JOB TITLE	
JOB NO. 60268249 Task 01	CALCULATION NO.
ORIGINATOR LC	DATE 5/30/12
REVIEWER	DATE
SCALE	SHEET NO. 1 OF

CALCULATE WIND LOAD USING ASCE 7-10 (Chapter 28)

RISK CATEGORY I (Table 1.5-1)

BASIC WIND SPEED V = 105 mph (Figure 26.5-1C)

WIND DIRECTIONALITY FACTOR, K_d = 0.85 (Table 26.6-1)

EXPOSURE CATEGORY : EXPOSURE B

TOPOGRAPHIC FACTOR, K_{zt} = 1.0

ENCLOSURE CLASSIFICATION ENCLOSED (Table 26.11-1)

INTERNAL PRESSURE COEFFICIENT $GC_{pi} = \pm 0.18$ (Table 26.11-1)

$K_z = 0.70$ for $z < 30'$ (Table 28.3-1)

$q_z = 0.00256 K_z K_{zt} K_d V^2 = 0.00256 \times 0.7 \times 1.0 \times 0.85 \times (105\ \text{mph})^2 = 16.793$ psf

$a = \min(10\% \times 14', 0.4 \times 12.9458', 4\% \times 14', 3') = \min(1.4', 5.18', 0.56', 3') = 0.56'$
(Figure 28.4-1)

GC_{pf} = 0.4 windward wall
= -0.69 windward roof
= -0.37 leeward roof
= -0.29 leeward wall
= 0.61 windward wall @ corner for 2a distance
= -1.07 windward roof @ corner for 2a distance
= -0.53 leeward roof @ corner for 2a distance
= -0.43 leeward wall @ corner for 2a distance

Roof Angle 0°-5°

Load Case A
Wind Perpendicular to
Long Side of Bldg

Grid: 4x4 = 1"

JOB TITLE

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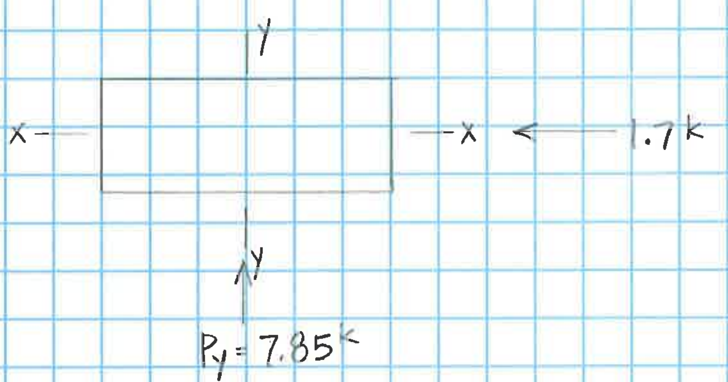
CALCULATE WIND LOAD USING ASCE 7-10 (Chapter 28)

GC_{pf} = 0.40 on windward wall ⑤
= −0.29 on leeward wall ⑥
= −0.45 on side walls ①&④
= −0.69 windward roof ②
= −0.37 leeward roof ③
= −0.48 on leeward wall @ corner for a distance
= −1.07 windward roof @ corner for a distance
= −0.53 leeward roof @ corner for a distance
= 0.61 windward wall @ corner for a distance
= −0.43 on leeward wall @ corner for a distance

Load Case B
Wind Perpendicular to Short side of Bldg

See Wind Load.xls for calculation of pressures.



$M_x = 7.85^k \times (12.9458' + 2')\tfrac{1}{2} = 58.7$ k ft

$M_y = 1.7^k \times (12.9458' + 2')\tfrac{1}{2} = 12.7$ k ft

Assume 2' thick base mat to be conservative.

Grid: 4x4 = 1"

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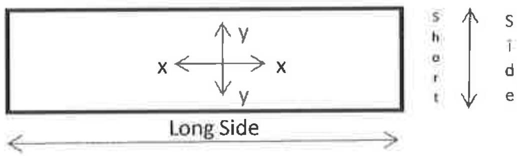
Sheet No. ________ of ________

Wind Load

qh (psf) = 16.793



Bldg Plan

	p (psf) max	p (psf) min	Load Case A GCpf	Load Case A GCpi (max)	Load Case A Gcpi (min)	Load Case B GCpf	Load Case B GCpi (max)	Load Case B GCpi (min)	Location
1	9.740	3.694	0.4			0.4			Windward Wall
2	-7.893	-4.870	-0.29			-0.29			Leeward Wall
3	-14.610	-11.587	-0.69			-0.69			Windward Roof
4	-9.236	-6.213	-0.37			-0.37			Leeward Roof
5	13.266	10.244	0.61	0.18	-0.18	0.61	0.18	-0.18	Windward Wall @ Corner
6	-10.244	-7.221	-0.43			-0.43			Leeward Wall @ Corner
7	-20.991	-17.969	-1.07			-1.07			Windward Roof @ Corner
8	-11.923	-8.900	-0.53			-0.53			Leeward Roof @ Corner
9	-10.580	-7.557	-			-0.45			Side Walls

