STRUCTURAL CALCULATIONS

FOR

LOUISIANA NATIONAL CEMETERY GRAVESITE EXPANSION

ZACHARY, LA

03/22/2012

Client: Jacobs
B&B Principal in Charge: Brian O. Reil
B&B Project Engineer: George A. McCluskey
B&B Job No.: S08-090.08

P.I.C./Administration Building

Project Description & Design Criteria *AT1*

Vertical Loads *AT2*

Roof Framing Design *AR1*

Column Design *AC1*

Lateral Design *AL1*

Wall Design *AW1*

Foundation Design *AFD1*

Appendix A: Design Tables *AA1*

Maintenance Building

Project Description & Design Criteria *MT1*

Lateral Design *ML1*

Wall Design *MW1*

Foundation Design *MFD1*

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Bulk Storage Building

Project Description & Design Criteria *BT1*

Roof Framing Design *BR1*

Lateral Design *BL1*

Wall Design *BW1*

Foundation Design *BFD1*

Committal Shelter

Project Description & Design Criteria *CT1*

Lateral Design *CL1*

Wall Design *CW1*

Site Miscellaneous Structures

Site Structures *SM1*

Project Description:

The expansion project for the VA National Cemetery in Zachary, Louisiana consists of three one-story structures and one shelter/canopy structure. The project is located in a hurricane zone with 110mph basic wind speed; requiring hurricane ties of all roof elements down to the foundation at all the buildings.

The Administration building is a one story building. The roof consists of prefabricated wood trusses with plywood sheathing supported by light-gage metal stud exterior bearing walls. The walls use Sureboard for shear resistance and bear on continuous footings.

The maintenance building is a pre-manufactured one-story metal building. The roof is part of the pre-manufactured design and consists of Z purlins and WF beams that span to WF columns. Rod bracing provides lateral resistance and it is also designed by the manufacturer. The columns will bear on spread footings and the building has a continuous footing all around. There is a bulk storage structure that is considered part of the maintenance building. Roof structure consists of wood framing with plywood sheathing. Wood stud non-bearing walls sit on top of 11' tall concrete walls, which sit on continuous footings.

There is also a shelter structure that has a pre-manufactured roof and lateral system comprised of tube members. The columns will bear on spread footings that will be connected with grade beams.

Design Criteria:

Building Type: Occupancy Category: I - Bulk Storage, Committal Shelter Structure
II – Administration and Maintenance Buildings

Design Codes: 2009 International Building Code
ASCE 7-05: Minimum Design Loads for Buildings and Other Structures, including *Supplement 1*
AISC 360-05: Specification for Structural Steel Buildings (13th Edition AISC)
ACI 318-08: Building Code Requirements for Structural Concrete
AF&PA 2005 NDS and 2008 SDPWS

Wind Criteria: 110 mph, Exposure C $I_w = 1.00$ $GC_{pi} = \pm 0.18$ (Enclosed Structure)

Seismic Criteria: Site Class: D $S_S = 0.122$ $S_1 = 0.055$ $S_{DS} = 0.130$ $S_{D1} = 0.088$

Seismic Design Category: B $I = 1.00$ $I_p = 1.0$

Lateral Force-Resisting System:

Administration Building
Light-gage metal stud bearing shear walls
$R = 6.5$ $\Omega_o = 3.0$ $C_d = 4.0$

Bulk Storage Building
Wood bearing and concrete shear walls
$R = 4.0$ $\Omega_o = 2.5$ $C_d = 4.0$

Maintenance Building/Committal Shelter
Light-gage metal stud bearing shear walls
$R = 3.5$ $\Omega_o = 3.0$ $C_d = 3.0$

Foundation Criteria:

Soils Report #870 CM 3022 by W.L.Burle Engineers, P.A., dated 2/15/2011

Allowable Bearing Pressures: (Non-structural elements)	DL	2300 psf
	DL + LL	2300 psf
	DL + LL + EQ/1.4	3067 psf
Allowable Lateral Pressures:	Retaining At-Rest Pressure	55 pcf
	Passive Pressure	300 pcf
	Coefficient of Friction	0.30

Material Specifications:

Concrete: (Cement ASTM C-150)		W/C Ratio
Footings, pier caps:	f'_c = 4000 psi	0.52
Slab-on-grade:	f'_c = 3500 psi	0.45
Shear Walls:	f'_c = 4000 psi	0.50

Structural Steel:

WF shapes:	ASTM A992 (50 ksi)
HSS (rectangular):	ASTM A500-B (46 ksi)
HSS (round):	ASTM A500-B (42 ksi)
Pipe:	ASTM A53 (35 ksi)
Channel & Angle:	ASTM A36 (36 ksi)
Plate:	ASTM A36 (36 ksi)
Anchor Rods (frames):	ASTM F1554 (gr. 36)

A: Roof Loads (Administration Bldg)

	Trusses	seismic weight
Asphalt Shingles	2.5	2.5
3/4" shtg	2.3	2.3
Trusses @ 32"cc	3.0	3.0
Insulation	0.5	0.5
sprinklers	1.0	1.0
MEP	2.0	2.0
ceilings	2.2	2.2
misc	1.5	1.5
DPO - dead load	15.0 psf	15.0 psf
LLR - reducible roof LL	20.0 psf	0.0 psf
TL	35.0 psf	15.0 psf

B: Roof Loads (Maintenance Bldg)

	deck	seismic weight
metal roofing (20ga)	1.8	1.8
Z' purlins (6"x16ga @ 24"cc)	1.5	1.5
Insulation	3.0	3.0
beams	2.9	2.9
columns	1.0	1.0
sprinklers	1.0	1.0
MEP	2.0	2.0
ceilings	1.8	1.8
misc	1.0	1.0
DPO - dead load	16.0 psf	16.0 psf
LLR - reducible roof LL	20.0 psf	0.0 psf
TL	36.0 psf	16.0 psf

C: Roof Loads (Bulk Storage)

	Joists	seismic weight
metal roofing (18ga)	1.8	1.8
single-ply roof (w/ re-roof)	5.5	5.5
5/8" shtg	1.9	1.9
2x10's @ 24"cc	1.7	1.7
sprinklers	1.0	1.0
MEP	1.0	1.0
misc	0.9	0.9
DPO - dead load	12.0 psf	12.0 psf
LLR - reducible roof LL	20.0 psf	0.0 psf
TL	32.0 psf	12.0 psf

D: Roof Loads (Committal Shelter)

	deck	seismic weight
metal roofing (20ga)	1.8	1.8
Z' purlins (6"x16ga @ 24"cc)	1.5	1.5
beams	2.2	2.2
columns	1.0	1.0
sprinklers	1.0	1.0
MEP	2.0	2.0
misc	1.0	1.0
DPO - dead load	10.5 psf	10.5 psf
LLR - reducible roof LL	20.0 psf	0.0 psf
TL	30.5 psf	10.5 psf

E: Wall Loads

Wall Finish:	Light-Gage	Wood	Concrete
6"x16ga mtl studs @ 16"cc	1.5		
2x6 wood studs @ 16"cc		1.5	
12" concrete			150.0
Metal siding (both sides)		3.0	
1/2" shtg (both sides)		3.4	
3 5/8" brick veneer	35.0		
1/2" Sureboard	4.3		
5/8" gyp board	2.8	2.8	
misc	1.4	1.3	
DPO - non-composite DL	45.0 psf	12.0 psf	150.0 psf

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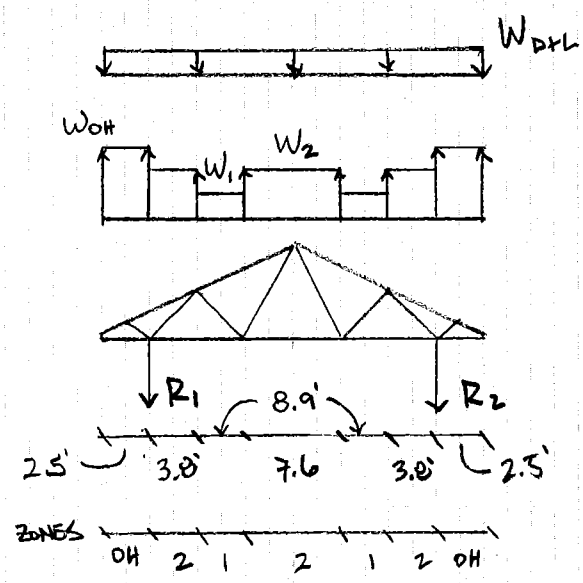
JOB _____
JOB NO. _____ DATE _____
CLIENT _____ BY _____ SHEET NO. AR1

ROOF FRAMING DESIGN (Admin bldg)

TYPICAL TRUSS LOADING



- Trusses will be designed by others
- Spaced @ 32" cc
- Gravity loads
  - ↳ W_{DL} = 15 PSF x 32"/12 = 40 PLF
  - W_{LL} = 20 PSF x 32"/12 = 53 PLF
- Wind uplift
  - ↳ W_1 = -14.4 PSF x 32"/12 = 38 PLF
  - W_2 = -23.7 PSF x 32"/12 = 63 PLF
  - W_{OH} = -51.1 PSF x 32"/12 = 136 PLF
- LC = 0.6D - W
  - ↳ W_{net_1} = 0.6 (40 PLF) - 38 PLF = -14 PLF
  - W_{net_2} = 0.6 (40 PLF) - 63 PLF = -39 PLF
  - $W_{net_{OH}}$ = 0.6 (40 PLF) - 136 PLF = -112 PLF

R_1 = 112 PLF x 2.5' + 39 PLF x (3.8' x 2) + 14 PLF (8.9') = 701 # (uplift)

- Assume trusses are centered btwn 2-studs.

F_u = 701 #/2 = 351 #

⇒ Use Simpson H10 ties @ ea truss to top PL conn

CAP = 995 #

⇒ Use Simpson SP4 ties @ ea stud CAP = 825 #

DLR @ stud = (45 PSF)(16"/12)(10.5') = 630 # x 0.6 = 378 #

strap CAP = 790 #
3-#10 = 789 #

⇒ Use 12ga x 1 3/8" x 0'-6" strap w/ 3-#10 to stud & 3-3/16" Titen screws to curb 3-Titen = 1245 # (w/ 1 1/2" embed)

ROOF FRAMING DESIGN (Admin Bldg)

HIP TRUSS LOADING

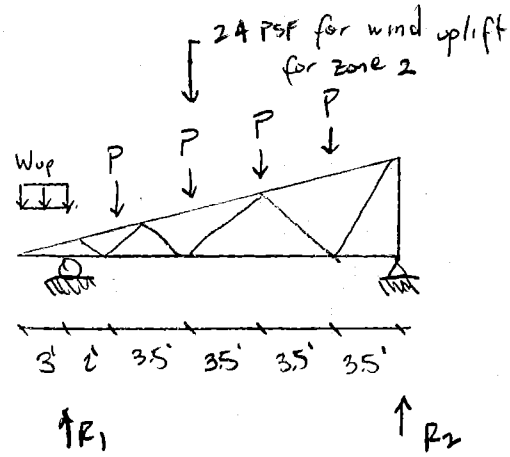
- Gravity Loads
  - ↳ Loads from trusses (P_T)

P_{DL} = 15 PSF × 2.67' × 4' = 160 #

P_{LL} = 20 PSF × 2.67' × 4' = 214 #

P_{UP} = -24 PSF × 2.67' × 4' = -256 #

P_{net} = 0.6 (160 #) - 256 # = -160 #
× 2 sides = -320 #



R_1 = - 320 # (3.5' + 7' + 10.5' + 14') / 16' = - 700 # - 77 PSF (32"/12)(2') = -1110 #

R_2 = - 320 # (2' + 5.5' + 9' + 12.5') / 16' = -580 #

⇒ Use Simpson THJA26 hanger @ girder truss

CAP: 960 #

⇒ Use Simpson A35 clips @ wall support

CAP: 2 × 595 # = 1190 #

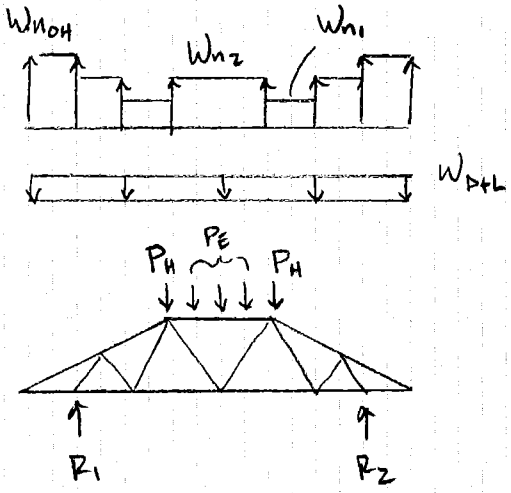
⇒ Use Simpson S/LTT20 @ bottom of built-up post

CAP: 1200 #

ROOF FRAMING DESIGN (Admin bldg)

GIRDER TRUSS LOADING

- Gravity loads
  - ↳ Loads from hip truss (P_H)
    - ↳ $P_{DL} = 290\# \times 2 = 580\#$
      $P_{LL} = 388\# \times 2 = 776\#$
      $P_{UP} = -464\# \times 2 = -928\#$
      $P_{net} = 0.6 \times 580\# - 928\# = -580\#$
  - ↳ Loads from End truss (P_E)
    - ↳ $P_{DL} = 15\ PSF \times 32''/12 \times 11'/2 = 220\#$
      $P_{LL} = 20\ PSF \times 32''/12 \times 11'/2 = 293\#$
      $P_{UP} = -14\ PSF \times 32''/12 \times 11'/2 = 211\#$
      $P_{net} = 0.6 \times 220\# - 211\# = -79\#$



$W_{DL} = 40\ PSF$

$W_{LL} = 53\ PLF$

$$R_1 = \underbrace{-580\#}_{\text{from hip truss}} - \underbrace{3 \times 79\#}_{\text{from end trusses}} - \underbrace{701\#/2}_{\text{from typ loading}} = 1167\# \quad \text{(uplift)}$$

⇒ use Simpson H14 ties @ ea girder truss

CAP: 1350#

CHECK WIND UPLIFT ON PLWD NAILS

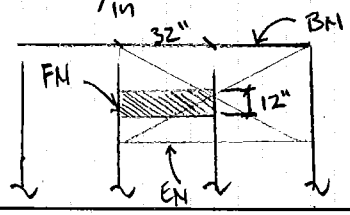
10d @ 12" cc x 32" Panel

Highest load = $112\ PLF \times 12''/12''/FT = 112\#$

10d withdrawal cap = $36\#/in \times 3'' = 108\# \times 1.6 = 172.8\#$ (C_d)

∴ 10d @ 12" cc OK ✓

* Highest uplift occurs @ overhang.



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CLIENT _____ BY _____ SHEET NO. AR4

ROOF FRAMING DESIGN

BEAMS @ BREEZEWAY ENTRANCE

SPAN = 14'

W = (15 PSF + 20 PSF) 5' = 175 PLF

W_{uplift} = 37.3 PLF x 4.5' = 168 PLF

V = 1225 #

M = 4288 #'

$EI_{req'd}$ = 450 (175 PLF) $(14')^3 / 10^6$ = 216.1

/29000 = 7.45

→ USE 6x8 beam

V_a = 2338 #

M_a = 5156 #

EI = 309 x 10^6

OR

USE H4-8 hdr min

I = 13.42

W_{NET} = 0.6 (175 PLF) - 168 PLF = -63 PLF

V = 378 # ⇒ Simpson HUC hangers ea end OK✓

BEAMS @ OPENINGS IN BREEZEWAY

SPAN = 10'-0"

W = (15 PSF + 20 PSF) 19'/2 = 333 PLF

V = 1665 #

M = 4163 #'

EI = 450 (333 PLF) $(10')^3 / 10^6$ = 150

⇒ USE 6x8 beam

OR

H4-8 hdr min

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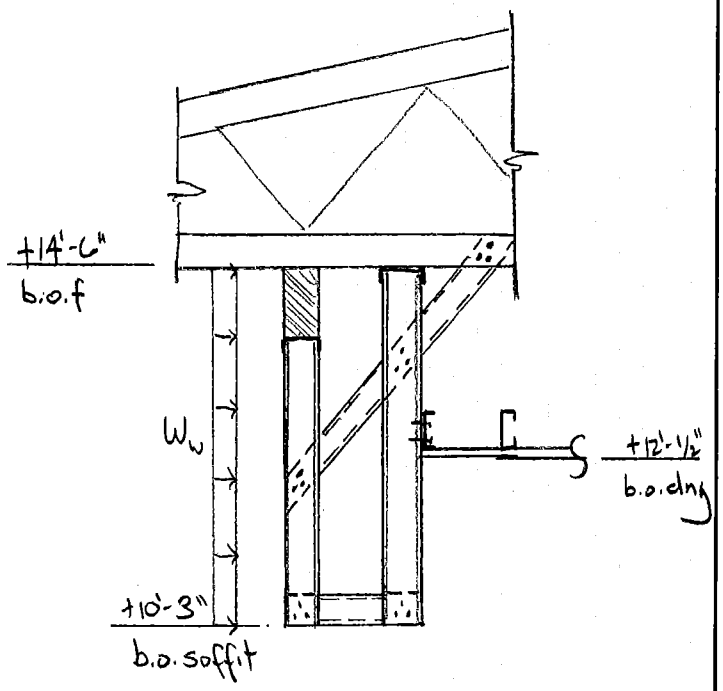
JOB _____
JOB NO. _____ DATE _____
CLIENT _____ BY _____ SHEET NO. AR5

ROOF FRAMING DESIGN

BEAMS @ BREEZEWAY ENTRANCE



Check wind load effects

SPAN = 12'

$W_w = 37.3 \text{ PSF} \times 4.25'/2 = 79.3 \text{ PLF}$

V = 474

M = 1422 #'

$EI_{req'd} = 450(79 \text{ PLF})(12')^3/10' = 61.4$

6x8 ok ✓

$EI_y = 166$

Check combined loading

$M_{ay} = 3791$ #'

$M_{ax} = 5156$ #'

$\Rightarrow \dfrac{4288 \text{ \#'}}{1.25 \times 5156 \text{ \#'}} + \dfrac{1422 \text{ \#'}}{3791 \text{ \#'} \times 1.6} = 0.90 \rightarrow$ 6x8 ok ✓

CEILING JOISTS @ BREEZEWAY

SPAN = 13'

$W_{uplift} = 31 \text{ PSF}(0.85)(0.8) = 21 \text{ PSF} \times 2' = 42 \text{ PLF}$

V = 273 #

M = 887 #'

$I_{req'd} = 450(42 \text{ PLF})(13')^2/29000 = 1.43 \text{ in}^4$

$\Rightarrow$ use 6" x 20ga ctg joists @ 24" cc

$I = 1.79 \text{ in}^4$

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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	10/26/2011
CLIENT	Jacbos	BY	GAM	SHEET NO.	AL1

ASCE 7-05 (incl. supp. #1) Seismic Base Shear Calculation

Location: Zachary — Mapped MCE Spectral Acceleration (from Lat/Long using USGS program):
Height (hn): 17.0 ft — Latitude = 30.6439 — Longitude = -91.2567 — Ss= 0.122g
Regularity: regular — S1= 0.055g
Seismic Force-Resisting Sys. (Table 12.2-1): Light gage Sureboard Shear Walls — R= 2
Period Coefficients: 0.028 (0.8) for steel MRF, 0.016 (0.9) for concrete MRF — Ct= 0.020
listed as Ct (x) — 0.030 (0.75) for steel EBF, 0.020 (0.75) for all others — x= 0.75
Building Period: per formula 12.8-7 — Ta = Ct*hn^x= 0.17 sec
by computer: (overrides above) — Tcomp= 0.00 sec — Cu= 1.7
see Table 12.8-1 for upper limit on calculated T: T <= Cu*Ta — T= 0.17 sec

Occupancy Category (OC): Secs. 1.5&11.5 & Tabs. 1-1&11.5-1 — OC= 2 — I= 1.00

Site Class Section 20.3 — Site Class D

Per sections 11.4.2 & 20.1, Site Class D may be used if soil properties are not known.

