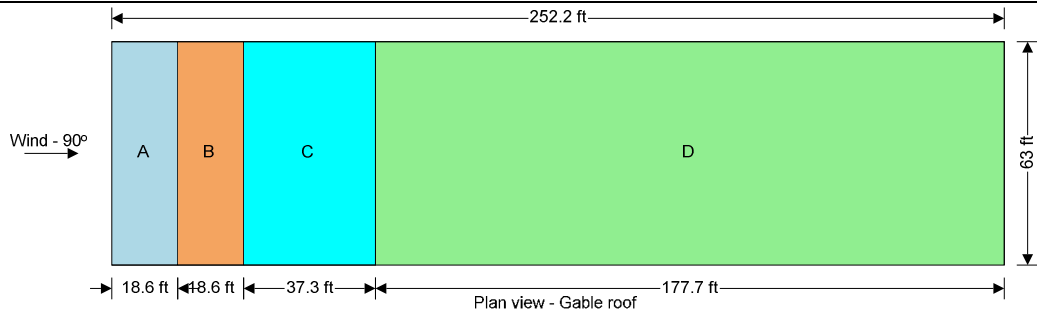
$$F_w = F_{w_wA_1} + F_{w_wA_2} + F_{w_wA_3} = 80.4 \text{ kips}$$

Overall horizontal loading

$$F_{w_total} = \max(F_w - F_l + F_{w_h}, F_{w_total_min}) = 79.4 \text{ kips}$$



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ROOF LOADS

Assume wood trusses @ 24" OC spanning from exterior wall to corridor walls. Asphalt shingles, 5/8" plywood sheathing, 5/8" gyp wall board on bottom chord. Include 5psf Collateral load.

DL Asphalt shingles = 2psf
 5/8" plywood = 2psf
 Wood trusses = 15psf
 5/8" gyp clg = 3psf
 Collateral = 5psf
 Total = 27psf

Use DL_{rf} = 30psf
 RLL=20psf

Roof loads @ exterior wall:

$$W_{DL,rf,ext} = (27.83ft/2) * DL_{rf} * ((6^2 + 12^2)^{.5}) / 12 = 467plf$$

$$W_{RLL,ext} = (27.83ft/2) * RLL = 278plf$$

Roof loads @ corridor wall:

$$W_{DL,rf,cor} = ((27.83ft/2) + (7.33ft/2)) * DL_{rf} * ((6^2 + 12^2)^{.5}) / 12 = 590plf$$

$$W_{RLL,cor} = ((27.83ft/2) + (7.33ft/2)) * RLL = 352plf$$

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2ND AND 3RD FLOOR LOADS

Assume 18" trusses @ 24" OC, 3/4" plywood subfloor, 2" LW concrete deck (130pcf), 5/8" gypsum clg. Include 5psf collateral load.

DL 2" LW concrete deck = 22psf
 3/4" plywood = 2.4psf
 Wood trusses = 10psf
 5/8" gyp ceiling = 3psf
 Collateral = 5psf
 Total = 42.4 psf

Use $DL_{flr} = 45psf$
 $LL_{flr} = 40psf$
 (hotel private rooms and corridors serving them)

Floor loads @ exterior walls:

$$W_{DL,flr,ext} = (27.83ft/2) * DL_{flr} = 626plf$$

$$W_{LL,flr,ext} = (27.83ft/2) * LL_{flr} = 557plf$$

Floor loads @ corridor walls:

$$W_{DL,flr,cor} = ((27.83ft/2) + (7.33ft/2)) * DL_{flr} = 791plf$$

$$W_{LL,flr,cor} = ((27.83ft/2) + (7.33ft/2)) * LL_{flr} = 703plf$$

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WALL LOADS

Assume first floor exterior wall brick veneer with 2x6 studs @ 16" OC, 1/2 sheathing, 5/8" gyp finish interior, 2nd and 3rd floor walls 2x6 studs @ 16" OC with hardiboard siding, 1/2" sheathing, 5/8" gyp finish interior.

$$DL_{1\text{flr},\text{wall}} = 48\text{psf}$$

$$DL_{2\&3\text{flr},\text{wall}} = 12\text{psf}$$

(wall dead loads are per ASCE7-10 chapter C3)

Wall heights:

$$H_{1\text{flr},\text{wall}} = 11.83\text{ft}$$

$$H_{2\text{flr},\text{wall}} = 8.46\text{ft}$$

$$H_{3\text{flr},\text{wall}} = 9.1\text{ft}$$

Dead load @ exterior walls:

$$W_{DL,\text{wall},\text{ext}} = (H_{1\text{flr},\text{wall}} * DL_{1\text{flr},\text{wall}}) + ((H_{2\text{flr},\text{wall}} + H_{3\text{flr},\text{wall}}) * DL_{2\&3\text{flr},\text{wall}}) = 780\text{plf}$$

Dead load @ corridor walls: Assume 12psf load conservatively

$$W_{DL,\text{wall},\text{cor}} = (H_{1\text{flr},\text{wall}} + H_{2\text{flr},\text{wall}} + H_{3\text{flr},\text{wall}}) * DL_{2\&3\text{flr},\text{wall}} = 353\text{plf}$$