				User Defined
Site coefficients Fa & Fv	Sec. 11.4.3; Tables 11.4-1 & 11.4-2	Fa=	1.60	0.00
	(straight line interpolation used)	Fv=	2.40	0.00

Adjusted Maximum Spectral Response Parameters — Section 11.4.3
Sms = Fa x Ss= — Sms= 0.195g
Sm1 = Fv x S1= — Sm1= 0.132g

Design Spectral Response Acceleration Parameters — Section 11.4.4
11.4.5: Ts = Sd1 / Sds = 0.676 — Sds = 2/3 x Sms= — **Sds= 0.130g**
0.8*Ts = 0.541 — Sd1 = 2/3 x Sm1= — **Sd1= 0.088g**

Seismic Design Category Section 11.6 — per Table 11.6-1, A
per Table 11.6-2, B

per section 11.6, is S1>=0.75? **no, use more severe value from Tables 11.6-1 & 11.6-2, EXCEPT SDC only per Table 11.6-1 is allowed if all 4 exceptions are met**

Exc #1: In both orthogonal directions, is Ta < 0.8*Ts ? — no
Exc #2: In both orthogonal directions, is T used for drift < 1.0*Ts ? — no
Exc #3: Is formula 12.8-2 used to determine Cs ? — no
Exc #4: Are diaphragms rigid ? Or for flexible diaphragms, are the vertical elements of LFRS spaced closer than 40 ft apart ? — no

Are all exceptions met ? NO — **use more severe value from Tables 11.6-1 & 11.6-2**
Seismic Design Category — **SDC= B**

Equivalent Lateral Force Procedure Section 12.8
See Table 12.6-1 for selection of analysis procedure

			For Force Calculations	For Drift Calculations*
per formula 12.8-2		Cs= Sds / (R/I)	0.0651	0.0651
per formula 12.8-3 for T < T_L		Cs< Sd1 / (R/I) / T	<0.2628	<0.2628
per formula 12.8-5		Cs> 0.044 Sds I > 0.01	>0.0100	NA
(formula 12.8-5 Incorporates proposed change of Suppl. #2)				
Does 1614A.1.7 apply?	no	-	NA	NA
per formula 12.8-6, if S1>=0.6g	FALSE	Cs> 0.5 S1 / (R/I)	NA	NA
		Cs=	0.0651	0.0651

Ultimate Strength Base Shear for FORCE Calculations: Vult= 0.0651 W

Ultimate Strength Base Shear for DRIFT Calculations*: Vult= 0.0651 W
(* For drift calculation, section 12.8.6.2 allows use of T without limit of T <= Cu*Ta)

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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	12/5/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	ALZ

WIND LOAD DETERMINATION - ALTERNATE ALL HEIGHTS METHOD (2010 CBC 1609A.6)

Based on Section 6.5 - Method 2 of ASCE 7-05

$$p_{net} = q_s K_z C_{net} [I K_{zt}]$$ (Eqn 16A-36)

V = 110 mph
q_s = 31.0 psf
I = 1.00
q_h = 18.0 ft

Exposure: C
zg = 900 ft
α = 9.5
K_{zt} = 1.00
$K_{Z\text{-LEEWARD}}$ = 0.88

Openings: Enclosed
Internal Pressure: ◉ ADDED
○ SUBTRACTED

Note: Internal pressure is either added or subtracted to both leeward and windward.

SOUTH to NORTH

			WINDWARD FACE SOUTH				LEEWARD FACE NORTH				TOTAL
STORY	Z ft	$K_{Z\text{-WINDWARD}}$	C_{net}	p_{net}	A sq ft	Load lb	C_{net}	p_{net}	A sq ft	Load lb	LOAD lb
Base	10.5	0.85	0.43	11.3 psf	400	4532	-0.51	-13.9 psf	400	-5578	10111
					400	4532			400	-5578	**10,111**

NORTH to SOUTH

			WINDWARD FACE NORTH				LEEWARD FACE SOUTH				TOTAL
STORY	Z ft	$K_{Z\text{-WINDWARD}}$	C_{net}	p_{net}	A sq ft	Load lb	C_{net}	p_{net}	A sq ft	Load lb	LOAD lb
Base	10.5	0.85	0.43	11.3 psf	400	4532	-0.51	-13.9 psf	400	-5578	10111
					400	4532			400	-5578	**10,111**

Roof Lateral Component - South to North

Wind Direction: Wind perpendicular to ridge
Windward Roof Slope = Slope 4:12 (18°)-Condition 2
Leeward Roof Slope = Slope 4:12 (18°)-Condition 1

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Horiz. Load lb
Windward	17.0	0.87	-0.05	-1.4 psf	530	-221
Leeward	17.0	0.87	-0.66	17.8 psf	530	2920
						2699

Total Load South to North
12810 lbs.

Roof Lateral Component - North to South

Wind Direction: Wind perpendicular to ridge
Windward Roof Slope = Slope 4:12 (18°)-Condition 2
Leeward Roof Slope = Slope 4:12 (18°)-Condition 1

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Horiz. Load lb
Windward	17.0	0.87	-0.05	-1.4 psf	530	-221
Leeward	17.0	0.87	-0.66	17.8 psf	530	2920
						2699

Total Load North to South
12810 lbs.

Roof Uplift Component

Wind Direction: Wind perpendicular to ridge
Windward Roof Slope = Slope 4:12 (18°)-Condition 1
Leeward Roof Slope = Slope 4:12 (18°)-Condition 1

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Vert. Load lb
Windward	17.0	0.87	-0.73	-19.7 psf	530	-9942
Leeward	17.0	0.87	-0.66	-17.8 psf	530	-8988
						-18930

Total Roof Uplift Load
18930 lbs.

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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	12/5/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	AL3

WIND LOAD DETERMINATION - ALTERNATE ALL HEIGHTS METHOD (2010 CBC 1609A.6)

Based on Section 6.5 - Method 2 of ASCE 7-05

$$p_{net} = q_s K_z C_{net} [I K_{zt}]$$ (Eqn 16A-36)

V = 110 mph
q_s = 31.0 psf
I = 1.00
q_h = 18.0 ft

Exposure: C
zg = 900 ft
α = 9.5
K_{zt} = 1.00
$K_{Z\text{-LEEWARD}}$ = 0.88

Openings: Enclosed
Internal Pressure: ◉ ADDED ○ SUBTRACTED

Note: Internal pressure is either added or subtracted to both leeward and windward.

EAST to WEST

			WINDWARD FACE EAST				LEEWARD FACE WEST				TOTAL
STORY	Z ft	$K_{Z\text{-WINDWARD}}$	C_{net}	p_{net}	A sq ft	Load lb	C_{net}	p_{net}	A sq ft	Load lb	LOAD lb
Breezeway	12.0	0.85	0.43	11.3 psf	250	2833	-0.51	-13.9 psf	250	-3487	6319
Base	10.5	0.85	0.43	11.3 psf	200	2266	-0.51	-13.9 psf	200	-2789	5055
					450	5099			450	-6276	**11,374**

WEST to EAST

			WINDWARD FACE WEST				LEEWARD FACE EAST				TOTAL
STORY	Z ft	$K_{Z\text{-WINDWARD}}$	C_{net}	p_{net}	A sq ft	Load lb	C_{net}	p_{net}	A sq ft	Load lb	LOAD lb
Breezeway	12.0	0.85	0.43	11.3 psf	250	2833	-0.51	-13.9 psf	250	-3487	6319
Base	10.5	0.85	0.43	11.3 psf	200	2266	-0.51	-13.9 psf	200	-2789	5055
					450	5099			450	-6276	**11,374**

Roof Lateral Component - East to West

Wind Direction: Wind perpendicular to ridge
Windward Roof Slope = Slope 4:12 (18°)-Condition 2
Leeward Roof Slope = Slope 4:12 (18°)-Condition 1

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Horiz. Load lb
Windward	17.0	0.87	-0.05	-1.4 psf	235	-98
Leeward	17.0	0.87	-0.66	17.8 psf	235	1295
						1197

Total Load East to West
12571 lbs.

Roof Lateral Component - West to East

Wind Direction: Wind perpendicular to ridge
Windward Roof Slope = Slope 4:12 (18°)-Condition 2
Leeward Roof Slope = Slope 4:12 (18°)-Condition 1

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Horiz. Load lb
Windward	17.0	0.87	-0.05	-1.4 psf	235	-98
Leeward	17.0	0.87	-0.66	17.8 psf	235	1295
						1197

Total Load West to East
12571 lbs.

Roof Uplift Component

Wind Direction: Wind perpendicular to ridge
Windward Roof Slope = Slope 4:12 (18°)-Condition 1
Leeward Roof Slope = Slope 4:12 (18°)-Condition 1

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Vert. Load lb
Windward	17.0	0.87	-0.73	-19.7 psf	235	-4408
Leeward	17.0	0.87	-0.66	-17.8 psf	235	-3985
						-8393

Total Roof Uplift Load
8393 lbs.

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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	12/5/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	A14

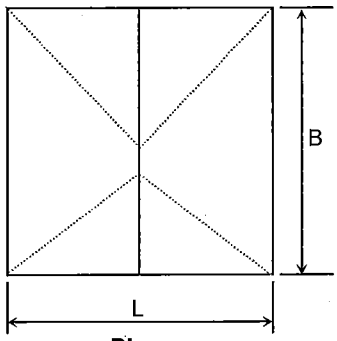
WIND LOADING ANALYSIS - Roof Components and Cladding MAIN ROOF COMPONENTS

ASCE 7-05 Method 2: Analytical Procedure (Section 6.5.12.4)
Any Height with Gable Roof θ ≤ 45° or Monoslope Roof θ ≤ 30°

ROOF DATA:

Roof Type: Figure 6-11C: Gable / Ridge 7° < θ ≤ 27°

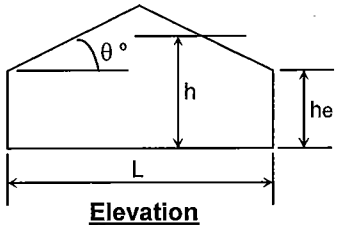
Bldg. Classification =	II	Table 1-1 Occupancy Category
Mean Roof Ht., h =	18.0 ft	use height of eve if θ ≤ 10°
Roof Angle, θ =	18.4	deg.
Building Width =	38.0 ft	normal to ridge
Building Length =	80.0 ft	parallel to ridge
Enclosure	Enclosed	Section 6.2 & Figure 6-5



Plan

WIND PRESSURE DATA:

Wind Speed, V =	110 mph	Wind Map, Figure 6-1
Exposure Category =	C	Sect. 6.5.6
I =	1.00	Table 1 (Importance factor)
Kh =	0.88	Kh = 2.01*(h/zg)^(2/a); (h = 15 ft min)
Topo. Factor, Kzt =	1.00	Sect. 6.5.7 & Figure 6-4
Direct. Factor, Kd =	0.85	Table 6-4
α =	9.50	Table 6-2
zg =	900	Table 6-2
qh =	23.2 psf	qh = 0.00256*Kh*Kzt*Kd*V^2*I (Sect. 6.5.10, Eq. 6-15)



Elevation

External Pressure Coefficients, GCp (Figures 6-11~17):

	a	b	c	d
Zone 1 2 3+	0.5	0.7	-0.2	0.3
Zone 1-	-0.9	-1.0	0.1	-0.8
Zone 2-	-1.7	-2.2	0.5	-1.2
Zone 3-	-2.6	-3.2	0.6	-2.0
Zone 1 2 OH	-2.2	-2.2	0.0	-2.2
Zone 3 OH	-3.7	-4.9	1.2	-2.5

GCp = **a** (at Ae ≤ 10 sf)
GCp = **b** + **c** LOG(Ae)
GCp = d (at Ae ≥ 100 sf)

Positive & Negative Internal Pressure Coefficients, GCpi (Figure 6-5):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

Design Net External Wind Pressures (Sect. 6.5.12.4):

For h ≤ 60 ft: p = qh*[(GCp) - (+/-GCpi)] (psf)
For h > 60 ft: p = q*(GCp) - qi*(+/-GCpi) (psf)

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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	12/5/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	AL5

WIND LOADING ANALYSIS - Roof Components and Cladding MAIN ROOF COMPONENTS

ASCE 7-05 Method 2: Analytical Procedure (Section 6.5.12.4)
Any Height with Gable Roof $\theta \leq 45°$ or Monoslope Roof $\theta \leq 30°$

Wind Load Tabulation for Roof Components & Cladding

Component	Ae sq ft		p = Net Design Pressures (psf)					
			Zone 1 2 3+	Zone 1-	Zone 2-	Zone 3-	Zone 1 2 OH	Zone 3 OH
Roof Element	101	GCp	0.30	-0.80	-1.20	-2.00	-2.20	-2.50
		pi+	**2.8**	**-22.8**	**-32.1**	**-50.6**	**-51.1**	**-58.1**
		pi-	**11.1**	**-14.4**	**-23.7**	**-42.3**	**-51.1**	**-58.1**
Roof Element	35	GCp	0.39	-0.85	-1.43	-2.27	-2.20	-3.05
		pi+	**4.9**	**-23.8**	**-37.3**	**-57.0**	**-51.1**	**-70.8**
		pi-	**13.3**	**-15.5**	**-29.0**	**-48.6**	**-51.1**	**-70.8**
Roof Element	21	GCp	0.44	-0.87	-1.54	-2.41	-2.20	-3.31
		pi+	**5.9**	**-24.3**	**-39.9**	**-60.1**	**-51.1**	**-77.0**
		pi-	**14.3**	**-16.0**	**-31.6**	**-51.7**	**-51.1**	**-77.0**
		GCp						
		pi+						
		pi-						
		GCp						
		pi+						
		pi-						
		GCp						
		pi+						
		pi-						
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		pi+						
		pi-						
		GCp						
		pi+						
		pi-						
		GCp						
		pi+						
		pi-						

Notes:

1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 2 (edge) **a = 3.8 ft**
3. Width of Zone 3 (corner) **a = 3.8 ft**
4. For parapets in excess of 3 ft in height is provided around perimeter of roof with $\theta \leq 10$ degrees, Zone 3 shall be treated as Zone 2.
5. qz = qh when $h \leq 60$ ft and for all roof components
   qi = qh assumed for all cases per Sect. 6.5.12.4.2
6. Per Section 6.1.4.2, the minimum wind load for C&C shall not be less than 10 psf.
7. Reduced wind pressures for stepped (low) roofs on buildings per Figure 6-12 have not been considered

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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	12/5/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	AL6

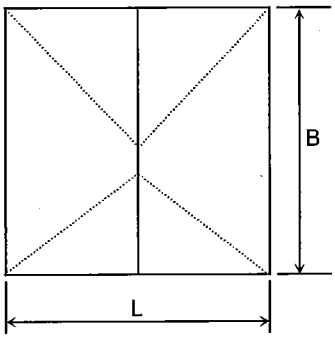
WIND LOADING ANALYSIS - Wall Components and Cladding WALL COMPONENTS

ASCE 7-05 Method 2: Analytical Procedure (Section 6.5.12.4)
Any Height with Gable Roof θ ≤ 45° or Monoslope Roof θ ≤ 30°

ROOF DATA:

Roof Type: Figure 6-11A: All Roof Types h ≤ 60 ft and θ > 10°

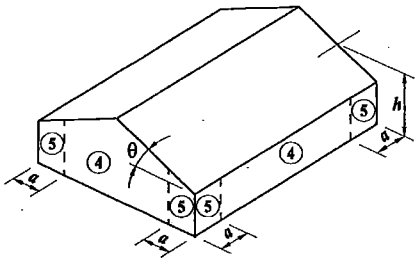
Bldg. Classification =	II	Table 1-1 Occupancy Category
Mean Roof Ht., h =	18.0 ft	use height of eve if θ ≤ 10°
Roof Angle, θ =	18.4	deg.
Building Width =	38.0 ft	normal to ridge
Building Length =	80.0 ft	parallel to ridge
Enclosure	Enclosed	Section 6.2 & Figure 6-5



Plan

WIND PRESSURE DATA:

Wind Speed, V =	110 mph	Wind Map, Figure 6-1
Exposure Category =	C	Sect. 6.5.6
I =	1.00	Table 6-1 (Importance factor)
Kh =	0.88	Kh = 2.01*(h/zg)^(2/a); (h = 15 ft min)
Topo. Factor, Kzt =	1.00	Sect. 6.5.7 & Figure 6-4
Direct. Factor, Kd =	0.85	Table 6-4
α =	9.50	Table 6-2
zg =	900	Table 6-2
qh =	23.2 psf	qh = 0.00256*Kz*Kzt*Kd*V^2*I; (where Kz = Kh) (Sect. 6.5.10, Eq. 6-15)



Isometric

External Pressure Coefficients, GCp (Figures 6-11~17):

	a	b	c	d
Windward Zone 4+	1.0	1.2	-0.2	0.7
Windward Zone 5+	1.0	1.2	-0.2	0.7
Leeward Zone 4-	-1.1	-1.3	0.2	-0.8
Leeward Zone 5-	-1.4	-1.8	0.4	-0.8

GCp = **a** (at Ae ≤ 10 sf)
GCp = **b** + **c** LOG(Ae)
GCp = **d** (at Ae ≥ 500 sf)

Positive & Negative Internal Pressure Coefficients, GCpi (Figure 6-5):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

Design Net External Wind Pressures (Sect. 6.5.12.4):

For h ≤ 60 ft: p = qh*[(GCp) - (+/-GCpi)] (psf)
For h > 60 ft: p = q*(GCp) - qi*(+/-GCpi) (psf) Windward, where q = qz and qi = qh
For h > 60 ft: p = q*(GCp) - qi*(+/-GCpi) (psf) Leeward, where q = qh and qi = qh

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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	12/5/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	AL7

WIND LOADING ANALYSIS - Wall Components and Cladding WALL COMPONENTS

ASCE 7-05 Method 2: Analytical Procedure (Section 6.5.12.4)
Any Height with Gable Roof θ ≤ 45° or Monoslope Roof θ ≤ 30°

Wind Load Tabulation for Wall Components & Cladding

Component	Ae	Z	Kz (windward)		p = Net Design Pressures (psf)			
					Windward		Leeward	
					Zone 4+	Zone 5+	Zone 4-	Zone 5-
				GCp				
				pi+				
				pi-				
Wall Stud at 1st	14 sq-ft	11 ft	0.88	GCp	0.97	0.97	-1.07	-1.35
				pi+	**18.4**	**18.4**	**-29.1**	**-35.5**
				pi-	**26.8**	**26.8**	**-20.8**	**-27.1**
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				

Notes:

1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 5 (edge) **a = 3.8 ft**
3. Evaluate parapets at z = top of parapet
4. qz = qh when h ≤ 60 ft and for all roof components
   qi = qh assumed for all cases per Sect. 6.5.12.4.2
5. Per Section 6.1.4.2, the minimum wind load for C&C shall not be less than 10 psf.
6. Reduced wind pressures for stepped (low) roofs on buildings per Figure 6-12 have not been considered
7. GCp reduced by 10% where h ≤ 60 ft and θ ≤ 10° per Figure 6-11A, note 5

LATERAL ANALYSIS (Admin bldg)

BLDG WT

ROOF (15 PSF) [40'x38' + 20'x45' + 26'x38'] = 51.1 k
WALLS (45PSF) [2x76' + 4x41'] (10.5'/2) = 74.7 k
MISC = 1.2 k

127.0 k => TOTAL BLDG WT

Seismic Force

$V_{base} = 0.0651\ W_T = 0.0651\ (127k) = 8.26k$ (ult.) (in both directions)

WIND Force

$V_{WIND} = 10111\# + 2699\# = 12810\# \approx 20k$ (N/S Direction)
$= 11374\# + 1197\# = 12571\# \approx 13k$ (E/W Direction)
(ASD)

∴ Wind Governs

$V_{WIND} = 20k / (38' \times 40' + 45' \times 14' + 38' \times 26') = 6.37$ PSF (N/S)

$13k / (38' \times 40' + 45' \times 14' + 38' \times 26') = 4.14$ PSF (E/W)

* In E/W direction, analysis assumes 1/2 of wind load applied to breezeway walls to account for additional wind effects.

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JOB _____
JOB NO. _____ DATE _____
CLIENT _____ BY _____ SHEET NO. AL9

LATERAL ANALYSIS

COLLECTOR DESIGN

N/S DIRECTION

LINE A

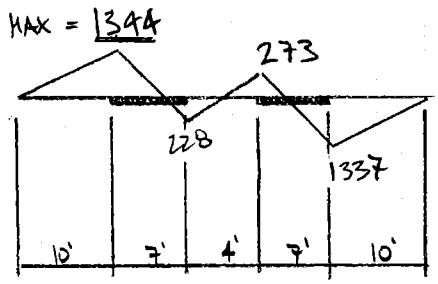
$F_{LINE} = 5.1$ K

$L_{DIAPH} = 38'$

$v_{DIAPH} = 134$ PLF

$L_{wall} = 14'$

$v_{wall} = 364$ PLF



LINE B

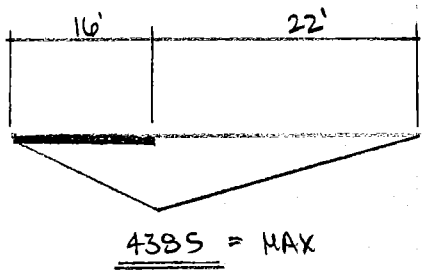
$F_{LINE} = 6.8$ K

$L_{DIAPH} = 45'$

$v_{DIAPH} = 151$ PLF

$L_{WALL} = 16'$

$v_{WALL} = 425$ PLF



LINE C

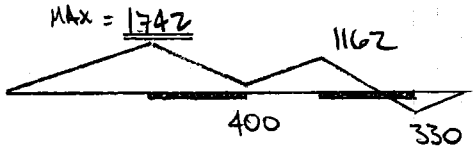
$F_{LINE} = 4.9$ K

$L_{DIAPH} = 45'$

$v_{DIAPH} = 109$ PLF

$L_{WALL} = 19'$

$v_{WALL} = 237$ PLF



⇒ use ½" Sure-Board @ all shear walls
w/ #8 SMS @ 6"cc EF (12"cc FP) CAP = 562 PLF

LATERAL ANALYSIS

COLLECTOR DESIGN

N/S DIRECTION (CONT'D)

LINE D

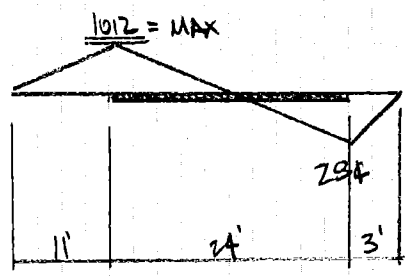
F_{LINE} = 3.5 k

L_{DIAPH} = 38'

v_{DIAPH} = 92 PLF

L_{WALL} = 24'

v_{WALL} = 146 PLF



E/W DIRECTION

LINE 1

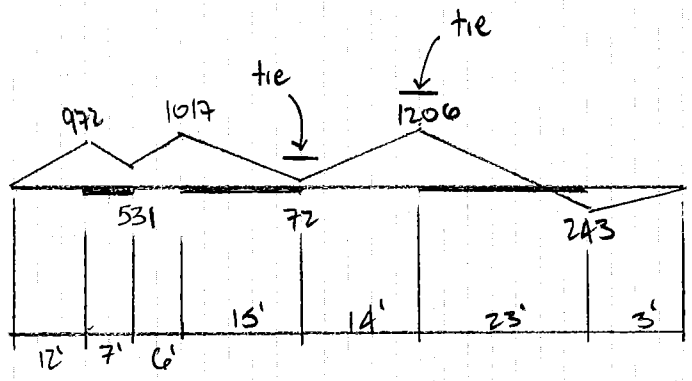
F_{LINE} = 6.5 k

L_{DIAPH} = 80'

v_{DIAPH} = 81 PLF

L_{WALL} = 45'

v_{WALL} = 144 PLF



LINE 2

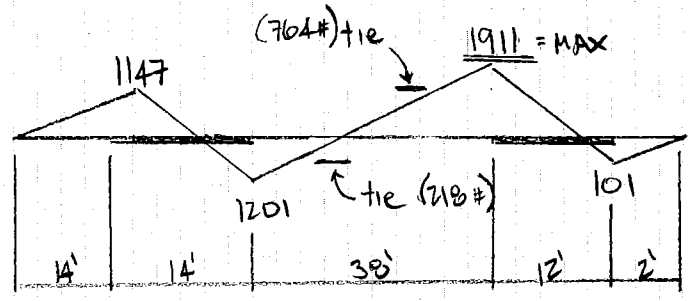
F_{LINE} = 6.5 k

L_{DIAPH} = 80'

v_{DIAPH} = 81 PLF

L_{WALL} = 26'

v_{WALL} = 250 PLF



⇒ Use ½" Sure-Board @ all shear walls
w/ #8 SMS @ 6" o.c. EF (12" o.c. FF) CAP = 562 PLF

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JOB ____ JOB NO. ____ DATE ____ CLIENT ____ BY ____ SHEET NO. AL11

LATERAL ANALYSIS

COLLECTOR DESIGN

FORCE TRANSFER @ ELEV CHANGE

Force @ LINE 1 = 1206 # (MAX)

ELEV DIFF = 14.5' - 10.5' = 4'

M = 1206 # (4') = 4824 #' = 57.89 K"

⇒ use 4-6"x18ga studs (J4)

M_n = 16.68 K" x 4 = 66.72 K"

Force @ Line 2 = 764 # (MAX)

ELEV DIFF = 4'

M = 764 # (4') = 3056 #' = 36.67 K"

⇒ use 3-6"x18ga studs (J3)

M_n = 16.68 K" x 3 = 50.04 K"

TIE @ ROOF SLOPE CHANGE

MAX Force = 1849 # - 82PLF x 3' = 1603#

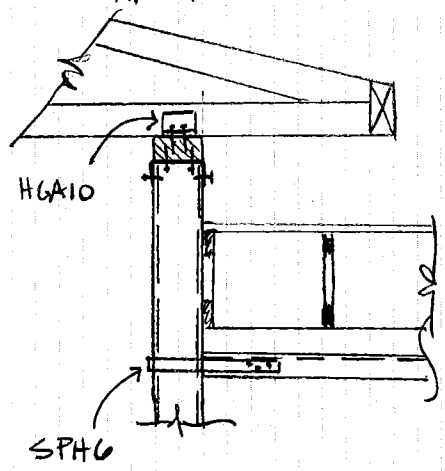
⇒ USE Simpson MSTA24 strap

CAP = 1640

⇒ use Simpson SPH6 tie @ low wall CAP = 1490 #

⇒ use Simpson HGA10 clip @ high truss (ea side)

CAP = 1165 # x 2



LATERAL ANALYSIS

FORCE DISTRIBUTION

N/S DIRECTION

$W_1 = 6.37 \text{ PSF} \times 38' = 242 \text{ PLF}$ $\quad W_2 = 6.37 \text{ PSF} (45') = 287 \text{ PLF}$

LINE A - $242 \text{ PLF} (38'/2 + 2.0') = 5.1 \text{ K}$

From high roof

LINE B - $242 \text{ PLF} (40')/2 = 4.8 \text{ K} + 287 \text{ PLF} (14')/2 = 2.0 \text{ K} \Rightarrow F_T = 6.8 \text{ K}$

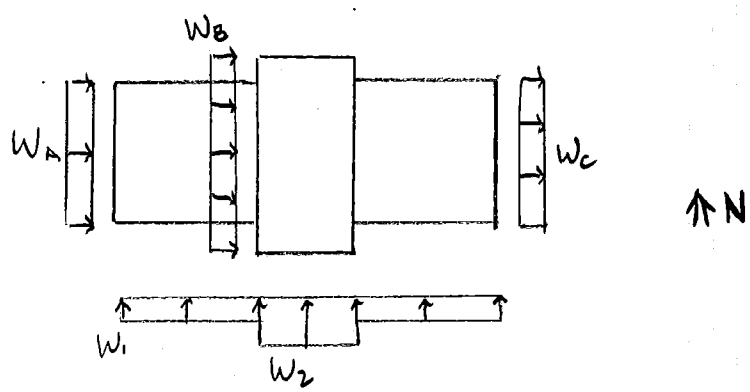
LINE C - $242 \text{ PLF} (24')/2 = 2.9 \text{ K} + 287 \text{ PLF} (14')/2 = 2.0 \text{ K} \Rightarrow F_T = 4.9 \text{ K}$

LINE D - $242 \text{ PLF} (24'/2 + 2.0') = 3.4 \text{ K}$

E/W DIRECTION

$W_1 = 3.19 \text{ PSF} \times 40' = 128 \text{ PLF}$ $\quad W_B = 3.19 \text{ PSF} \times 14' = 45 \text{ PLF}$ $\quad W_C = 3.19 \text{ PSF} \times 24' = 83 \text{ PLF}$

LINE 2/3 - $121 \text{ PLF} (38'/2) + 45 \text{ PLF} (23'/2) + 77 \text{ PLF} (38'/2) = 5.0 \text{ K}$



LATERAL ANALYSIS

DIAPHRAGM DESIGN

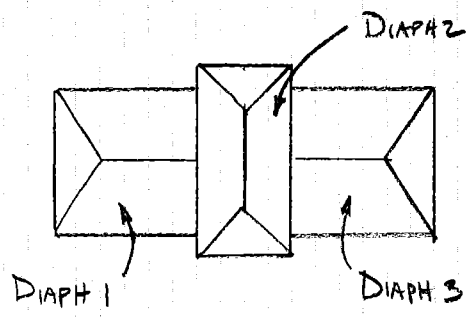
CHECK DIAPHRAGM RATIOS ⇒ max length to width ratio is 3:1 (unblocked)

DIAPH 1 = 40'/38' = 1.05

DIAPH 2 = 45'/20' = 2.25

DIAPH 3 = 38'/26' = 1.46

⇒ All diaphragms ok ✓



N/S DIRECTION

W_1 = 242 PLF W_2 = 287 PLF

DIAPH 1/3 - v_1 = 242 PLF × 40'/2 = 4.8 k/38' = 127 PLF

2 - v_2 = 287 PLF × 20'/2 = 2.9 k/45' = 64 PLF

⇒ Use 3/4" sh'ty (unblocked) w/ 10d @ 6" cc EN, 10d @ 12" cc FM

CHORD FORCES - DIAPH 1 - 242 PLF × 40'²/8×38' = 1273 #

DIAPH 2 - 287 PLF × 20'²/8×45' = 319 #

E/W DIRECTION

W_1 = 128 PLF W_2 = 45 PLF W_3 = 83 PLF

DIAPH 1 - 128 PLF × 38'/2 = 2.4 k/40' = 60 PLF

2 - 45 PLF × 45'/2 = 1.0 k/20' = 50 PLF

3 - 83 PLF × 38'/2 = 1.6 k/26' = 62 PLF

CHORD FORCES - DIAPH 1 - 128 PLF × 38'²/8×40' = 578 #

2 - 45 PLF × 45'²/8×20' = 570 #

3 - 83 PLF × 38'²/8×26' = 576 #

LATERAL ANALYSIS

SHEAR WALL DESIGN (LC = 0.6D ± W)

N/S DIRECTION

LINE A - SW1 & SW2

$L_{WALL} = 7'$ $\quad d = 7' - 1' = 6'$

$F_{SW} = 364\ PLF \times 7' = 2548\#$

$OTM = 2548\#(10.5') = 26.8\ k'$

$DLRM = [(15\ PSF)(14'/2) + (45\ PSF)(10.5')](7')^2/2 = 14.1\ k'$

$T = \dfrac{0.6(14.15\ k') - 26.8\ k'}{6'} = -3.1\ k$

∴ Use Simpson S/HDU4
CAP = 3970#
(w/ 16ga studs)

LINE B - SW1

$L_{WALL} = 16'$ $\quad d = 15'$

$F_{SW} = 2.0'$ @ high roof
$\quad\quad 4.8$ @ low roof

$OTM = 2.0k(15.25') + 4.8k(10.5') = 80.9\ k'$

$DLRM = [(15\ PSF)(6') + (45\ PSF)(15.25')](16')^2/2 = 99.4\ k'$

$T = \dfrac{0.6 \times (99.4\ k') - 80.9\ k'}{15'} = -1.38\ k$

∴ Use Simpson S/HDU4
CAP = 3970#
(w/ 16ga studs)

LATERAL ANALYSIS

SHEAR WALL DESIGN (LC: 0.6D ± W)

N/S DIRECTION

LINE C - SW2 (worst case)

$L_{WALL} = 10'$ $d = 9'$

$F_{SW} =$ 105 PLF x 10' = 1.05k @ high roof
153 PLF x 10' = 1.53k @ low roof

2.0k/19'
2.9k/19'

$OTM = 1.05k(15.25') + 1.53k(10.5') = 32.1k'$

$DLRM = [(15\,PSF)(6') + (45\,PSF)(15.25')](10')^2/2 = 38.8k'$

$T = \frac{0.6(38.8k') - 32.1k'}{9'} = -0.98k$ ∴ use Simpson S/HDU4

LINE D - SW1

$L_{WALL} = 24'$ $d = 23'$

$F_{SW} = 3.5k$

$OTM = 3.5k(10.5') = 36.75k'$

$DLRM = [(15PSF)(7') + (45\,PSF)(10.5')](23')^2/2 = 152.7k'$

$T = \frac{0.6(152.7k') - 36.8k'}{23'} = 2.4k$ ∴ No overturning

⇒ Worst case T = 3.4k ⇒ TRY #8 SMS — CAP @ 16ga stud "Post"

needed = 3.4k/303# = 11 screws

$H_{WALL} = 10.5' \times 12 = 126''$ #8 SMS @ 6" cc ok ✓

126"/6" = 21 spaces. (# of screws in 10.5' = 20)

use 2-16ga studs as "Post"

LATERAL ANALYSIS

SHEAR WALL DESIGN

E/W DIRECTION

LINE 1 - SW1 (worst case)

$L_{WALL} = 7'$ $d = 6'$

$F_{WALL} = 144\ PLF \times 7' = 1.01\ k$

$OTM = 1.01k(10.5') = 10.6\ k'$

$DLRM = [(15PSF) \times 38'/2 + (45\ PSF)(10.5')](15')^2/2 = 85.2\ k'$

$T = \frac{0.6(85.2k') - 10.6k'}{6'} = 6.75\ k$ ∴ No overturning

LINE 2 - SW2 (worst case)

$L_{WALL} = 12'$ $d = 11'$

$F_{WALL} = 250\ PLF \times 12' = 3.0\ k$

$OTM = 3.0k(10.5') = 31.5\ k'$

$DLRM = [(15PSF)(38'/2) + (45\ PSF)(10.5')](12')^2/2 = 54.5\ k'$

$T = \frac{0.6(54.5k') - 31.5k'}{11'} = 0.109\ k$ ∴ No overturning

WALL DESIGN

TYPICAL STUDS (Zone 4 & 5)

$H_T = 10.5'$

$P_{DL} = 15\ PSF\ (32''/12)(38'/2) = 760\#$

$P_{LL} = 20\ PSF\ (32''/12)(38'/2) = 1014\#$

$W_{WIND} = 27.1\ PSF\ (16/12) = 36\ PLF$

$V = 36\ PLF \times 10.5'/2 = 189\#$

$M = 36\ PLF \times (10.5')^2/8 = 496\#' = 5.95\ k''$ ⇒ TRY 6" x 18"ga studs

$I_{REQ'D} = 675\ (36\ PLF)(10.5')^3/29000\ ksi = 0.97\ in^4$ $M_a = 16.68\ k''$

bridging @ 4' cc $I_x = 2.32\ in^4$

$KL/r = 1.0 \times 4' \times 12/0.576'' = 83.3$ $< 4.71\sqrt{\frac{29000\ ksi}{33\ ksi}} = 139.6$

$F_e = \frac{\pi^2 \times 29000}{(83.3)^2} = 41.2\ ksi$ ⇒ $F_{cr} = \left(0.658^{33/41.2}\right) 33\ ksi = 23.6\ ksi$

$P_n/\Omega = 23.6\ ksi \times 0.447\ in^2/1.67 = 6.32\ k$

$\frac{1.774\ k}{6.32\ k} + \frac{5.95\ k''}{16.68\ k''} = 0.64 < 1.0$ ⇒ use 6" x 18ga studs

BUILT-UP 'POST' CAPACITY

J1 - 6.32 k

J2 - 12.64 k

J3 - 18.96 k

J4 - 25.28 k

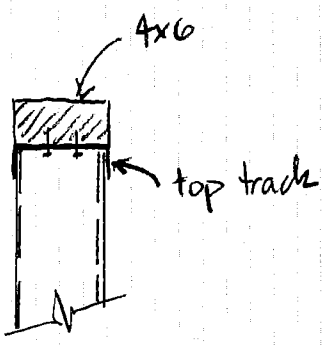
WALL DESIGN

TOP PLATE DESIGN

Max chord force = 1273 #, typ 580 #

Max Collector force = 4385 # @ LINE B, typ 1742 #

Top plate will be a 4x6 on top of a 6"x 18ga track



@ LINE B use MST 37 strap w/ 42-16d nails

CAP = 5080 #

@ typical splice use MSTA30 strap w/ 22-10d nails.

CAP = 2050 #

CHECK AXIAL STRESS

Collector MAX Force = 4385 #

$F_t' = 675 \text{ PSI } (1.6) = 1080 \text{ PSI}$

$f_t = 4385 \# / 19.25 \text{ in}^2 = 228 \text{ PSI} \ll F_t'$ ∴ 4x6 PL OK ✓

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JOB _____
JOB NO. _____ DATE _____
CLIENT _____ BY _____ SHEET NO. AW3

TYPICAL BRICK SUPPORT (Admin bldg)

Brick veneer needs to be supported @ door & window opngs.

Check typ door/window opng

SPAN = 6'-6' max

BRICK HEIGHT = 3'-0" max BRICK WT = 112 PCF x 3 5/8" = 34 PSF

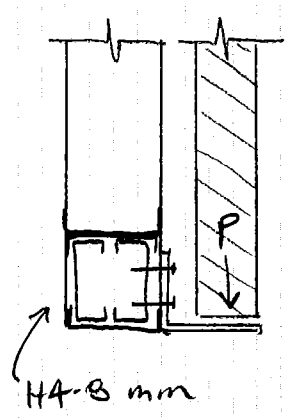
W = (1.2 x 34 PSF) 3.0' = 122 PLF (102 PLF @ ASD)

V = 397 #

M = 644 #'

$Z_{req'd}$ = 644 #' (12) / 0.9 x 36 ksi = 0.239 in^3

$I_{req'd}$ = 675 (102 PLF) (6.5)^3 / 29000 ksi = 0.652 in^4



⇒ USE L6x4x3/8 (LLH)

P (per foot) = 122 #

M = 122 # (4") = 488 #"

$t_{req'd} = \left(\frac{4(488\#")}{0.75 \times 36\,ksi \times 12"} \right)^{1/2} = 0.08"$ ⇒ 3/8" angle ok ✓

Fasteners → #10 SHS @ 12" cc CAP = 263 # @ 18ga studs.

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JOB _____
JOB NO. _____ DATE _____
CLIENT _____ BY _____ SHEET NO. AW4

WALL DESIGN

TYP WINDOW HEADER

SPAN = 6'-6" max

W_{WIND} = 27 PSF (4.17') = 113 PLF

V = 367 #

M = 597 #'

$I_{reqd} = 675(113\,PLF)(6.5')^3 / 29000 = 0.722$ ⇒ Use min HA-8 min

ok by inspection

TYP JAMB STUDS

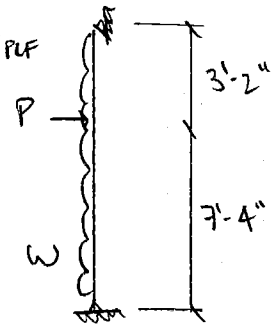
HT = 10.5'

P = 280 # ⇒ $W_{eq} = 8(280)(3.17')(7.33')/10.5^3 = 45$ PLF

W = 27 PSF (16/12) = 36 PLF

V =

$M = \frac{280\#(3.17')(7.33')}{10.5'} + \frac{36\,PLF(7.33')(10.5-7.33')}{2}$



M = 1038 #'

$I_{reqd} = 675(36\,PLF + 45\,PLF)(10.5')^3 / 29000\,ksi = 2.18\ in^4$ ⇒ 2-6"x18ga studs

I = 2.316 x 2

TYP SILL

SPAN = 6'-6"

W_{wind} = 27 PSF (3.67') = 99 PLF

V = 322 #

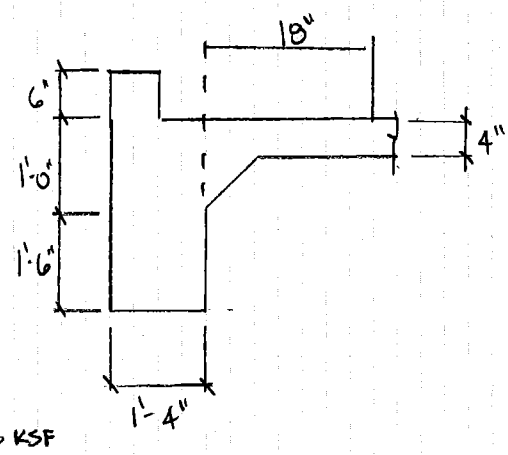
M = 522 #'

$I_{reqd} = 675(99\,PLF)(6.5')^3 / 29000 = 0.632$ ⇒ 2-6"x18ga studs ok✓

FOUNDATION DESIGN

Bearing Pressure = 2300 PSF (D+L)

Footing + part of slab resist overturning



$W_{DL} = (4" \times 18" + 16" \times 30") \frac{150 \text{ PCF}}{144} = 575 \text{ PLF}$

check 12" section of ftg

$q_{max} = 2300 \text{ PSF}$

$l^2_{max} = 6 A_s a d / q_{max} = 0.31 in^2 (6)(1.96)(27") / 2.3 \text{ KSF}$

$l_{max} = (42.8)^{1/2} = 6.54'$

$P_{allow} = l_{max} \times q_{max} = 6.54' \times 2.3 \text{ KSF} = 15.0 \text{ k}$ ⇒ use pad footings when reactions are more than 15.0 k (DOWN)

$P^2 = 8 A_s a d W_{DL} = 8(0.31 in^2)(1.96)(27")(0.575 \text{ PLF})$

$P^2 = 67.758$ ⇒ $P = 8.23 \text{ k}$ ⇒ use pad footings when reactions are more than 8.23 k (uplift)

⇒ Max uplift = 3.1 k ∴ No pad ftgs req'd.

AA-1

Report Number: 0126
Issued: 1/2010
Expires: 2/2012
Revised: 02/08/2011

TABLE 1 - NOMINAL AND ALLOWABLE SHEAR RESISTANCE TO WIND OR SEISMIC FORCES AND DISPLACEMENT (inches) FOR SHEAR WALLS WITH SURE-BOARD® SERIES 200 STRUCTURAL PANELS ATTACHED TO LIGHT GAGE STEEL C-STUDS AT 24" O.C. WITH SCREWS (pounds per foot) [1]

STEEL FRAMING	FASTENER SPACING AT PANEL EDGES INCHES ON CENTER [6]											
	6			4			3			2		
Minimum Gage[5]	**Vn** 2,3,4,7 (plf)	**Vasd** 2,3,8 (plf)	**ΔVasd** 9 (inch)	**Vn** 2,3,4,7 (plf)	**Vasd** 2,3,8 (plf)	**ΔVasd** 9 (inch)	**Vn** 2,3,4,7 (plf)	**Vasd** 2,3,8 (plf)	**ΔVasd** 9 (inch)	**Vn** 2,3,4,7 (plf)	**Vasd** 2,3,8 (plf)	**ΔVasd** 9 (inch)
20 (0.033 inch)	**1,085**	**434**	**0.21**	**1,545**	**618**	**0.21**	**1,730**	**692**	**0.24**	**1,915**	**766**	**0.26**
	1,543 [10]	**617**	**0.17**	**2,211** [10]	**885**	**0.22**	**2,486** [10]	**977**	**0.22**	**2,537** [10]	**906**	**0.16**
18 (0.043 inch)	**1,405** [10]	**562**	**0.24**	**1,925** [10]	**770**	**0.23**	**2,821** [10]	**1,126**	**0.25**	**2,989** [10]	**1,196**	**0.21**
16 (0.054 inch)	**1,697**	**678**	**0.25**	**2,306**	**922**	**0.25**	**2,957** [10]	**1,092**	**0.26**	**3,647** [10]	**1,253**	**0.28**
14 (0.071 inch)	-------	-------	-----	-------	-------	-----	-------	------	-----	**3,292**	**1,257**	**0.24**
14 (0.071 inch) 2-Sided *Fasteners 6" O.C. into intermediate framing	-------	-------	-----	-------	-------	-----	-------	------	-----	**4,635***	**1,700**	**0.22**

For SI: 1 inch = 25.4 mm, 1 lb/linear = 0.0146 N/mm.