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FOUNDATION LOADS

Exterior wall:

$$W_{DL, \text{fnd, ext}} = W_{DL, \text{rf, ext}} + W_{DL, \text{flr, ext}} + W_{DL, \text{wall, ext}} = 2500 \text{plf}$$

$$W_{LL, \text{fnd, ext}} = W_{RLL, \text{ext}} + W_{LL, \text{flr, ext}} = 1392 \text{plf}$$

$$W_{\text{total, fnd, ext}} = 3892 \text{plf}$$

Corridor wall:

$$W_{DL, \text{fnd, cor}} = W_{DL, \text{rf, cor}} + W_{DL, \text{flr, cor}} + W_{DL, \text{wall, cor}} = 2525 \text{plf}$$

$$W_{LL, \text{fnd, cor}} = W_{RLL, \text{cor}} + W_{LL, \text{flr, cor}} = 1758 \text{plf}$$

$$W_{\text{total, fnd, cor}} = 4283 \text{plf}$$

Bearing pressure check:

Per soils report allowable bearing pressure for continuous grade beams is 1600psf. Loads will be supported by piles. However, determine required footing width for applied loads to prevent soil failure prior to pile transfer.

Exterior wall:

$$L_{\text{req, ext}} = 3892 \text{lbs} / 1600 \text{psf} = 2.43 \text{ft}^2$$

Corridor wall:

$$L_{\text{req, cor}} = 4283 \text{lbs} / 1600 \text{psf} = 2.67 \text{ft}^2$$

Note: Required bearing width excessive. Size grade beams as required to transfer loads to piles. Assume no soil support for required moment capacity.

Assume 24"x24" grade beams, with soil supported slab.

Weight of grade beams:

$$W_{\text{gdbm}} = 2\text{ft} \times 2\text{ft} \times 1\text{ft} \times 150 \text{pcf} = 600 \text{plf}$$

Design loads for piles:

$$W_{\text{total, pile, ext}} = W_{\text{total, fnd, ext}} + W_{\text{gdbm}} = 4492 \text{plf}$$

$$W_{\text{total, pile, cor}} = W_{\text{total, fnd, cor}} + W_{\text{gdbm}} = 4883 \text{plf}$$

Per geotech report, use the following pile capacities for 12" butt piles:

30': 60kips

40': 72kips

50': 85kips

60': 97kips

Required spacing for pile lengths

Exterior wall:

$$30': 60 \text{kips} / 4492 \text{plf} = 13.35 \text{ft}$$

Corridor wall:

$$30': 60 \text{kips} / 4883 \text{plf} = 12.29 \text{ft}$$

Note: 30' piles appear to be adequate. Depths in excess of 30ft not evaluated.

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GRADE BEAM STRENGTH

Assume 24"x24" GB with (3)#5 bars top and btm.

LRFD loads for beam strength:

$$W_{LRFD,gb,ext} = (1.2*(W_{DL,fnd,ext} + W_{gdbm})) + (1.6*W_{LL,fnd,ext}) = 5947\text{plf}$$

$$W_{LRFD,gb,cor} = (1.2*(W_{DL,fnd,cor} + W_{gdbm})) + (1.6*W_{LL,fnd,cor}) = 6563\text{plf}$$

Base required grade beam strength on worst case (corridor). Piles are laid out in CAD to limit spacing to less than 12.29ft. Additional piles are added at grade beam intersections. Resulting worst case span is approximately 11.33ft due to building geometry.

$$M_u = (6563 * 11.33\text{ft}^2) / 8 = 105.31\text{k-ft} = 1263.73\text{k-in}$$

$$\phi M_n = \phi * (A_s * f_c * a * b * (d - (a/2)))$$

$$\phi = 0.9$$

$$A_s = 0.31\text{in}^2 * 3 = 0.93\text{in}^2$$

$$F'_c = 4000\text{psi}$$

$$A = \beta * c = 0.68\text{in}$$

$$C = (A_s * f_y) / (0.85 * f'_c * b) / \beta = 0.8\text{in}$$

$$B = 24\text{in}$$

$$D = 24\text{in} - 3\text{in} - (3/8\text{in}) - (5/8\text{in} / 2) = 20.3125\text{in}$$

$$F_y = 60000\text{psi}$$

$$\phi M_n = 1091\text{k-in}$$

NO GOOD

Try increasing depth of grade beam to 30"

$$D = 30\text{in} - 3\text{in} - (3/8\text{in}) - (5/8\text{in} / 3) = 26.3125\text{in}$$

$$\phi M_n = 1419\text{k-in}$$

OK

Use 24" wide x 30" deep grade beams at all load bearing grade beams.

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OVERTURNING STABILITY CHECK

Conservatively assume all overturning moments are resisted by reactions at perimeter wall. Use ASD load combination $0.9DL+1.0WL$ for overturning check. Assume that shear walls and wall framing holdowns are of adequate strength to transfer lateral loads to the foundation.

Overturning moment:

$$M_o = 432.2 \text{ kips} \times 21.3 \text{ ft} = \mathbf{9205.9 \text{ kip_ft}}$$

Restoring moment:

$$W_{DL, \text{fnd, ext}} = 2500 \text{ plf}$$

$$L = 252.25 \text{ ft}$$

$$W_{gdbm, \text{ext}} = 150 \text{ pcf} \times 30 \text{ in} \times 24 \text{ in} = \mathbf{750.0 \text{ plf}}$$

$$D = 47.58 \text{ ft}$$

$$M_r = 0.9 \times (W_{DL, \text{fnd, ext}} + W_{gdbm, \text{ext}}) \times L \times D = \mathbf{35106.0 \text{ kip_ft}}$$

Weight of structure and exterior footing combined is adequate to resist overturning moment caused by wind.