[1] These values are for short-term loads due to wind or earthquake.

[2] The screws are described in Section 2.2.2 and are installed in accordance with Section 2.4 of IAPMO ES ER-0126.

[3] Tabulated values listed in tables are for panels applied to one side or two sides of a wall.

[4] For load and resistance factor design (LRFD) loads, the tabulated **Vn** load values must be multiplied by the resistance factor Φ = 0.55 for Seismic / 0.60 for Wind.

[5] Section 2.2.3 in IAPMO ES ER-0126, describes minimum base metal thickness associated with gages.

[6] All panel edges must be blocked. Panels can be installed vertically or horizontally. Fasteners must be spaced a maximum of 12 inches on center along intermediate framing members, except as noted in Table 1 above.

[7] **Vn** = Nominal Strength.

[8] **Vasd** = ASD Design Load.

[9] Δ **Vasd**= Deflection at **Vasd** design Load.

[10] Nominal strength is based on double c-stud collector to be designed using one gage thicker than the framing material used in shear wall.

Project Description: (Maintenance bldg)

The expansion project for the VA National Cemetery in Zachary, Louisiana consists of three one-story structures and one shelter/canopy structure. The project is located in a hurricane zone with 110mph basic wind speed; requiring hurricane ties of all roof elements down to the foundation at all the buildings.

The Administration building is a one story building. The roof consists of prefabricated wood trusses with plywood sheathing supported by light-gage metal stud exterior bearing walls. The walls use Sureboard for shear resistance and bear on continuous footings.

The maintenance building is a pre-manufactured one-story metal building. The roof is part of the pre-manufactured design and consists of Z purlins and WF beams that span to WF columns. Rod bracing provides lateral resistance and it is also designed by the manufacturer. The columns will bear on spread footings and the building has a continuous footing all around. There is a bulk storage structure that is considered part of the maintenance building. Roof structure consists of wood framing with plywood sheathing. Wood stud non-bearing walls sit on top of 11' tall concrete walls, which sit on continuous footings.

There is also a shelter structure that has a pre-manufactured roof and lateral system comprised of tube members. The columns will bear on spread footings that will be connected with grade beams.

Design Criteria:

Building Type: Occupancy Category: I - Bulk Storage, Committal Shelter Structure
II – Administration and Maintenance Buildings

Design Codes: 2009 International Building Code
ASCE 7-05: Minimum Design Loads for Buildings and Other Structures, including *Supplement 1*
AISC 360-05: Specification for Structural Steel Buildings (13th Edition AISC)
ACI 318-08: Building Code Requirements for Structural Concrete
AF&PA 2005 NDS and 2008 SDPWS

Wind Criteria: 110 mph, Exposure C $I_w = 1.00$ $GC_{pi} = \pm 0.18$ (Enclosed Structure)

Seismic Criteria: Site Class: D $S_S = 0.122$ $S_1 = 0.055$ $S_{DS} = 0.130$ $S_{D1} = 0.088$

Seismic Design Category: B $I = 1.00$ $I_p = 1.0$

Lateral Force-Resisting System:

Administration Building
Light-gage metal stud bearing shear walls
$R = 6.5$ $\Omega_o = 3.0$ $C_d = 4.0$

Bulk Storage Building
Wood bearing and concrete shear walls
$R = 4.0$ $\Omega_o = 2.5$ $C_d = 4.0$

Maintenance Building/Committal Shelter
Light-gage metal stud bearing shear walls
$R = 3.5$ $\Omega_o = 3.0$ $C_d = 3.0$

Foundation Criteria:

Soils Report #870 CM 3022 by W.L.Burle Engineers, P.A., dated 2/15/2011

Allowable Bearing Pressures: (Non-structural elements)	DL	2300 psf
	DL + LL	2300 psf
	DL + LL + EQ/1.4	3067 psf
Allowable Lateral Pressures:	Retaining At-Rest Pressure	55 pcf
	Passive Pressure	300 pcf
	Coefficient of Friction	0.30

A: Roof Loads (Administration Bldg)

	Trusses	seismic weight
Asphalt Shingles	2.5	2.5
3/4" shtg	2.3	2.3
Trusses @ 32"cc	3.0	3.0
Insulation	0.5	0.5
sprinklers	1.0	1.0
MEP	2.0	2.0
ceilings	2.2	2.2
misc	1.5	1.5
DPO - dead load	15.0 psf	15.0 psf
LLR - reducible roof LL	20.0 psf	0.0 psf
TL	35.0 psf	15.0 psf

B: Roof Loads (Maintenance Bldg)

	deck	seismic weight
metal roofing (20ga)	1.8	1.8
Z' purlins (6"x16ga @ 24"cc)	1.5	1.5
Insulation	3.0	3.0
beams	2.9	2.9
columns	1.0	1.0
sprinklers	1.0	1.0
MEP	2.0	2.0
ceilings	1.8	1.8
misc	1.0	1.0
DPO - dead load	16.0 psf	16.0 psf
LLR - reducible roof LL	20.0 psf	0.0 psf
TL	36.0 psf	16.0 psf

C: Roof Loads (Bulk Storage)

	Joists	seismic weight
metal roofing (18ga)	1.8	1.8
single-ply roof (w/ re-roof)	5.5	5.5
5/8" shtg	1.9	1.9
2x10's @ 24"cc	1.7	1.7
sprinklers	1.0	1.0
MEP	1.0	1.0
misc	0.9	0.9
DPO - dead load	12.0 psf	12.0 psf
LLR - reducible roof LL	20.0 psf	0.0 psf
TL	32.0 psf	12.0 psf

D: Roof Loads (Committal Shelter)

	deck	seismic weight
metal roofing (20ga)	1.8	1.8
Z' purlins (6"x16ga @ 24"cc)	1.5	1.5
beams	2.2	2.2
columns	1.0	1.0
sprinklers	1.0	1.0
MEP	2.0	2.0
misc	1.0	1.0
DPO - dead load	10.5 psf	10.5 psf
LLR - reducible roof LL	20.0 psf	0.0 psf
TL	30.5 psf	10.5 psf

E: Wall Loads

Wall Finish:	Light-Gage	Wood	Concrete
6"x16ga mtl studs @ 16"cc	1.5		
2x6 wood studs @ 16"cc		1.5	
12" concrete			150.0
Metal siding (both sides)		3.0	
1/2" shtg (both sides)		3.4	
3 5/8" brick veneer	35.0		
1/2" Sureboard	4.3		
5/8" gyp board	2.8	2.8	
misc	1.4	1.3	
DPO - non-composite DL	45.0 psf	12.0 psf	150.0 psf

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JOB _____
JOB NO. _____ DATE _____
CLIENT _____ BY _____ SHEET NO. MR1

MEZZANINE FRAMING DESIGN

TYPICAL JOISTS

SPAN = 12'-6"

$W = (12\ PSF + 60\ PSF)\ 16/12 = 96\ PLF$

$V = 600\ \#$

$M = 1875\ \#'$

$S_{req'd} = 1875\ \#'(12) / 0.6 \times 33\ ksi = 1.14\ in^3$

$I_{req'd} = 450\ (96\ PLF)(12.5')^3 / 29000\ ksi = 2.91\ in^4$ ⇒ use 8" x 18ga joists

$I = 4.633\ in^4$

$S = 1.158\ in^3$

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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	10/26/2011
CLIENT	Jacbos	BY	GAM	SHEET NO.	ML1

ASCE 7-05 (incl. supp. #1) Seismic Base Shear Calculation (Maintenance bldg)

Location: Zachary — Mapped MCE Spectral Acceleration (from Lat/Long using USGS program):
Height (hn): 14.0 ft — Latitude = 30.6439 — Longitude = -91.2567 — Ss= 0.122g
Regularity: regular — S1= 0.055g
Seismic Force-Resisting Sys. (Table 12.2-1): Wood and concrete shear walls — R= 3.5
Period Coefficients: 0.028 (0.8) for steel MRF, 0.016 (0.9) for concrete MRF — Ct= 0.028
listed as Ct (x) — 0.030 (0.75) for steel EBF, 0.020 (0.75) for all others — x= 0.8
Building Period: per formula 12.8-7 — Ta = Ct*hn^x= 0.23 sec
by computer: (overrides above) — Tcomp= 0.00 sec — Cu= 1.7
see Table 12.8-1 for upper limit on calculated T: T <= Cu*Ta — T= 0.23 sec
Occupancy Category (OC): Secs. 1.5&11.5 & Tabs. 1-1&11.5-1 — OC= 2 — I= 1.00

Site Class Section 20.3 — Site Class D
Per sections 11.4.2 & 20.1, Site Class D may be used if soil properties are not known.

				User Defined
Site coefficients Fa & Fv	Sec. 11.4.3; Tables 11.4-1 & 11.4-2	Fa=	1.60	0.00
	(straight line interpolation used)	Fv=	2.40	0.00

Adjusted Maximum Spectral Response Parameters — Section 11.4.3
Sms = Fa x Ss= — Sms= 0.195g
Sm1 = Fv x S1= — Sm1= 0.132g

Design Spectral Response Acceleration Parameters — Section 11.4.4
11.4.5: Ts = Sd1 / Sds = 0.676 — Sds = 2/3 x Sms= — **Sds= 0.130g**
0.8*Ts = 0.541 — Sd1 = 2/3 x Sm1= — **Sd1= 0.088g**

Seismic Design Category Section 11.6 — per Table 11.6-1, A
per Table 11.6-2, B
per section 11.6, is S1>=0.75? **no, use more severe value from Tables 11.6-1 & 11.6-2, EXCEPT SDC only per Table 11.6-1 is allowed if all 4 exceptions are met**
Exc #1: In both orthogonal directions, is Ta < 0.8*Ts ? — no
Exc #2: In both orthogonal directions, is T used for drift < 1.0*Ts ? — no
Exc #3: Is formula 12.8-2 used to determine Cs ? — no
Exc #4: Are diaphragms rigid ? Or for flexible diaphragms, are the vertical elements of LFRS spaced closer than 40 ft apart ? — no
Are all exceptions met ? NO — **use more severe value from Tables 11.6-1 & 11.6-2**
Seismic Design Category — **SDC= B**

Equivalent Lateral Force Procedure Section 12.8
See Table 12.6-1 for selection of analysis procedure

			For Force Calculations	For Drift Calculations*
per formula 12.8-2		Cs= Sds / (R/I)	0.0372	0.0372
per formula 12.8-3 for T < T_L		Cs< Sd1 / (R/I) / T	<0.1087	<0.1087
per formula 12.8-5		Cs> 0.044 Sds I > 0.01	>0.0100	NA
(formula 12.8-5 Incorporates proposed change of Suppl. #2)				
Does 1614A.1.7 apply?	no	-	NA	NA
per formula 12.8-6, if S1>=0.6g	FALSE	Cs> 0.5 S1 / (R/I)	NA	NA
		Cs=	0.0372	0.0372

Ultimate Strength Base Shear for FORCE Calculations: Vult= 0.0372 W

Ultimate Strength Base Shear for DRIFT Calculations*: Vult= 0.0372 W
(* For drift calculation, section 12.8.6.2 allows use of T without limit of T <= Cu*Ta)

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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	12/5/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	MLZ

WIND LOAD DETERMINATION - ALTERNATE ALL HEIGHTS METHOD (2010 CBC 1609A.6) (maintenance)

Based on Section 6.5 - Method 2 of ASCE 7-05

$$p_{net} = q_s K_z C_{net} [I K_{zt}]$$ (Eqn 16A-36)

V = 110 mph
q_s = 31.0 psf
I = 1.00
q_h = 18.0 ft

Exposure: C
zg = 900 ft
α = 9.5
K_{zt} = 1.00
$K_{Z\text{-LEEWARD}}$ = 0.88

Openings: Enclosed
Internal Pressure: (●) ADDED ( ) SUBTRACTED

Note: Internal pressure is either added or subtracted to both leeward and windward.

SOUTH to NORTH

			WINDWARD FACE SOUTH				LEEWARD FACE NORTH				TOTAL
STORY	Z ft	$K_{Z\text{-WINDWARD}}$	C_{net}	p_{net}	A sq ft	Load lb	C_{net}	p_{net}	A sq ft	Load lb	LOAD lb
Base	15.0	0.85	0.43	11.3 psf	475	5382	-0.51	-13.9 psf	475	-6624	12006
					475	5382			475	-6624	**12,006**

NORTH to SOUTH

			WINDWARD FACE NORTH				LEEWARD FACE SOUTH				TOTAL
STORY	Z ft	$K_{Z\text{-WINDWARD}}$	C_{net}	p_{net}	A sq ft	Load lb	C_{net}	p_{net}	A sq ft	Load lb	LOAD lb
Base	15.0	0.85	0.43	11.3 psf	475	5382	-0.51	-13.9 psf	475	-6624	12006
					475	5382			475	-6624	**12,006**

Roof Lateral Component - South to North

Wind Direction: Wind perpendicular to ridge
Windward Roof Slope = Slope ≤ 2:12 (10°)-Condition 2
Leeward Roof Slope = Slope ≤ 2:12 (10°)-Condition 1

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Horiz. Load lb
Windward	18.5	0.89	-0.28	-7.7 psf	225	-301
Leeward	18.5	0.89	-0.66	18.2 psf	225	709
						408

Total Load South to North
12415 lbs.

Roof Lateral Component - North to South

Wind Direction: Wind perpendicular to ridge
Windward Roof Slope = Slope ≤ 2:12 (10°)-Condition 2
Leeward Roof Slope = Slope ≤ 2:12 (10°)-Condition 1

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Horiz. Load lb
Windward	18.5	0.89	-0.28	-7.7 psf	225	-301
Leeward	18.5	0.89	-0.66	18.2 psf	225	709
						408

Total Load North to South
12415 lbs.

Roof Uplift Component

Wind Direction: Wind perpendicular to ridge
Windward Roof Slope = Slope ≤ 2:12 (10°)-Condition 1
Leeward Roof Slope = Slope ≤ 2:12 (10°)-Condition 1

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Vert. Load lb
Windward	18.5	0.89	-1.09	-30.0 psf	225	-6643
Leeward	18.5	0.89	-0.66	-18.2 psf	225	-4022
						-10665

Total Roof Uplift Load
10665 lbs.

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STRUCTURAL ENGINEERS, INC.

JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	12/5/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	ML3

WIND LOAD DETERMINATION - ALTERNATE ALL HEIGHTS METHOD (2010 CBC 1609A.6) (maintenance)

Based on Section 6.5 - Method 2 of ASCE 7-05

$p_{net} = q_s K_z C_{net} [I K_{zt}]$ (Eqn 16A-36)

V = 110 mph
q_s = 31.0 psf
I = 1.00
q_h = 18.0 ft

Exposure: C
zg = 900 ft
α = 9.5
K_{zt} = 1.00
$K_{Z\text{-LEEWARD}}$ = 0.88

Openings: Enclosed
Internal Pressure: ◉ ADDED ○ SUBTRACTED

Note: Internal pressure is either added or subtracted to both leeward and windward.

EAST to WEST

			WINDWARD FACE EAST				LEEWARD FACE WEST				TOTAL
STORY	Z ft	$K_{Z\text{-WINDWARD}}$	C_{net}	p_{net}	A sq ft	Load lb	C_{net}	p_{net}	A sq ft	Load lb	LOAD lb
Base	16.5	0.87	0.43	11.5 psf	375	4329	-0.51	-13.9 psf	375	-5230	9559
					375	4329			375	-5230	**9,559**

WEST to EAST

			WINDWARD FACE WEST				LEEWARD FACE EAST				TOTAL
STORY	Z ft	$K_{Z\text{-WINDWARD}}$	C_{net}	p_{net}	A sq ft	Load lb	C_{net}	p_{net}	A sq ft	Load lb	LOAD lb
Base	16.5	0.87	0.43	11.5 psf	375	4329	-0.51	-13.9 psf	375	-5230	9559
					375	4329			375	-5230	**9,559**

Roof Lateral Component - East to West

Wind Direction: Wind parallel to ridge or flat roof:

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Horiz. Load lb
Windward	0.0	0.85	0.73	19.2 psf	0	0
	0.0	0.85	-1.09	28.7 psf	0	0
						0

Total Load East to West
9559 lbs.

Roof Lateral Component - West to East

Wind Direction: Wind parallel to ridge or flat roof:

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Horiz. Load lb
Windward	0.0	0.85	0.73	19.2 psf	0	0
Leeward	0.0	0.85	-1.09	28.7 psf	0	0
						0

Total Load West to East
9559 lbs.

Roof Uplift Component

Wind Direction: Wind parallel to ridge or flat roof:

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Vert. Load lb
Windward	0.0	0.85	0.73	19.2 psf	0	0
Leeward	0.0	0.85	-1.09	-28.7 psf	0	0
						0

Total Roof Uplift Load
0 lbs.

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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	12/5/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	MLA

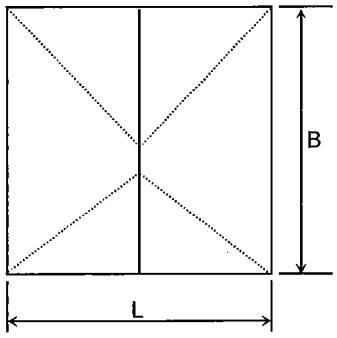
WIND LOADING ANALYSIS - Wall Components and Cladding WALL COMPONENTS

ASCE 7-05 Method 2: Analytical Procedure (Section 6.5.12.4)
Any Height with Gable Roof θ ≤ 45° or Monoslope Roof θ ≤ 30°

ROOF DATA:

Roof Type: Figure 6-11A: All Roof Types h ≤ 60 ft and θ ≤ 10°

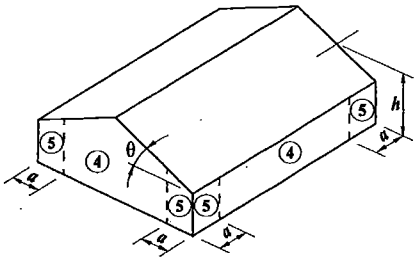
Bldg. Classification =	II	Table 1-1 Occupancy Category
Mean Roof Ht., h =	18.5 ft	use height of eve if θ ≤ 10°
Roof Angle, θ =	9.5	deg.
Building Width =	42.0 ft	normal to ridge
Building Length =	63.0 ft	parallel to ridge
Enclosure	Enclosed	Section 6.2 & Figure 6-5



Plan

WIND PRESSURE DATA:

Wind Speed, V =	110 mph	Wind Map, Figure 6-1
Exposure Category =	C	Sect. 6.5.6
I =	1.00	Table 6-1 (Importance factor)
Kh =	0.89	Kh = 2.01*(h/zg)^(2/a); (h = 15 ft min)
Topo. Factor, Kzt =	1.00	Sect. 6.5.7 & Figure 6-4
Direct. Factor, Kd =	0.85	Table 6-4
α =	9.50	Table 6-2
zg =	900	Table 6-2
qh =	23.4 psf	qh = 0.00256*Kz*Kzt*Kd*V^2*I; (where Kz = Kh) (Sect. 6.5.10, Eq. 6-15)



Isometric

External Pressure Coefficients, GCp (Figures 6-11~17):

	a	b	c	d
Windward Zone 4+	0.9	1.1	-0.2	0.6
Windward Zone 5+	0.9	1.1	-0.2	0.6
Leeward Zone 4-	-1.0	-1.1	0.2	-0.7
Leeward Zone 5-	-1.3	-1.6	0.3	-0.7

GCp = **a** (at Ae ≤ 10 sf)
GCp = **b** + **c** LOG(Ae)
GCp = **d** (at Ae ≥ 500 sf)

Positive & Negative Internal Pressure Coefficients, GCpi (Figure 6-5):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

Design Net External Wind Pressures (Sect. 6.5.12.4):

For h ≤ 60 ft: p = qh*[(GCp) - (+/-GCpi)] (psf)
For h > 60 ft: p = q*(GCp) - qi*(+/-GCpi) (psf) Windward, where q = qz and qi = qh
For h > 60 ft: p = q*(GCp) - qi*(+/-GCpi) (psf) Leeward, where q = qh and qi = qh

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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	12/5/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	4L5

WIND LOADING ANALYSIS - Wall Components and Cladding WALL COMPONENTS

ASCE 7-05 Method 2: Analytical Procedure (Section 6.5.12.4)
Any Height with Gable Roof θ ≤ 45° or Monoslope Roof θ ≤ 30°

Wind Load Tabulation for Wall Components & Cladding

Component	Ae	Z	Kz (windward)		p = Net Design Pressures (psf)			
					Windward		Leeward	
					Zone 4+	Zone 5+	Zone 4-	Zone 5-
				GCp				
				pi+				
				pi-				
CMU wall	5 sq-ft	4 ft	0.89	GCp	0.90	0.90	-0.99	-1.26
				pi+	**16.8**	**16.8**	**-27.3**	**-33.6**
				pi-	**25.2**	**25.2**	**-18.9**	**-25.2**
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				

Notes:

1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 5 (edge) **a = 4.2 ft**
3. Evaluate parapets at z = top of parapet
4. qz = qh when h ≤ 60 ft and for all roof components
   qi = qh assumed for all cases per Sect. 6.5.12.4.2
5. Per Section 6.1.4.2, the minimum wind load for C&C shall not be less than 10 psf.
6. Reduced wind pressures for stepped (low) roofs on buildings per Figure 6-12 have not been considered
7. GCp reduced by 10% where h ≤ 60 ft and θ ≤ 10° per Figure 6-11A, note 5

LATERAL ANALYSIS (MAINTENANCE BLDG)

BLDG WT

ROOF	16 PSF (42' x 63')	=	42.3 k
WALLS	10 PSF (2)(40' + 60')(15'/2)	=	15.0 k
MISC		=	1.7 k
			59 k

Seismic Force

$V_{base} = 0.0372(59k) = 2.19\ k$ (ult.) in both directions

Wind Force

$V_{WIND} = 12.0\ k + 0.408k = \underline{12.42\ k}$ (N/S Direction)

$= 9.56k = \underline{9.56\ k}$ (E/W Direction)

∴ Wind Governs

LATERAL ANALYSIS (Approximation)

LOAD DISTRIBUTION TO FRAMES

N/S Direction (rigid frames on all grid lines)

decreased to 12.42k, design ok✓

$F = 13.88k / 63' = 220$ PLF

LINE A/D - 220 PLF × 20'/2 = 2.2 k

B/C - 220 PLF × 20' = 4.4 k

* Bldg manufacturer to design structure, these calculations are to determine the approx size of ftgs.

E/W Direction (rigid frames @ lines 1 & 3)

$F = 9.56k / 42' = 228$ PLF

LINE 1/3 - 228 PLF × 42'/2 = 4.78 k

FRAME FDN DESIGN

WORST CASE = 4.78 k (E/W)

OTM = 4.78 k (15') = 71.7 k'

$T/C = 71.7k' / 20' = 3.59$ k

$P_{COL} = (16 PSF)(20' \times 42'/2) = 6.72k$ $P_{FTG} = 150 PCF (5')^2 (1.5') = 5.63k$

$M_{FTG} = 4.78k/2 (2') = 4.78k'$

$e = \frac{4.78k'}{6.72k + 5.63k} = 0.39'$ $L/6 = 5'/6 = 0.83'$

$SP = \frac{6.72k + 5.63k}{5' \times 5'} \left(1 + \frac{0.39' \times 6}{5'}\right) = 725$ PSF

$A_{s\,min} = 0.0014 (18") (60") = 1.51 in^2 / 5 = 0.30$

⇒ 5' × 1'-6" deep ftgs ok✓
w/ 5-#5 t&b

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Cantilevered Retaining Wall Design

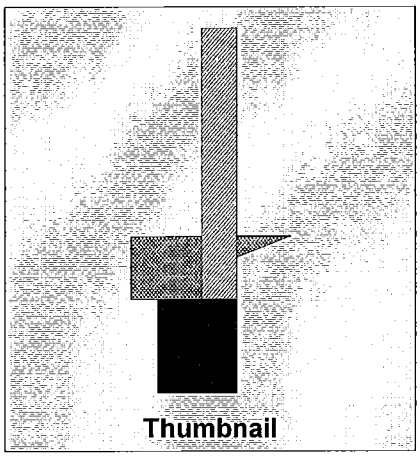
Code: IBC 2009

Criteria

Retained Height	=	1.00 ft
Wall height above soil	=	3.33 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	12.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	3,067.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	41.0 psf/ft
Toe Active Pressure	=	41.0 psf/ft
Passive Pressure	=	83.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
The above lateral load has been increased by a factor of		1.60
Wind on Exposed Stem	=	25.2 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Wall Stability Ratios

Overturning	=	1.91	OK
Slab Resists All Sliding !			
Total Bearing Load	=	753 lbs	
...resultant ecc.	=	3.91 in	
Soil Pressure @ Toe	=	1,187 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	3,067 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,662 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	3.0 psi	OK
Footing Shear @ Heel	=	0.0 psi	OK
Allowable	=	82.2 psi	

Sliding Calcs Slab Resists All Sliding !

Lateral Sliding Force	=	83.9 lbs

Load Factors

Building Code	IBC 2009
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

Top Stem

			Stem OK
Design Height Above Ftg	ft =		0.00
Wall Material Above "Ht"	=		Masonry
Thickness	=		8.00
Rebar Size	=		# 4
Rebar Spacing	=		16.00
Rebar Placed at	=		Center

Design Data

fb/FB + fa/Fa	=	0.153
Total Force @ Section	lbs =	134.3
Moment....Actual	ft-# =	357.8
Moment.....Allowable	=	2,332.7
Shear.....Actual	psi =	3.0
Shear.....Allowable	psi =	69.7
Wall Weight	=	75.0
Rebar Depth 'd'	in =	3.75
LAP SPLICE IF ABOVE	in =	36.00
LAP SPLICE IF BELOW	in =	
HOOK EMBED INTO FTG	in =	6.00

Lap splice above base reduced by stress ratio

Masonry Data

f'm	psi =	1,500
Fy	psi =	60,000
Solid Grouting	=	Yes
Modular Ratio 'n'	=	21.48
Equiv. Solid Thick.	in =	7.60
Masonry Block Type	=	Lightweight
Masonry Design Method	=	LRFD

Concrete Data

f'c	psi =	
Fy	psi =	

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Cantilevered Retaining Wall Design

Code: IBC 2009

Footing Dimensions & Strengths

Toe Width	=	0.83 ft
Heel Width	=	0.67
Total Footing Width	=	1.50
Footing Thickness	=	18.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	2.25 ft

f'c = 3,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0014
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	1,662	0 psf
Mu' : Upward	=	609	0 ft-#
Mu' : Downward	=	233	7 ft-#
Mu: Design	=	376	7 ft-#
Actual 1-Way Shear	=	2.99	0.00 psi
Allow 1-Way Shear	=	82.16	82.16 psi

Toe Reinforcing = # 5 @ 18.00 in
Heel Reinforcing = # 5 @ 18.00 in
Key Reinforcing = None Spec'd

Other Acceptable Sizes & Spacings
Toe: Not req'd, Mu < S * Fr
Heel: Not req'd, Mu < S * Fr
Key: No key defined

Summary of Overturning & Resisting Forces & Moments

Item		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#
Heel Active Pressure	=	128.1	0.83	106.8
Surcharge over Heel	=			
Toe Active Pressure	=	-128.1	0.83	-106.8
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=	83.9	4.17	349.5
Total	=	83.9	**O.T.M.** =	349.5

Resisting/Overturning Ratio = **1.91**
Vertical Loads used for Soil Pressure = 752.9 lbs

		.....RESISTING..... Force lbs	Distance ft	Moment ft-#
Soil Over Heel	=	0.0	1.50	0.1
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	91.3	0.42	37.9
Surcharge Over Toe	=			
Stem Weight(s)	=	324.8	1.16	377.8
Earth @ Stem Transitions	=			
Footing Weight	=	336.8	0.75	252.1
Key Weight	=		2.25	
Vert. Component	=			
Total =		752.9 lbs	**R.M.=**	667.9

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

DESIGNER NOTES:

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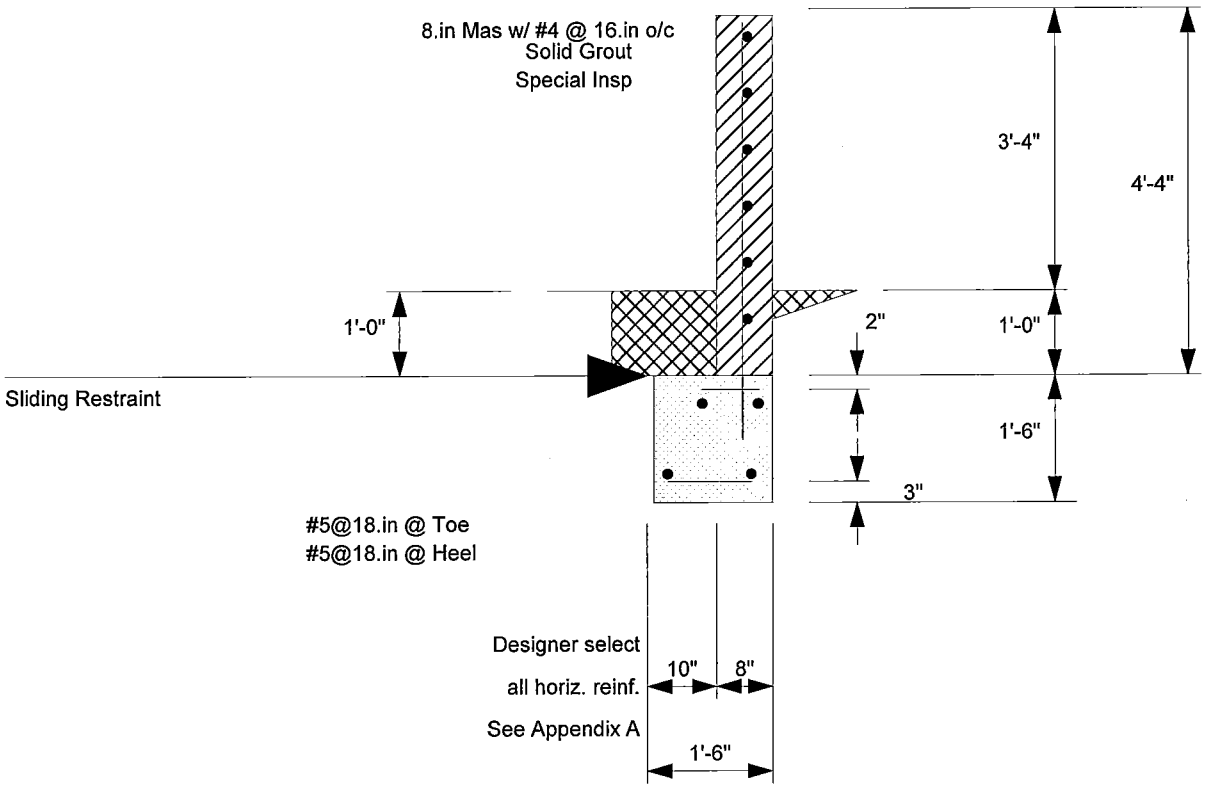
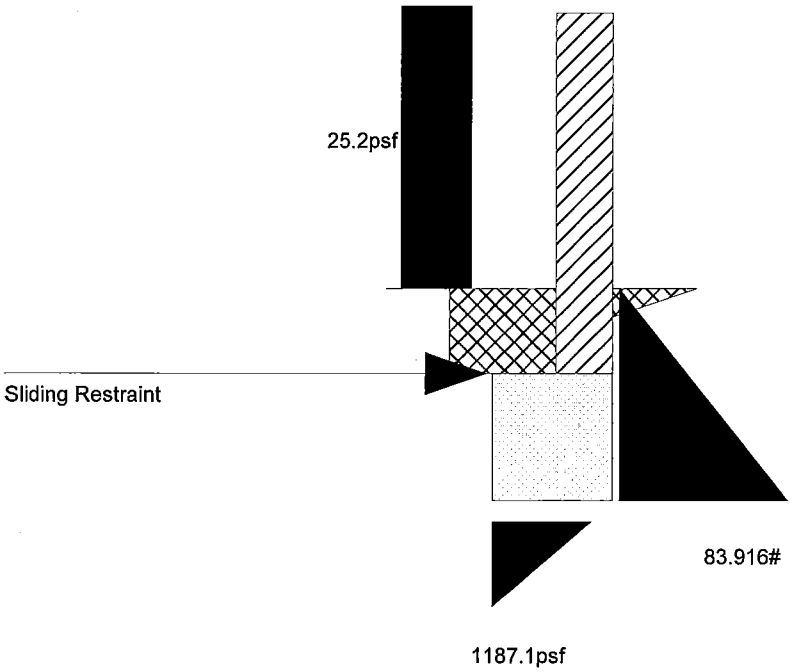
MW3


8.in Mas w/ #4 @ 16.in o/c
Solid Grout
Special Insp
3'-4"
4'-4"
1'-0"
2"
1'-0"
Sliding Restraint
1'-6"
3"
#5@18.in @ Toe
#5@18.in @ Heel
Designer select
all horiz. reinf.
See Appendix A
10"
8"
1'-6"

μW4



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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	10/26/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	HFD1

Foundation Design Parameters

Soils Report : #870 by W.L. Burle Engineers, P.A. dated 2/15/2011
Foundations to bear on engineered fill or native

Allowable Bearing Pressures

	D	D+L	D+L+E/1.4 (W)
SP max=	2300 psf	2300 psf	3067 psf

Allowable Skin Friction Pressures (for uplift loads) (Enter '0' for friction value if skin friction is not used)

sidewall friction
100 psf

Pad Footing Schedule

Width	Depth	Reinforcing	Allowable Vertical Loads (kips)			
			D	D+L	D+L+E/1.4	Uplift Capacity
3'-0" square	12"	4 - # 5 bars	20.7	20.7	27.6	2.2
4'-0" square	18"	5 - # 5 bars	36.8	36.8	49.1	5.0
5'-0" square	18"	7 - # 5 bars	57.5	57.5	76.7	7.8
6'-0" square	18"	7 - # 6 bars	82.8	82.8	110.4	11.2
7'-0" square	24"	9 - # 6 bars	112.7	112.7	150.3	24.1
8'-0" square	24"	9 - # 7 bars	147.2	147.2	196.3	30.6
9'-0" square	30"	10 - # 7 bars	186.3	186.3	248.4	45.1
10'-0" square	30"	11 - # 8 bars	230.0	230.0	306.7	54.6
11'-0" square	36"	11 - # 8 bars	278.3	278.3	371.1	75.3
12'-0" square	36"	13 - # 8 bars	331.2	331.2	441.6	88.3

Notes:
1. Footing uplift capacity = 0.9 (ftg wt. + overburden + slab) + sidewall friction x footing sidewall area

Foundation Design - Dead, Live, and Seismic Loads (E/1.4 - Working Stress Level Forces)

Soils Report : #870 by W.L. Burle Engineers, P.A. dated 2/15/2011
Foundations to bear on engineered fill or native

Allowable Bearing Pressures

	Dead	D+L	D+L+E/1.4	sidewall friction
SP max=	2300 psf	2300 psf	3067 psf	100 psf

For Uplift at Frame Cols

$d_{top\ of\ ftg}$= 12.00 in
t_{slab}= 4.00 in

Pad Footing Design — 3'-0" square

column width (bcol): 4.00 in
bcol+1/2 dist to base plate edge: 7.00 in

f'c=	3.00 ksi	Fy=	60.00 ksi	ϕ(moment)=	0.9	ϕ(shear)=	0.75
Pdl=	20.7 K	Ptl=	20.7 K	Ptl+Plat(down)=	27.6 K	0.9Pdl-Plat(up)=	2.2 K

Max Soil Pressure=		3.067 ksf	ok	Ultimate Factor= 1.5		
Spu=	4.601 ksf	bar area=	0.31 in2	As= 1.24 in2	>As(min)=	0.60 in2
Mu = Spu(b)(b/2-bcol/2)^2/2=		**10.08 Kft**		Vu= Spu(b)(b/2-(d+bcol/2)=	**7.04 K**	(one way)
a=AsFy/0.85F'cb=		0.81 in		Vu= Spu(b^2-(bcol+d)^2)=	**33.85 K**	(two way)
ϕMn=ϕAsFy(d-a/2)=		**44.47 Kft**	ok	ϕVn(one way)=ϕ(2)(f'c)^0.5bd=	**24.77 K**	ok
				ϕVn(two way)=ϕ(4)(f'c)^0.5b_od=	**84.63 K**	ok

Use	**3.00 ft square ftg**	**Dftg=**	**12.00 in**
rebar:	**4 - # 5 bars** ea way bottom		

0.625 in = dbar

Pad Footing Design — 4'-0" square

column width (bcol): 4.00 in
bcol+1/2 dist to base plate edge: 7.00 in

f'c=	3.00 ksi	Fy=	60.00 ksi	ϕ(moment)=	0.9	ϕ(shear)=	0.75
Pdl=	36.8 K	Ptl=	36.8 K	Ptl+Plat(down)=	49.1 K	0.9Pdl-Plat(up)=	5.0 K

Max Soil Pressure=		3.067 ksf	ok	Ultimate Factor= 1.5		
Spu=	4.601 ksf	bar area=	0.31 in2	As= 1.55 in2	>As(min)=	1.21 in2
Mu = Spu(b)(b/2-bcol/2)^2/2=		**26.85 Kft**		Vu= Spu(b)(b/2-(d+bcol/2)=	**9.39 K**	(one way)
a=AsFy/0.85F'cb=		0.76 in		Vu= Spu(b^2-(bcol+d)^2)=	**59.01 K**	(two way)
ϕMn=ϕAsFy(d-a/2)=		**97.62 Kft**	ok	ϕVn(one way)=ϕ(2)(f'c)^0.5bd=	**56.69 K**	ok
				ϕVn(two way)=ϕ(4)(f'c)^0.5b_od=	**201.96 K**	ok

Use	**4.00 ft square ftg**	**Dftg=**	**18.00 in**
rebar:	**5 - # 5 bars** ea way bottom		

0.625 in = dbar

Pad Footing Design — 5'-0" square

column width (bcol): 5.00 in
bcol+1/2 dist to base plate edge: 8.00 in

f'c=	3.00 ksi	Fy=	60.00 ksi	ϕ(moment)=	0.9	ϕ(shear)=	0.75
Pdl=	57.5 K	Ptl=	57.5 K	Ptl+Plat(down)=	76.7 K	0.9Pdl-Plat(up)=	7.8 K

Max Soil Pressure=		3.067 ksf	ok	Ultimate Factor= 1.5		
Spu=	4.601 ksf	bar area=	0.31 in2	As= 2.17 in2	>As(min)=	1.51 in2
Mu = Spu(b)(b/2-bcol/2)^2/2=		**53.99 Kft**		Vu= Spu(b)(b/2-(d+bcol/2)=	**22.28 K**	(one way)
a=AsFy/0.85F'cb=		0.85 in		Vu= Spu(b^2-(bcol+d)^2)=	**99.02 K**	(two way)
ϕMn=ϕAsFy(d-a/2)=		**136.22 Kft**	ok	ϕVn(one way)=ϕ(2)(f'c)^0.5bd=	**70.86 K**	ok
				ϕVn(two way)=ϕ(4)(f'c)^0.5b_od=	**211.40 K**	ok

Use	**5.00 ft square ftg**	**Dftg=**	**18.00 in**
rebar:	**7 - # 5 bars** ea way bottom		

0.625 in = dbar

Project Description: (Bulk Storage)

The expansion project for the VA National Cemetery in Zachary, Louisiana consists of three one-story structures and one shelter/canopy structure. The project is located in a hurricane zone with 110mph basic wind speed; requiring hurricane ties of all roof elements down to the foundation at all the buildings.

The Administration building is a one story building. The roof consists of prefabricated wood trusses with plywood sheathing supported by light-gage metal stud exterior bearing walls. The walls use Sureboard for shear resistance and bear on continuous footings.

The maintenance building is a pre-manufactured one-story metal building. The roof is part of the pre-manufactured design and consists of Z purlins and WF beams that span to WF columns. Rod bracing provides lateral resistance and it is also designed by the manufacturer. The columns will bear on spread footings and the building has a continuous footing all around. There is a bulk storage structure that is considered part of the maintenance building. Roof structure consists of wood framing with plywood sheathing. Wood stud non-bearing walls sit on top of 11' tall concrete walls, which sit on continuous footings.

There is also a shelter structure that has a pre-manufactured roof and lateral system comprised of tube members. The columns will bear on spread footings that will be connected with grade beams.

Design Criteria:

Building Type: Occupancy Category: I - Bulk Storage, Committal Shelter Structure
II – Administration and Maintenance Buildings

Design Codes: 2009 International Building Code
ASCE 7-05: Minimum Design Loads for Buildings and Other Structures, including *Supplement 1*
AISC 360-05: Specification for Structural Steel Buildings (13th Edition AISC)
ACI 318-08: Building Code Requirements for Structural Concrete
AF&PA 2005 NDS and 2008 SDPWS

Wind Criteria: 110 mph, Exposure C $I_w = 1.00$ $GC_{pi} = \pm 0.18$ (Enclosed Structure)

Seismic Criteria: Site Class: D $S_S = 0.122$ $S_1 = 0.055$ $S_{DS} = 0.130$ $S_{D1} = 0.088$

Seismic Design Category: B $I = 1.00$ $I_p = 1.0$

Lateral Force-Resisting System:

Administration Building
Light-gage metal stud bearing shear walls
$R = 6.5$ $\Omega_o = 3.0$ $C_d = 4.0$

Bulk Storage Building
Wood bearing and concrete shear walls
$R = 4.0$ $\Omega_o = 2.5$ $C_d = 4.0$

Maintenance Building/Committal Shelter
Light-gage metal stud bearing shear walls
$R = 3.5$ $\Omega_o = 3.0$ $C_d = 3.0$

Foundation Criteria:

Soils Report #870 CM 3022 by W.L.Burle Engineers, P.A., dated 2/15/2011

Allowable Bearing Pressures:	DL	2300 psf
(Non-structural elements)	DL + LL	2300 psf
	DL + LL + EQ/1.4	3067 psf
Allowable Lateral Pressures:	Retaining At-Rest Pressure	55 pcf
	Passive Pressure	300 pcf
	Coefficient of Friction	0.30

A: Roof Loads (Administration Bldg)

	Trusses	seismic weight
Asphalt Shingles	2.5	2.5
3/4" shtg	2.3	2.3
Trusses @ 32"cc	3.0	3.0
Insulation	0.5	0.5
sprinklers	1.0	1.0
MEP	2.0	2.0
ceilings	2.2	2.2
misc	1.5	1.5
DPO - dead load	15.0 psf	15.0 psf
LLR - reducible roof LL	20.0 psf	0.0 psf
TL	35.0 psf	15.0 psf

B: Roof Loads (Maintenance Bldg)

	deck	seismic weight
metal roofing (20ga)	1.8	1.8
Z' purlins (6"x16ga @ 24"cc)	1.5	1.5
Insulation	3.0	3.0
beams	2.9	2.9
columns	1.0	1.0
sprinklers	1.0	1.0
MEP	2.0	2.0
ceilings	1.8	1.8
misc	1.0	1.0
DPO - dead load	16.0 psf	16.0 psf
LLR - reducible roof LL	20.0 psf	0.0 psf
TL	36.0 psf	16.0 psf

C: Roof Loads (Bulk Storage)

	Joists	seismic weight
metal roofing (18ga)	1.8	1.8
single-ply roof (w/ re-roof)	5.5	5.5
5/8" shtg	1.9	1.9
2x10's @ 24"cc	1.7	1.7
sprinklers	1.0	1.0
MEP	1.0	1.0
misc	0.9	0.9
DPO - dead load	12.0 psf	12.0 psf
LLR - reducible roof LL	20.0 psf	0.0 psf
TL	32.0 psf	12.0 psf

D: Roof Loads (Committal Shelter)

	deck	seismic weight
metal roofing (20ga)	1.8	1.8
Z' purlins (6"x16ga @ 24"cc)	1.5	1.5
beams	2.2	2.2
columns	1.0	1.0
sprinklers	1.0	1.0
MEP	2.0	2.0
misc	1.0	1.0
DPO - dead load	10.5 psf	10.5 psf
LLR - reducible roof LL	20.0 psf	0.0 psf
TL	30.5 psf	10.5 psf

E: Wall Loads

Wall Finish:	Light-Gage	Wood	Concrete
6"x16ga mtl studs @ 16"cc	1.5		
2x6 wood studs @ 16"cc		1.5	
12" concrete			150.0
Metal siding (both sides)		3.0	
1/2" shtg (both sides)		3.4	
3 5/8" brick veneer	35.0		
1/2" Sureboard	4.3		
5/8" gyp board	2.8	2.8	
misc	1.4	1.3	
DPO - non-composite DL	45.0 psf	12.0 psf	150.0 psf

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JOB ____ JOB NO. ____ DATE ____ CLIENT ____ BY ____ SHEET NO. BR1

Roof Framing Design (Bulk Storage)

Typical Joists

Span = 11'-6"

W = (12 PSF + 20 PSF) 24" o.c. = 64 PLF

V = 368 #

M = 1058 #'

$EI_{req'd} = 450(64\,PLF)(11.5')^3 / 29000\,ksi = 36.96$

$W_{uplift} = 38\,PSF \times 2' = 76\,PLF$

$W_{net} = 0.6(64\,PLF) - 76\,PLF = -38\,PLF$

→ Use 2x10 joists

$V_a = 879$ #

$M_a = 1765$ #'

$EI = 158 \times 10^6$ lbs in²

$V_{uplift} = 38\,PLF(11.5')/2 = 219$ #

2-10d ok ✓

2 x 36 #/in x 3" x 1.6 = 346 #

Check Uplift on 2x10 outriggers

Span = 3'-6"

W = (0.6 x 12 PSF - 45.8 PSF) 2' = - 77.2 PLF

V = 135 #

M = 118 #'

$EI = 450(77.2\,PLF)(3.5')^3 / 10^6 = 1.49$

⇒ 2x10 outriggers ok ✓

TM conn to top PL's

Check 2x10 fascia

W = (0.6 x 12 PSF - 45.8 PSF) 1.75' = -67.6 PLF

V = 389 #

M = 1118 #'

$EI_{req'd} = 450(67.6\,PLF)(11.5')^3 / 10^6 = 46.26$

⇒ 2x10 fascia ok ✓

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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	10/26/2011
CLIENT	Jacbos	BY	GAM	SHEET NO.	BL1

ASCE 7-05 (incl. supp. #1) Seismic Base Shear Calculation (storage)

Location: Zachary — Mapped MCE Spectral Acceleration (from Lat/Long using USGS program):
Height (hn): 14.0 ft — Latitude = 30.6439 — Longitude = -91.2567 — Ss= 0.122g
Regularity: regular — S1= 0.055g
Seismic Force-Resisting Sys. (Table 12.2-1): Wood and concrete shear walls — R= 4
Period Coefficients: 0.028 (0.8) for steel MRF, 0.016 (0.9) for concrete MRF — Ct= 0.020
listed as Ct (x) — 0.030 (0.75) for steel EBF, 0.020 (0.75) for all others — x= 0.75
Building Period: per formula 12.8-7 — Ta = Ct*hn^x= 0.14 sec
by computer: (overrides above) — Tcomp= 0.00 sec — Cu= 1.7
see Table 12.8-1 for upper limit on calculated T: T <= Cu*Ta — T= 0.14 sec

Occupancy Category (OC): Secs. 1.5&11.5 & Tabs. 1-1&11.5-1 — OC= 2 — I= 1.00

Site Class Section 20.3 — Site Class D

Per sections 11.4.2 & 20.1, Site Class D may be used if soil properties are not known.

				User Defined
Site coefficients Fa & Fv	Sec. 11.4.3; Tables 11.4-1 & 11.4-2	Fa=	1.60	0.00
	(straight line interpolation used)	Fv=	2.40	0.00

Adjusted Maximum Spectral Response Parameters — Section 11.4.3
Sms = Fa x Ss= — Sms= 0.195g
Sm1 = Fv x S1= — Sm1= 0.132g

Design Spectral Response Acceleration Parameters — Section 11.4.4
11.4.5: Ts = Sd1 / Sds = 0.676 — Sds = 2/3 x Sms= — **Sds= 0.130g**
0.8*Ts = 0.541 — Sd1 = 2/3 x Sm1= — **Sd1= 0.088g**

Seismic Design Category Section 11.6 — per Table 11.6-1, A
per Table 11.6-2, B

per section 11.6, is S1>=0.75? **no, use more severe value from Tables 11.6-1 & 11.6-2, EXCEPT SDC only per Table 11.6-1 is allowed if all 4 exceptions are met**

Exc #1: In both orthogonal directions, is Ta < 0.8*Ts ? — no
Exc #2: In both orthogonal directions, is T used for drift < 1.0*Ts ? — no
Exc #3: Is formula 12.8-2 used to determine Cs ? — no
Exc #4: Are diaphragms rigid ? Or for flexible diaphragms, are the vertical elements of LFRS spaced closer than 40 ft apart ? — no

Are all exceptions met ? NO — **use more severe value from Tables 11.6-1 & 11.6-2**
Seismic Design Category — **SDC= B**

Equivalent Lateral Force Procedure Section 12.8
See Table 12.6-1 for selection of analysis procedure

			For Force Calculations	For Drift Calculations*
per formula 12.8-2		Cs= Sds / (R/I)	0.0325	0.0325
per formula 12.8-3 for T < T_L		Cs< Sd1 / (R/I) / T	<0.1520	<0.1520
per formula 12.8-5		Cs> 0.044 Sds I > 0.01	>0.0100	NA
(formula 12.8-5 Incorporates proposed change of Suppl. #2)				
Does 1614A.1.7 apply?	no	-	NA	NA
per formula 12.8-6, if S1>=0.6g	FALSE	Cs> 0.5 S1 / (R/I)	NA	NA
		Cs=	0.0325	0.0325

Ultimate Strength Base Shear for FORCE Calculations: Vult= 0.0325 W

Ultimate Strength Base Shear for DRIFT Calculations*: Vult= 0.0325 W
(* For drift calculation, section 12.8.6.2 allows use of T without limit of T <= Cu*Ta)

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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	12/5/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	BL2

WIND LOAD DETERMINATION - ALTERNATE ALL HEIGHTS METHOD (2010 CBC 1609A.6) (storage)

Based on Section 6.5 - Method 2 of ASCE 7-05

$p_{net} = q_s K_z C_{net} [IK_{zt}]$ (Eqn 16A-36)

V = 110 mph
q_s = 31.0 psf
I = 1.00
q_h = 14.0 ft

Exposure: C
zg = 900 ft
α = 9.5
K_{zt} = 1.00
$K_{Z\text{-LEEWARD}}$ = 0.85

Openings: Partially Enclosed
Internal Pressure: ◉ ADDED ○ SUBTRACTED

Note: Internal pressure is either added or subtracted to both leeward and windward.

SOUTH to NORTH

			WINDWARD FACE SOUTH				LEEWARD FACE NORTH				TOTAL
STORY	Z ft	$K_{Z\text{-WINDWARD}}$	C_{net}	p_{net}	A sq ft	Load lb	C_{net}	p_{net}	A sq ft	Load lb	LOAD lb
Base	13.0	0.85	0.11	2.9 psf	100	290	-0.83	-21.9 psf	100	-2187	2477
					100	290			100	-2187	**2,477**

NORTH to SOUTH

			WINDWARD FACE NORTH				LEEWARD FACE SOUTH				TOTAL
STORY	Z ft	$K_{Z\text{-WINDWARD}}$	C_{net}	p_{net}	A sq ft	Load lb	C_{net}	p_{net}	A sq ft	Load lb	LOAD lb
Base	13.0	0.85	0.11	2.9 psf	100	290	-0.83	-21.9 psf	100	-2187	2477
					100	290			100	-2187	**2,477**

Roof Lateral Component - South to North

Wind Direction: Wind perpendicular to ridge
Windward Roof Slope = Slope ≤ 2:12 (10°)-Condition 1
Leeward Roof Slope = Slope ≤ 2:12 (10°)-Condition 1

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Horiz. Load lb
Windward	13.0	0.85	-1.41	-37.2 psf	0	0
Leeward	13.0	0.85	-0.97	25.6 psf	0	0
						0

Total Load South to North
2477 lbs.

Roof Lateral Component - North to South

Wind Direction: Wind perpendicular to ridge
Windward Roof Slope = Slope ≤ 2:12 (10°)-Condition 1
Leeward Roof Slope = Slope ≤ 2:12 (10°)-Condition 1

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Horiz. Load lb
Windward	13.0	0.85	-1.41	-37.2 psf	0	0
Leeward	13.0	0.85	-0.97	25.6 psf	0	0
						0

Total Load North to South
2477 lbs.

Roof Uplift Component

Wind Direction: Wind perpendicular to ridge
Windward Roof Slope = Slope ≤ 2:12 (10°)-Condition 1
Leeward Roof Slope = Slope ≤ 2:12 (10°)-Condition 1

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Vert. Load lb
Windward	13.0	0.85	-1.41	-37.2 psf	0	0
Leeward	13.0	0.85	-0.97	-25.6 psf	0	0
						0

Total Roof Uplift Load
0 lbs.

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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	12/5/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	BL3

WIND LOAD DETERMINATION - ALTERNATE ALL HEIGHTS METHOD (2010 CBC 1609A.6) (storage)

Based on Section 6.5 - Method 2 of ASCE 7-05

$$p_{net} = q_s K_z C_{net} [I K_{zt}]$$ (Eqn 16A-36)

V = 110 mph
q_s = 31.0 psf
I = 1.00
q_h = 18.0 ft

Exposure: C
zg = 900 ft
α = 9.5
K_{zt} = 1.00
$K_{Z\text{-LEEWARD}}$ = 0.88

Openings: Partially Enclosed
Internal Pressure: ◉ ADDED ○ SUBTRACTED

Note: Internal pressure is either added or subtracted to both leeward and windward.

EAST to WEST

			WINDWARD FACE EAST				LEEWARD FACE WEST				TOTAL
STORY	Z ft	$K_{Z\text{-WINDWARD}}$	C_{net}	p_{net}	A sq ft	Load lb	C_{net}	p_{net}	A sq ft	Load lb	LOAD lb
Base	12.0	0.85	0.11	2.9 psf	222	643	-0.83	-22.7 psf	222	-5039	5682
					222	643			222	-5039	**5,682**

WEST to EAST

			WINDWARD FACE WEST				LEEWARD FACE EAST				TOTAL
STORY	Z ft	$K_{Z\text{-WINDWARD}}$	C_{net}	p_{net}	A sq ft	Load lb	C_{net}	p_{net}	A sq ft	Load lb	LOAD lb
Base	12.0	0.85	0.11	2.9 psf	222	643	-0.83	-22.7 psf	222	-5039	5682
					222	643			222	-5039	**5,682**

Roof Lateral Component - East to West

Wind Direction: Wind perpendicular to ridge
Windward Roof Slope = Slope ≤ 2:12 (10°)-Condition 2
Leeward Roof Slope = Slope ≤ 2:12 (10°)-Condition 1

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Horiz. Load lb
Windward	14.0	0.85	-0.6	-15.8 psf	86	-236
Leeward	12.0	0.85	-0.97	25.6 psf	86	382
						146

Total Load East to West
5828 lbs.

Roof Lateral Component - West to East

Wind Direction: Wind perpendicular to ridge
Windward Roof Slope = Slope ≤ 2:12 (10°)-Condition 2
Leeward Roof Slope = Slope ≤ 2:12 (10°)-Condition 1

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Horiz. Load lb
Windward	12.0	0.85	-0.6	-15.8 psf	86	-236
Leeward	14.0	0.85	-0.97	25.6 psf	86	382
						146

Total Load West to East
5828 lbs.

Roof Uplift Component

Wind Direction: Wind perpendicular to ridge
Windward Roof Slope = Slope ≤ 2:12 (10°)-Condition 1
Leeward Roof Slope = Slope ≤ 2:12 (10°)-Condition 1

	Z ft	Kz	C_{net}	p_{net}	A sq ft	Vert. Load lb
Windward	14.0	0.85	-1.41	-37.2 psf	86	-3147
Leeward	12.0	0.85	-0.97	-25.6 psf	86	-2165
						-5311

Total Roof Uplift Load
5311 lbs.

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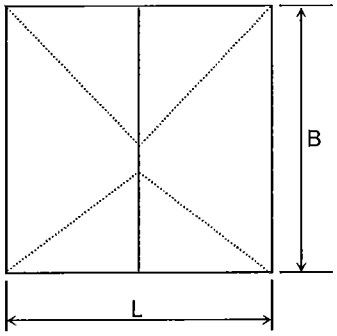
JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	12/5/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	BL4

WIND LOADING ANALYSIS - Roof Components and Cladding (bulk storage) MAIN ROOF COMPONENTS
ASCE 7-05 Method 2: Analytical Procedure (Section 6.5.12.4)
Any Height with Gable Roof θ ≤ 45° or Monoslope Roof θ ≤ 30°

ROOF DATA:

Roof Type: Figure 6-14A: Monoslope 3° < θ ≤ 10°

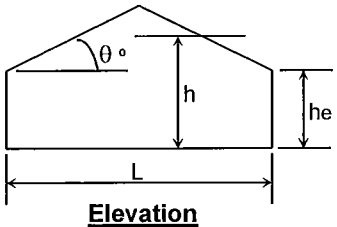
Bldg. Classification =	II	Table 1-1 Occupancy Category
Mean Roof Ht., h =	14.0 ft	use height of eve if θ ≤ 10°
Roof Angle, θ =	9.5	deg.
Building Width =	14.0 ft	normal to ridge
Building Length =	37.0 ft	parallel to ridge
Enclosure	Partial	Section 6.2 & Figure 6-5



Plan

WIND PRESSURE DATA:

Wind Speed, V =	110 mph	Wind Map, Figure 6-1
Exposure Category =	C	Sect. 6.5.6
I =	1.00	Table 1 (Importance factor)
Kh =	0.85	Kh = 2.01*(h/zg)^(2/a); (h = 15 ft min)
Topo. Factor, Kzt =	1.00	Sect. 6.5.7 & Figure 6-4
Direct. Factor, Kd =	0.85	Table 6-4
α =	9.50	Table 6-2
zg =	900	Table 6-2
qh =	22.4 psf	qh = 0.00256*Kh*Kzt*Kd*V^2*I (Sect. 6.5.10, Eq. 6-15)



Elevation

External Pressure Coefficients, GCp (Figures 6-11~17):

	a	b	c	d
Zone 1 2 3+	0.3	0.4	-0.1	0.2
Zone 1-	-1.1	-1.1	0.0	-1.1
Zone 2-	-1.6	-1.7	0.1	-1.5
Zone 3-	-2.6	-3.6	1.0	-1.6
Zone 1 2 OH	-1.7	-1.8	0.1	-1.6
Zone 3 OH	-2.8	-4.8	2.0	-0.8

GCp = **a** (at Ae ≤ 10 sf)
GCp = **b** + **c** LOG(Ae)
GCp = d (at Ae ≥ 100 sf)

Positive & Negative Internal Pressure Coefficients, GCpi (Figure 6-5):

+GCpi Coef. =	0.55	(positive internal pressure)
-GCpi Coef. =	-0.55	(negative internal pressure)

Design Net External Wind Pressures (Sect. 6.5.12.4):

For h ≤ 60 ft: p = qh*[(GCp) - (+/-GCpi)] (psf)
For h > 60 ft: p = q*(GCp) - qi*(+/-GCpi) (psf)

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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	12/5/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	BL5

WIND LOADING ANALYSIS - Roof Components and Cladding MAIN ROOF COMPONENTS

ASCE 7-05 Method 2: Analytical Procedure (Section 6.5.12.4)
Any Height with Gable Roof θ ≤ 45° or Monoslope Roof θ ≤ 30°

Wind Load Tabulation for Roof Components & Cladding

Component	Ae sq ft		p = Net Design Pressures (psf)					
			Zone 1 2 3+	Zone 1-	Zone 2-	Zone 3-	Zone 1 2 OH	Zone 3 OH
Roof Element	8	GCp	0.30	-1.10	-1.60	-2.60	-1.70	-2.80
		pi+	**-5.6**	**-36.9**	**-48.1**	**-70.4**	**-38.0**	**-62.6**
		pi-	**19.0**	**-12.3**	**-23.5**	**-45.8**	**-38.0**	**-62.6**
		GCp						
		pi+						
		pi-						
		GCp						
		pi+						
		pi-						
		GCp						
		pi+						
		pi-						
		GCp						
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		pi+						
		pi-						
		GCp						
		pi+						
		pi-						
		GCp						
		pi+						
		pi-						

Notes:

1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 2 (edge) **a = 3.0 ft**
3. Width of Zone 3 (corner) **a = 3.0 ft**
4. For parapets in excess of 3 ft in height is provided around perimeter of roof with θ ≤ 10 degrees, Zone 3 shall be treated as Zone 2.
5. qz = qh when h ≤ 60 ft and for all roof components
   qi = qh assumed for all cases per Sect. 6.5.12.4.2
6. Per Section 6.1.4.2, the minimum wind load for C&C shall not be less than 10 psf.
7. Reduced wind pressures for stepped (low) roofs on buildings per Figure 6-12 have not been considered

BUEHLER & BUEHLER STRUCTURAL ENGINEERS, INC.

JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	12/5/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	BL6

WIND LOADING ANALYSIS - Wall Components and Cladding (bulk storage)

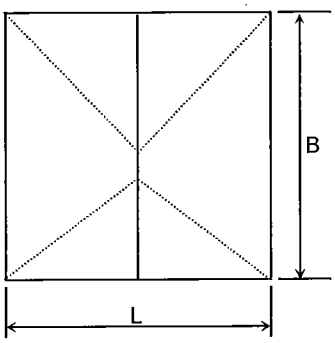
WALL COMPONENTS

ASCE 7-05 Method 2: Analytical Procedure (Section 6.5.12.4)
Any Height with Gable Roof θ ≤ 45° or Monoslope Roof θ ≤ 30°

ROOF DATA:

Roof Type: Figure 6-11A: All Roof Types h ≤ 60 ft and θ ≤ 10°

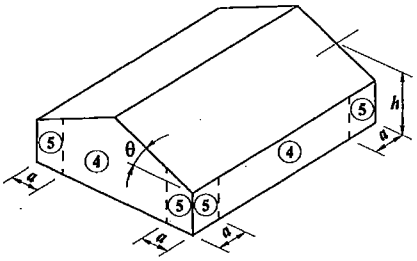
Bldg. Classification =	II	Table 1-1 Occupancy Category
Mean Roof Ht., h =	14.0 ft	use height of eve if θ ≤ 10°
Roof Angle, θ =	9.5	deg.
Building Width =	14.0 ft	normal to ridge
Building Length =	37.0 ft	parallel to ridge
Enclosure	Partial	Section 6.2 & Figure 6-5



Plan

WIND PRESSURE DATA:

Wind Speed, V =	110 mph	Wind Map, Figure 6-1
Exposure Category =	C	Sect. 6.5.6
I =	1.00	Table 6-1 (Importance factor)
Kh =	0.85	Kh = 2.01*(h/zg)^(2/a); (h = 15 ft min)
Topo. Factor, Kzt =	1.00	Sect. 6.5.7 & Figure 6-4
Direct. Factor, Kd =	0.85	Table 6-4
α =	9.50	Table 6-2
zg =	900	Table 6-2
qh =	22.4 psf	qh = 0.00256*Kz*Kzt*Kd*V^2*I; (where Kz = Kh) (Sect. 6.5.10, Eq. 6-15)



Isometric

External Pressure Coefficients, GCp (Figures 6-11~17):

	a	b	c	d
Windward Zone 4+	0.9	1.1	-0.2	0.6
Windward Zone 5+	0.9	1.1	-0.2	0.6
Leeward Zone 4-	-1.0	-1.1	0.2	-0.7
Leeward Zone 5-	-1.3	-1.6	0.3	-0.7

GCp = **a** (at Ae ≤ 10 sf)
GCp = **b** + **c** LOG(Ae)
GCp = **d** (at Ae ≥ 500 sf)

Positive & Negative Internal Pressure Coefficients, GCpi (Figure 6-5):

+GCpi Coef. =	0.55	(positive internal pressure)
-GCpi Coef. =	-0.55	(negative internal pressure)

Design Net External Wind Pressures (Sect. 6.5.12.4):

For h ≤ 60 ft: p = qh*[(GCp) - (+/-GCpi)] (psf)
For h > 60 ft: p = q*(GCp) - qi*(+/-GCpi) (psf) Windward, where q = qz and qi = qh
For h > 60 ft: p = q*(GCp) - qi*(+/-GCpi) (psf) Leeward, where q = qh and qi = qh

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JOB	Port Hudson Cemetery Gravesite Expansion				
JOB NO.	S08-090.08			DATE	12/5/2011
CLIENT	Jacobs	BY	GAM	SHEET NO.	BL7

WIND LOADING ANALYSIS - Wall Components and Cladding WALL COMPONENTS

ASCE 7-05 Method 2: Analytical Procedure (Section 6.5.12.4)
Any Height with Gable Roof θ ≤ 45° or Monoslope Roof θ ≤ 30°

Wind Load Tabulation for Wall Components & Cladding

Component	Ae	Z	Kz (windward)		p = Net Design Pressures (psf)			
					Windward		Leeward	
					Zone 4+	Zone 5+	Zone 4-	Zone 5-
				GCp				
				pi+				
				pi-				
Wall Stud	5 sq-ft	14 ft	0.85	GCp	0.90	0.90	-0.99	-1.26
				pi+	**7.8**	**7.8**	**-34.4**	**-40.5**
				pi-	**32.4**	**32.4**	**-9.8**	**-15.9**
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				
				GCp				
				pi+				
				pi-				

Notes:

1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 5 (edge) **a = 3.0 ft**
3. Evaluate parapets at z = top of parapet
4. qz = qh when h ≤ 60 ft and for all roof components
   qi = qh assumed for all cases per Sect. 6.5.12.4.2
5. Per Section 6.1.4.2, the minimum wind load for C&C shall not be less than 10 psf.
6. Reduced wind pressures for stepped (low) roofs on buildings per Figure 6-12 have not been considered
7. GCp reduced by 10% where h ≤ 60 ft and θ ≤ 10° per Figure 6-11A, note 5

LATERAL ANALYSIS (Storage bldg)

BLDG WT

ROOF	(12 PSF)(37' x 14')	=	6.22 k
WOOD WALLS	(12 PSF)(4 x 14' + 33')(3')	=	3.20 k
CONC WALLS	(150 PSF)(4 x 14' + 33')(10.5'/3)	=	46.73 k
MISC			1.85 k
			58.0 k

Seismic Force

$V_{base} = 0.0325(58k) = 1.88k$ (ult.) (in both directions)

Wind Force

$V_{WIND} = 2477\# = 2.5k$ (N/S Direction)

$= 5682\# + 146\# = 5828\# \approx 6.7k$ (E/W Direction)

↳ decreased, previous load of 6.7k ok for design.

∴ WIND GOVERNS

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LATERAL ANALYSIS

LINE LOADS

N/S Direction

LINE A - 2.5 k

E/W Direction

LINE 1 - 6.7 k (1/6) = 1.11 k
2 - 6.7 k (1/3) = 2.23 k
3 - 6.7 k (1/3) = 2.23 k
4 - 6.7 k (1/6) = 1.11 k

DIAPHRAGM DESIGN

N/S Direction

Entire load is resisted on back wall

$W_{N/S} = 2.5\,k / 14' = 179$ PLF

$v_{DIAPH} = 179\text{ PLF} \times 14' = 2.5k / 37' = 68$ PLF

E/W Direction

$W_{E/W} = 6.7k / 37' = 181$ PLF

$v_{DIAPH} = 181\text{ PLF} \times 11.5'/2 = 1041\,\#/14' = 74$ PLF

USE 5/8" sh'tg w/ 10d @ 6" o.c. EN + 10d @ 12" FN

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JOB NO. _____ DATE _____
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Lateral Analysis

Shear Wall Design

N/S Direction

$F_{WALL} = 2.5\ k$ $\quad L_{WALL} = 37'$

$v_{WALL} = 2.5k / 37' = 68\ PLF$

$OTM = 2.5k\ (1.5') = 3.75\ k'$

$DLRM = [12\ PSF \times 3.5' + 12\ PSF\ (1.5')]\ (37')^2/2 = 41.1\ k'$

$T/C = \dfrac{0.6\ (41.1\ k') - 3.75\ k'}{36'} = 0.58$ $\quad \therefore$ No overturning.

use HDU2 min

E/W Direction

$F_{WALL\ MAX} = 2.23k$ $\quad L_{WALL} = 12'$

$v_{WALL} = 2.23k / 12' = 186\ PLF$

$OTM = 2.23k\ (3') = 6.7\ k'$

$DLRM = [12\ PSF \times 6' + 12\ PSF\ (3')]\ (12')^2/2 = 7.78\ k'$

$T/C = \dfrac{(0.6 \times 7.76\ k') - 6.7\ k'}{11'} = -0.185\ k$

$\therefore$ typical anchor bolts ok ✓ for uplift

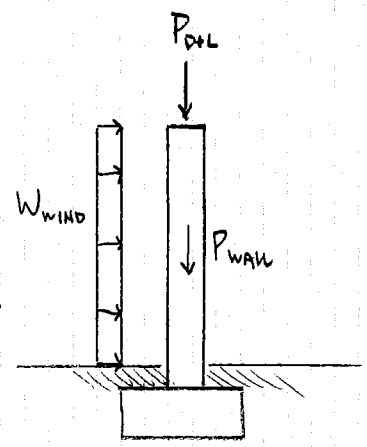
LATERAL ANALYSIS

Conc WALL DESIGN

Roof loads

$P_{DL} = (1.2 \times 12\ PSF)\ 11.5'/2 = 83\ PLF$

$P_{LL} = (1.6 \times 20\ PSF)\ 11.5'/2 = 184\ PLF$



Wind Loads

$W_{WIND} = (32.4\ PSF + 7.8\ PSF)(1') = \underline{40.2\ PSF}$ ∴ WIND GOVERNS

Seismic Loads

WALL Wt = $150\ PCF\ (1' \times 11.5') = 1725\ PLF$

$F_p = 0.4\ S_{DS}\ I\ W_p = 0.4\ (0.0325)(1.0)(1725\ PLF) = 22.4\ PLF$

IN-PLANE LOADS

MAX V = 2.23 K

WALL L = 12'

∴ Conc walls ok for shear by inspection

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Cantilevered Retaining Wall Design

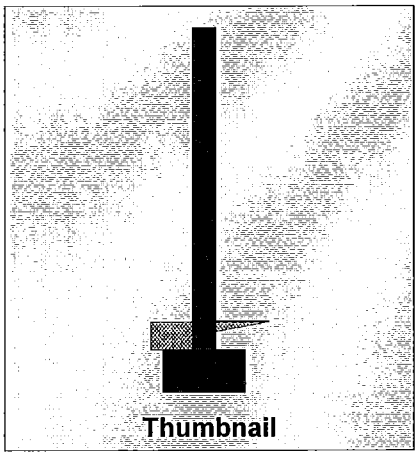
Code: IBC 2009

Criteria

Retained Height	=	1.00 ft
Wall height above soil	=	10.50 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	12.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	3,067.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	41.0 psf/ft
Toe Active Pressure	=	41.0 psf/ft
Passive Pressure	=	83.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	83.0 lbs
Axial Live Load	=	184.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
The above lateral load has been increased by a factor of		1.60
Wind on Exposed Stem	=	40.2 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Design Summary

Wall Stability Ratios

Overturning	=	1.54	OK
Slab Resists All Sliding !			
Total Bearing Load	=	3,055 lbs	
...resultant ecc.	=	12.85 in	
Soil Pressure @ Toe	=	2,999 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	3,067 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	4,235 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	0.9 psi	OK
Footing Shear @ Heel	=	3.2 psi	OK
Allowable	=	82.2 psi	

Sliding Calcs Slab Resists All Sliding !

Lateral Sliding Force	=	422.1 lbs

Stem Construction

Top Stem

		Stem OK
Design Height Above Ftg	ft =	0.00
Wall Material Above "Ht"	=	Concrete
Thickness	=	12.00
Rebar Size	=	# 5
Rebar Spacing	=	16.00
Rebar Placed at	=	Center

Design Data

fb/FB + fa/Fa	=	**0.692**
Total Force @ Section	lbs =	675.4
Moment....Actual	ft-# =	4,221.0
Moment.....Allowable	=	6,098.1
Shear.....Actual	psi =	9.4
Shear.....Allowable	psi =	94.9
Wall Weight	=	150.0
Rebar Depth 'd'	in =	6.00
LAP SPLICE IF ABOVE	in =	12.80
LAP SPLICE IF BELOW	in =	
HOOK EMBED INTO FTG	in =	9.59

Lap splice above base reduced by stress ratio

Masonry Data

f'm	psi =	
Fs	psi =	
Solid Grouting	=	
Modular Ratio 'n'	=	
Short Term Factor	=	
Equiv. Solid Thick.	=	
Masonry Block Type	=	Medium Weight
Masonry Design Method	=	ASD

Concrete Data

f'c	psi =	4,000.0
Fy	psi =	60,000.0

Load Factors

Building Code	IBC 2009
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

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Cantilevered Retaining Wall Design

Code: IBC 2009

Footing Dimensions & Strengths

Toe Width	=	1.25 ft
Heel Width	=	2.25
Total Footing Width	=	3.50
Footing Thickness	=	18.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	2.25 ft

f'c = 3,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0014
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	4,235	0 psf
Mu' : Upward	=	2,632	0 ft-#
Mu' : Downward	=	366	366 ft-#
Mu: Design	=	2,265	366 ft-#
Actual 1-Way Shear	=	0.89	3.15 psi
Allow 1-Way Shear	=	82.16	82.16 psi

Toe Reinforcing = # 5 @ 18.00 in
Heel Reinforcing = # 5 @ 18.00 in
Key Reinforcing = None Spec'd

Other Acceptable Sizes & Spacings
Toe: Not req'd, Mu < S * Fr
Heel: Not req'd, Mu < S * Fr
Key: No key defined

Summary of Overturning & Resisting Forces & Moments

Item		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#
Heel Active Pressure	=	128.1	0.83	106.8
Surcharge over Heel	=			
Toe Active Pressure	=	-128.1	0.83	-106.8
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=	422.1	7.75	3,271.3
Total	=	422.1	**O.T.M.** =	3,271.3

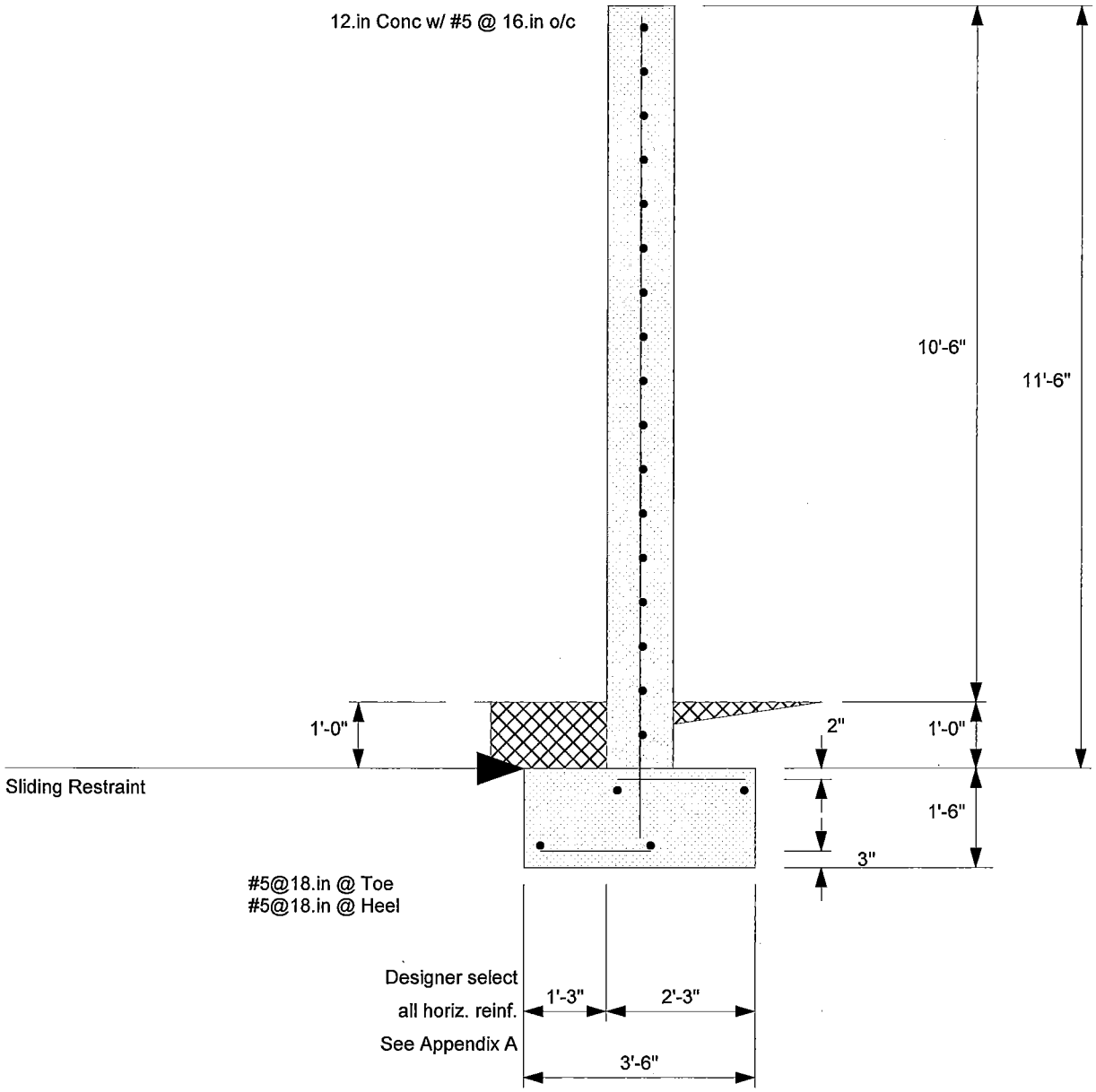
Resisting/Overturning Ratio = **1.54**
Vertical Loads used for Soil Pressure = 3,054.5 lbs

		.....RESISTING..... Force lbs	Distance ft	Moment ft-#
Soil Over Heel	=	137.5	2.88	395.3
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=	83.0	1.75	145.3
* Axial Live Load on Stem	=	184.0	1.75	322.0
Soil Over Toe	=	137.5	0.63	85.9
Surcharge Over Toe	=			
Stem Weight(s)	=	1,725.0	1.75	3,018.8
Earth @ Stem Transitions	=			
Footing Weight	=	787.5	1.75	1,378.1
Key Weight	=		2.25	
Vert. Component	=			
Total	=	2,870.5 lbs	**R.M.=**	5,023.4

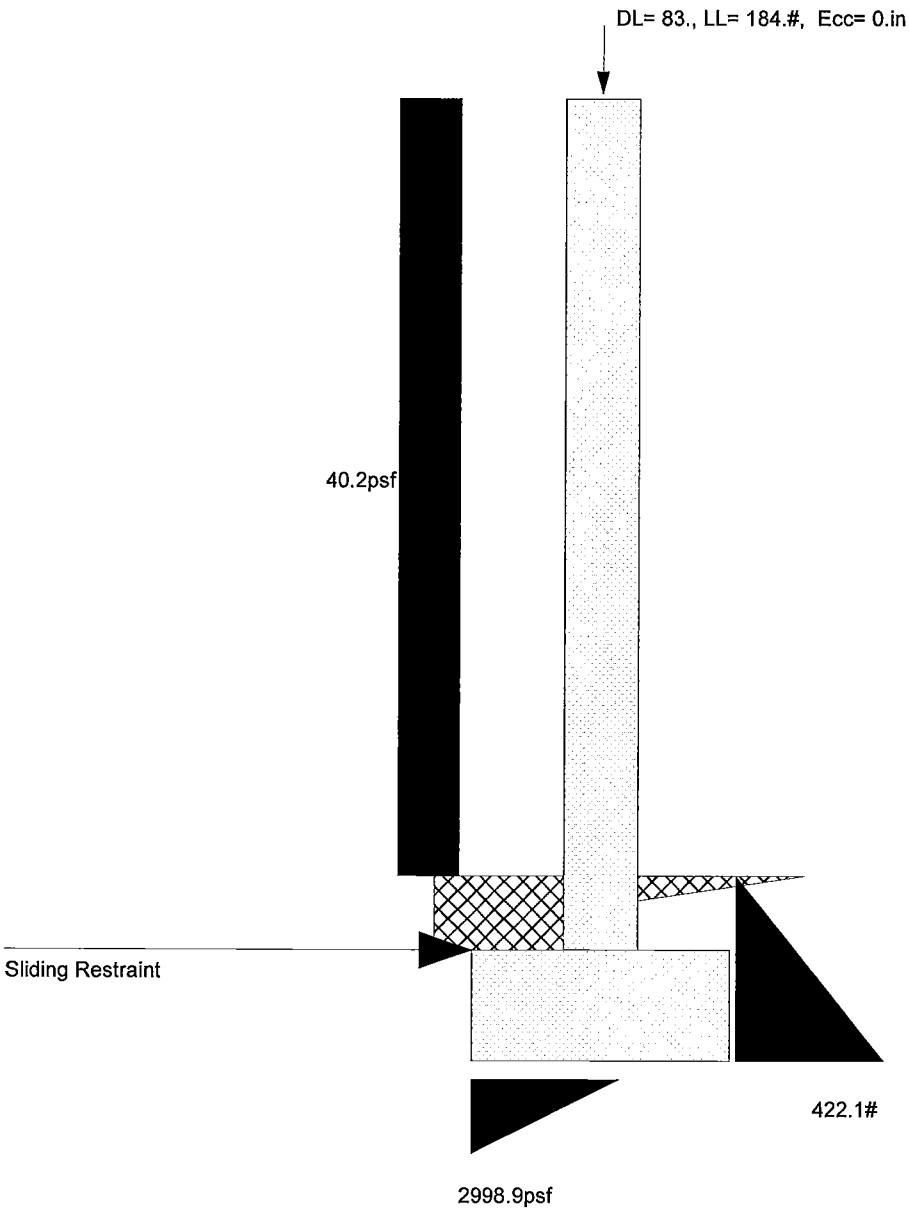
* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

DESIGNER NOTES:

BFD3


12.in Conc w/ #5 @ 16.in o/c
10'-6"
11'-6"
1'-0"
2"
1'-0"
Sliding Restraint
1'-6"
3"
#5@18.in @ Toe
#5@18.in @ Heel
Designer select
all horiz. reinf.
See Appendix A
1'-3"
2'-3"
3'-6"

BFD 4



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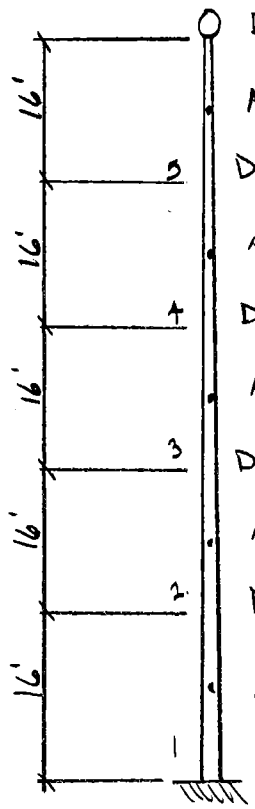
Misc Site Designs



Dia = 4" A = 6.4 FT²
AVG = 4.8" Cd = 0.94

Dia = 5.6 A = 8.5 FT²
AVG = 6.4" Cd = 0.65

Dia = 7.2" A = 10.7 FT²
AVG = 8.0" Cd = 0.48

Dia = 8.8" A = 12.8 FT²
AVG = 9.6" Cd = 0.45

Dia = 10.4" A = 14.9 FT²
AVG = 11.2" Cd = 0.45

Dia = 12"

Wind Pressures Per Segments

SEG #	WIND P (PSF)	WIND F (lbs)	L (FT)	M (LB·FT)
5	40.2	241.8	72'	17409.6
4	27.8	153.6	56'	8601.6
3	20.5	105.3	40'	4212
2	19.2	110.6	24'	2654
1	19.2	128.7	8'	1030

$\Sigma = 740\#$ $\Sigma = \underline{33907\ \#' = M_P}$ ⇒ Moment due to wind on pole

Moment due to flag load ⇒ $M_F = W_F(L - b/2) = 286\#(80' - 15'/2)$

$\underline{M_F = 20735\ \#'}$

Total Bending Moment = 33907 #' + 20735 #' = $\underline{54642\ \#' = M_T}$

Total Shear = 740 # + 286 # = $\underline{1026\ \# = V_T}$

Misc Site Designs

Flag Pole Design

Compressive Stress = 1300 lbs / $\pi(12''/2)^2 - \pi(11.25''/2)^2$ = 95 psi = f_a

Bending Stress = 54642 #' / $0.098175\left(\frac{12^4 - 11.25^4}{12}\right)$ = 16987 psi = f_b

Shear Stress = 2 (1020 #) / $\pi(6)^2 - \pi(5.625)^2$ = 150 psi = f_v

Check Combined Stresses

$$\frac{f_a}{0.6F_y} + \frac{f_b}{C_a F_b} + \left(\frac{f_v}{F_v}\right)^2 < 1.0$$

$I = 0.049087\,(d^4 - d_1^4) = 231.6\ in^4$

$S = 0.098175\left(\frac{d^4 - d_1^4}{d}\right) = 38.6\ in^4$

$d_1 = 12'' - 2 \times 0.375'' = 11.25''$

$d = 12''$

$F_y = 25$ ksi

$F_b = 24$ ksi

$F_v = 11.33$ ksi

$$C_a = 1 - \left(0.38\,D_p L^2 / 2.46\,E I_B\right) \div 0.52 = 1 - \left[\frac{(0.38 \times 1300\,\# \times 80^2 \times 12^2)}{2.46 \times 10 \times 10^6 \times 231.6}\right] / 0.52$$

$C_a = 0.846$

$$\frac{95\text{ psi}}{0.6 \times 25\text{ ksi}} + \frac{16987\text{ psi}}{0.846 \times 24\text{ ksi}} + \left(\frac{150\text{ psi}}{11.33\text{ ksi}}\right)^2 = 0.84 < 1.0$$

⇒ **Flag Pole OK** ✓

Project Description: (Committal shelter)

The expansion project for the VA National Cemetery in Zachary, Louisiana consists of three one-story structures and one shelter/canopy structure. The project is located in a hurricane zone with 110mph basic wind speed; requiring hurricane ties of all roof elements down to the foundation at all the buildings.

The Administration building is a one story building. The roof consists of prefabricated wood trusses with plywood sheathing supported by light-gage metal stud exterior bearing walls. The walls use Sureboard for shear resistance and bear on continuous footings.

The maintenance building is a pre-manufactured one-story metal building. The roof is part of the pre-manufactured design and consists of Z purlins and WF beams that span to WF columns. Rod bracing provides lateral resistance and it is also designed by the manufacturer. The columns will bear on spread footings and the building has a continuous footing all around. There is a bulk storage structure that is considered part of the maintenance building. Roof structure consists of wood framing with plywood sheathing. Wood stud non-bearing walls sit on top of 11' tall concrete walls, which sit on continuous footings.

There is also a shelter structure that has a pre-manufactured roof and lateral system comprised of tube members. The columns will bear on spread footings that will be connected with grade beams.

Design Criteria:

Building Type: Occupancy Category: I - Bulk Storage, Committal Shelter Structure
II – Administration and Maintenance Buildings

Design Codes: 2009 International Building Code
ASCE 7-05: Minimum Design Loads for Buildings and Other Structures, including *Supplement 1*
AISC 360-05: Specification for Structural Steel Buildings (13th Edition AISC)
ACI 318-08: Building Code Requirements for Structural Concrete
AF&PA 2005 NDS and 2008 SDPWS

Wind Criteria: 110 mph, Exposure C $I_w = 1.00$ $GC_{pi} = \pm 0.18$ (Enclosed Structure)

Seismic Criteria: Site Class: D $S_S = 0.122$ $S_1 = 0.055$ $S_{DS} = 0.130$ $S_{D1} = 0.088$

Seismic Design Category: B $I = 1.00$ $I_p = 1.0$

Lateral Force-Resisting System:

Administration Building
Light-gage metal stud bearing shear walls
$R = 6.5$ $\Omega_o = 3.0$ $C_d = 4.0$

Bulk Storage Building
Wood bearing and concrete shear walls
$R = 4.0$ $\Omega_o = 2.5$ $C_d = 4.0$

Maintenance Building/Committal Shelter
Light-gage metal stud bearing shear walls
$R = 3.5$ $\Omega_o = 3.0$ $C_d = 3.0$

Foundation Criteria:

Soils Report #870 CM 3022 by W.L.Burle Engineers, P.A., dated 2/15/2011

Allowable Bearing Pressures: (Non-structural elements)	DL	2300 psf
	DL + LL	2300 psf
	DL + LL + EQ/1.4	3067 psf
Allowable Lateral Pressures:	Retaining At-Rest Pressure	55 pcf
	Passive Pressure	300 pcf
	Coefficient of Friction	0.30

D: Roof Loads (Committal Shelter)

	deck	seismic weight
metal roofing (20ga)	1.8	1.8
Z' purlins (6"x16ga @ 24"cc)	1.5	1.5
beams	2.2	2.2
columns	1.0	1.0
sprinklers	1.0	1.0
MEP	2.0	2.0
misc	1.0	1.0
DPO - dead load	10.5 psf	10.5 psf
LLR - reducible roof LL	20.0 psf	0.0 psf
TL	30.5 psf	10.5 psf

E: Wall Loads

Wall Finish:	Light-Gage	Wood	Concrete
6"x16ga mtl studs @ 16"cc	1.5		
2x6 wood studs @ 16"cc		1.5	
12" concrete			150.0
Metal siding (both sides)		3.0	
1/2" shtg (both sides)		3.4	
3 5/8" brick veneer	35.0		
1/2" Sureboard	4.3		
5/8" gyp board	2.8	2.8	
misc	1.4	1.3	
DPO - non-composite DL	45.0 psf	12.0 psf	150.0 psf

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LATERAL ANALYSIS (Committal Shelter)

BLDG WT

ROOF (11 PSF) (20' x 38') = 10.9 k
MISC = 1.1 k
12.0 k

Seismic Force

V_{base} = 0.0372 (12k) = 0.45 k (ult.) (in both directions)

Wind Force

$V_{wind} = q_h G C_N$

q_h = 31 PSF @ 110 mph

G = 0.85

C_{NW} = 1.1
C_{NL} = -0.4 @ $\gamma = 0°$ (perp to ridge)

C_N = -0.6 @ $\gamma = 90°$ (parallel to ridge)

V_{WIND} = 31 PSF (0.85) (1.1 + 0.4) (5' x 38') = 7.51 k (N/S Direction)

31 PSF (0.85) (0.9) (32' x 5'/2) = 1.89 k (E/W Direction)

∴ WIND GOVERNS

LATERAL ANALYSIS (Approximation)

Bldg manufacturer to design structure, these calculations are to determine the approximate size of the footings.

LOAD Distribution to frames

N/S Direction

$F = 7.51\,k / 2\text{ FRAMES} = 3.76k$

E/W Direction

$F = 1.89\,k / 2\text{ FRAMES} = 0.95k$

FRAME FDN DESIGN

WORST CASE = 3.76k

$OTM = 3.76k\,(15.5') = 58.3\,k'$

$T/C = 58.3k' / 19' = 3.07\,k'$

⇒ 9' ⊞ + 7' ⊞ ftgs ok by inspection

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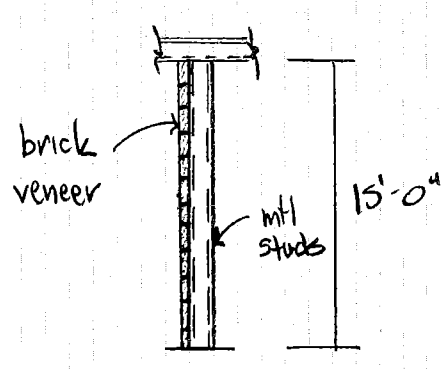
WALL DESIGN (Committal Shelter)

$P_{wind} = 27.1$ PSF



$W_{wind} = 27.1 \text{ PSF } (16/12) = 36.1 \text{ PLF}$

$W_{DL} = (1.5 \text{ PSF} + 35 \text{ PSF} + 2.8 \text{ PSF})\ 16/12 = 52.3 \text{ PLF}$

$P_{DL} = 52.3 \text{ PLF} \times 15' = 786 \#$

$M_{wind} = 36.1 \text{ PLF} \times 15'^2/8 = 1015 \#' = 12.18 \text{ k}''$

$I_{reqd} = 675\,(36.1 \text{ PLF})(15')^3/29000 \text{ ksi} = 2.83 \text{ in}^4$ ⇒ Try 6"x16ga studs

$I = 2.86 \text{ in}^4$

bridging @ 48" o.c.

$kl/r = 4' \times 12/0.57 = 84.2 < 4.71\sqrt{\frac{29000}{33}} \text{ ksi} = 139.6$

$F_e = \frac{\pi^2 \times 29000}{(84.2)^2} = 40.4 \text{ ksi}$ $\quad F_{cr} = (0.658^{33/40.4})\,33 \text{ ksi} = 23.4 \text{ ksi}$

$P_n/\Omega = 23.4 \text{ ksi} \times 0.556 \text{ in}^2/1.67 = 7.79 \text{ k}$

$\frac{0.786 \text{ k}}{7.79 \text{ k}} + \frac{12.18 \text{ k}''}{21.17 \text{ k}''} = 0.67 < 1.0$ ⇒ **use 6"x16ga studs @ 16" o.c.**

SITE MISC DESIGNS

80' TALL FLAG POLE

12" dia @ base
↓ (5 segments)
4" dia @ top

MATERIAL = 6063 Aluminum T6 temper $F_y = 25$ ksi $F_{sy} = 14$ ksi $E = 10 \times 10^6$ psi
THICKNESS = 0.375" $F_b = 24$ ksi $F_v = 11.33$ ksi

WEIGHT = 1200 lbs + 100 lbs for flag = 1300 lbs

WIND PRESSURE ⇒ $P = 0.00256 \times V^2 \times C_d \times C_h \times G$

$V = 110$ mph

$C_d = 129 / \left(110 \times \frac{(12+4)}{24} \right)^{1.3} = 0.48$

$C_h = 1.21$

$G = 1.14$

$\Rightarrow P = 0.00256\,(110\text{ mph})^2\,(0.48)(1.21)(1.14) = 20.5$ psf

WIND FORCE ⇒ $W_p = P \times A \times C_d$

$W_p = 20.5\text{ psf} \times \left[\frac{12" + 4"}{2 \times 12} \times 80' \right] \times 0.48$

FLAG POLE LOAD ⇒ $W_p = 525$ # (see next page for a more detailed analysis)

WIND FORCE ⇒ $W_F = 0.0014 \times V^2 \times \sqrt{A_F} \times C_h \times G$

$W_F = 0.0014\,(110\text{ mph})^2\,(10' \times 15')^{1/2} \times 1.21 \times 1.14$

FLAG LOAD ⇒ $W_F = 286$ #

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MISC SITE DESIGNS

FLAG POLE FOOTING

$M_{e\,base} = 54.64\ k'$

$P_{POLE} = 1.3\ k$

Non-constrained foundation $\Rightarrow$ $d = 0.5A\left\{1 + \left[1 + (4.36h/A)\right]^{1/2}\right\}$

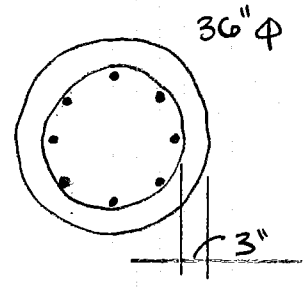
Assume 3' dia footing w/ 12' embed.

$A = 2.34\,(1026\#) \Big/ 100\ PSF\,(4')(3') = 2.0$

$d = 0.5(2)\left\{1 + \left[1 + \left(4.36 \times 50'/2\right)\right]^{1/2}\right\} = 11.5'$ $\Rightarrow$ <u>use 3'φ x 12' deep footing</u>

CHECK REINFORCEMENT

$M_{base} = 1.6 \times 54.64\,k' = 87.4\ k'$



$A_{s\,req'd} = \dfrac{87.4\,k'(12)}{0.9 \times 60ksi \times 0.875 \times 33''} = 0.67\ in^2$

$A_{s\,min} = 0.0014(\pi)(18'')^2 = 1.43\ in^2 / 6\,bars = 0.24$

$\Rightarrow$ TRY 6 - #5 bars

$\phi M_n = 0.9 \times 33'' \times 60ksi\,(6 \times 0.31) = 1657\,k'' = 138\,k'$

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Misc Site Designs

Columbarium Design

$S_{DS} = 0.130$ * Soil type D

$S_{D1} = 0.088$

* For columbarium, wind forces will govern by inspection.

Wind Forces

$q_z = 0.00256\, K_z\, K_{zt}\, K_d\, V^2\, I$ (ASCE7-05 6-15)

$K_z = 0.85$
$K_{zt} = 1.0$
$K_d = 0.85$
$I = 1.0$
$V = 110$ mph

$q_z = 22.4$ PSF

$F = q_z\, G\, C_f\, A_s$ (ASCE7-05 6-27)

$F = 22.4$ PSF $(0.85)(1.60)\, A_s$

$F = 30.5$ PSF $\times (29.25' \times 9.67') = 8627$ # ⇒ Wind governs

Seismic Weight

Caps = 150 PCF $[(2 \times 6.42' \times 0.5' \times 2.42') + (25.25' \times 4.83' \times 0.58')] = 12.94$ k

Niche = 150 PCF $[25.25' \times 7' \times 0.25] = 6.63$ k

↳ Structure is not solid

CMU Ends = 80 PSF $[(5.21' \times 1.36' \times 9.67')2] + 38$ PSF $[2(5.83' + 2' + 1.25') \times 9.67'] = 17.64$ k

37.2 k

$V_{base} = 0.13(37.2k) = 4836$ #

For Foundation

see Retain Pro Analysis.

Approx Wt per foot (worst case) = 1660 #

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Cantilevered Retaining Wall Design

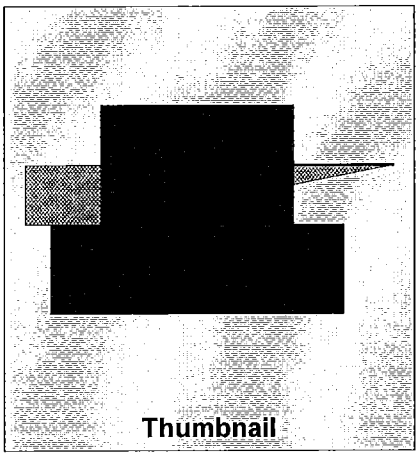
Code: IBC 2006

Criteria

Retained Height	=	1.00 ft
Wall height above soil	=	1.00 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	12.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	3,067.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	165.0 psf/ft
Toe Active Pressure	=	165.0 psf/ft
Passive Pressure	=	330.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	1,660.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	48.8 #/ft
...Height to Top	=	10.67 ft
...Height to Bottom	=	1.00 ft
The above lateral load has been increased by a factor of		1.60
Wind on Exposed Stem	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Design Summary

Wall Stability Ratios

Overturning	=	3.66	OK
Sliding	=	5.52	OK
Total Bearing Load	=	4,341 lbs	
...resultant ecc.	=	9.56 in	
Soil Pressure @ Toe	=	1,355 psf	OK
Soil Pressure @ Heel	=	134 psf	OK
Allowable	=	3,067 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,897 psf	
ACI Factored @ Heel	=	188 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	0.7 psi	OK
Allowable	=	82.2 psi	

Sliding Calcs (Vertical Component NOT Used)

Lateral Sliding Force	=	471.9 lbs	
less 100% Passive Force	= -	866.3 lbs	
less 100% Friction Force	= -	1,736.6 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	0.0 lbs	OK

Load Factors

Building Code	IBC 2006
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

Top Stem

		Stem OK
Design Height Above Ftg	ft =	0.00
Wall Material Above "Ht"	=	Concrete
Thickness	=	46.00
Rebar Size	=	# 5
Rebar Spacing	=	18.00
Rebar Placed at	=	Edge

Design Data

fb/FB + fa/Fa	=	0.067
Total Force @ Section	lbs =	471.9
Moment....Actual	ft-# =	2,753.5
Moment.....Allowable	=	40,905.4
Shear.....Actual	psi =	1.4
Shear.....Allowable	psi =	82.2
Wall Weight	=	575.0
Rebar Depth 'd'	in =	44.19
LAP SPLICE IF ABOVE	in =	12.00
LAP SPLICE IF BELOW	in =	
HOOK EMBED INTO FTG	in =	9.59

Lap splice above base reduced by stress ratio

Masonry Data

f'm	psi =	
Fs	psi =	
Solid Grouting	=	
Modular Ratio 'n'	=	
Short Term Factor	=	
Equiv. Solid Thick.	=	
Masonry Block Type	=	Medium Weight
Masonry Design Method	=	ASD

Concrete Data

f'c	psi =	3,000.0
Fy	psi =	60,000.0

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Cantilevered Retaining Wall Design

Code: IBC 2006

Footing Dimensions & Strengths

Toe Width	=	1.00 ft
Heel Width	=	4.83
Total Footing Width	=	5.83
Footing Thickness	=	18.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 3,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	1,897	188 psf
Mu' : Upward	=	900	142 ft-#
Mu' : Downward	=	235	233 ft-#
Mu: Design	=	665	91 ft-#
Actual 1-Way Shear	=	0.00	0.72 psi
Allow 1-Way Shear	=	82.16	82.16 psi

Toe Reinforcing = # 5 @ 18.00 in
Heel Reinforcing = # 5 @ 18.00 in
Key Reinforcing = None Spec'd

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S * Fr
Heel: Not req'd, Mu < S * Fr
Key: No key defined

Summary of Overturning & Resisting Forces & Moments

Item		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#
Heel Active Pressure	=	515.6	0.83	429.7
Surcharge over Heel	=			
Toe Active Pressure	=	-515.6	0.83	-429.7
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=	471.9	7.34	3,461.4
Load @ Stem Above Soil	=			
Total	=	471.9	**O.T.M.** =	3,461.4

Resisting/Overturning Ratio = **3.66**
Vertical Loads used for Soil Pressure = 4,341.4 lbs

		.....RESISTING..... Force lbs	Distance ft	Moment ft-#
Soil Over Heel	=	109.6	5.33	584.5
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=	1,660.0	2.92	4,841.7
* Axial Live Load on Stem	=			
Soil Over Toe	=	110.0	0.50	55.0
Surcharge Over Toe	=			
Stem Weight(s)	=	1,150.0	2.92	3,354.2
Earth @ Stem Transitions	=			
Footing Weight	=	1,311.8	2.92	3,823.8
Key Weight	=			
Vert. Component	=			
Total	=	4,341.4 lbs	**R.M.=**	12,659.1

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

DESIGNER NOTES:

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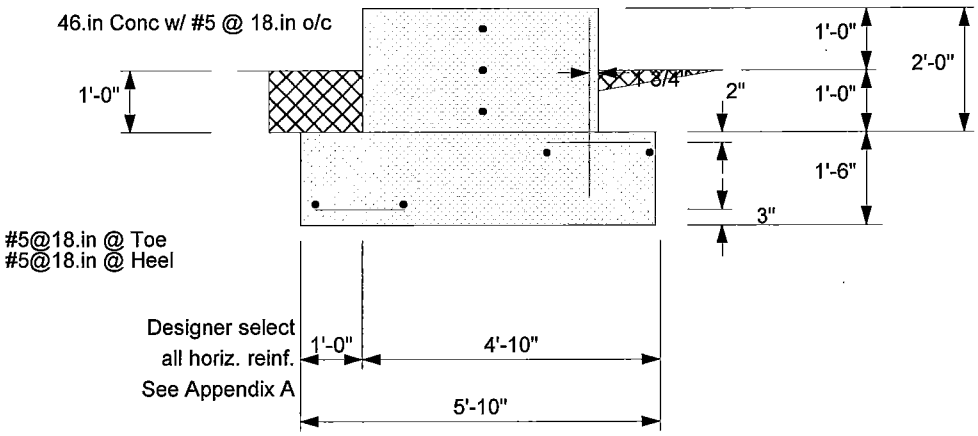
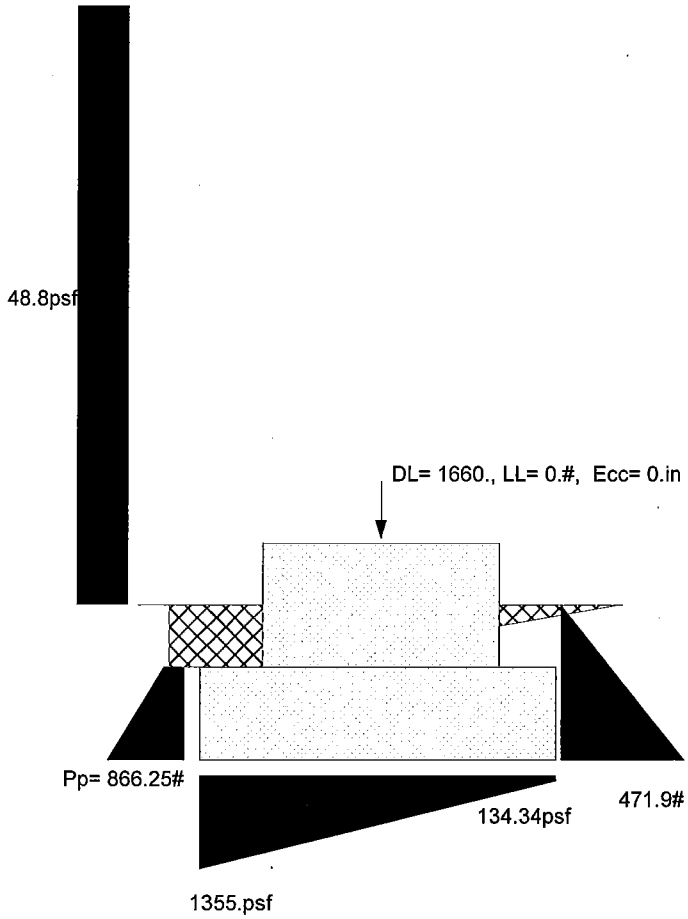
SM7


46.in Conc w/ #5 @ 18.in o/c
1'-0"
3/4"
2"
1'-0"
1'-0"
2'-0"
1'-6"
3"
#5@18.in @ Toe
#5@18.in @ Heel
Designer select
all horiz. reinf.
See Appendix A
1'-0"
4'-10"
5'-10"

SM8