Positive and negative signs signify pressures acting toward and away from the surfaces, respectively.
Load Case A: Wind Perpendicular to the Long Side of Building.
Load Case B: Wind Perpendicular to the Short Side of Building.

Height of Wall (ft) =	12.9458
Long Side Length (ft) =	34
Short Side Length (ft) =	14
"a" from corner (ft) =	0.56

Load Case	A	B
Wind parallel to y-y direction (k) =	7846.4	0
Wind parallel to x-x direction (k) =	-	1681.276

No torsional effects are considered.
Uplift on the roof is not considered for the design of the foundation.

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CALCULATE SEISMIC LATERAL FORCE USING ASCE 7-10

Consider the generator and container equivalent/similar to steel storage racks in order to use Chapter 15.5 as prescribed in Chapter 11.1.3 . 15.5.3.1 refers to Chapter 15.3

Consider the superstructure = 90^k (generator & container)

Weight of Spread Footing = 150 pcf × 15.3333' × 35.3333' × 2' = 162.53^k

$$\frac{90^k}{90^k + 162.53^k} = 0.36 > 0.25 \Rightarrow \text{Use Chapter 15.3.2}$$

Assume the fundamental period of the non building structure is less than 0.06s and follow Chapter 15.5.

Design for minimum seismic force according to Chapter 15.5.3.

$R_p = R = 4$ (steel storage racks) (Table 15.4-1)

$a_p = 2.5$ (Chapter 15.5.3.1)

$I_p = 1.0$ (Chapter 13.1.3)

z = midheight of container = 12.9458' × 1/2 = 6.473'

$W_p \approx 90^k$

Assume Site Class D due to lack of information (Chapter 11.4.2)

From USGS website (https://geohazards.usgs.gov/secure/designmaps/us/regions.php):

$0.2015 \le S_S \le 0.2228$ Use 0.21 (average)

$0.06126 \le S_1 \le 0.06429$ Use 0.063 (average)

$F_a = 1.6$ (Table 11.4-1)

$F_v = 2.4$ (Table 11.4-2)

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(F_a S_s) = \frac{2}{3} \times 1.6 \times 0.21 = 0.224$

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(F_v S_1) = \frac{2}{3} \times 2.4 \times 0.063 = 0.10$

Seismic Design Category B ASCE 7-10 Tables 11.6-1 & 11.6-2.

$$F_p = \frac{0.4 a_p S_{DS} W_p}{(R_p/I_p)}\left(1 + 2\frac{z}{h}\right) = \frac{0.4(2.5)\,0.224 \times 90^k}{(4/1.0)}\left(1 + 2 \times \tfrac{1}{2}\right) = 10.1^k \text{ (Chapter 13.3.1)}$$

$F_p(\max) = 1.6 S_{DS} I_p W_p = 1.6 \times 0.224 \times 1 \times 90^k = 32.3^k$

$F_p(\min) = 0.3 S_{DS} I_p W_p = 0.3 \times 0.224 \times 1 \times 90^k = 6.05^k$

$M_x = M_y = 10.1^k (12.9458' + 2')\,\tfrac{1}{2} = 75.3$ k-ft Controls lateral design

REFERENCE

Printed 5/31/12 from https://geohazards.usgs.gov/secure/designmaps/us/regions.php

Download Counties CSV file

JOB NO. 60268249

CALCULATION NO.

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DATE 5/31/12

REVIEWER

DATE

SCALE

SHEET NO. 5 OF

CALCULATE ROOF SNOW LOAD USING ASCE 7-10 (Chapter 7)

$C_e = 0.9$ (Table 7-2)

$C_t = 1.1$ (Table 7-3)

$I_s = 0.8$ (Table 1.5-2)

$P_g = 30$ psf (Figure 7-1)

$P_f = 0.7 C_e C_t I_s P_g = 0.7 \times 0.9 \times 1.1 \times 0.8 \times 30$ psf $= 16.632$ psf

Grid: 4x4 = 1"

GRAVITY LOADS

SELF WEIGHT ≈ 90^k (container & generator) ⇒ $90^k/(15.3333' \times 35.3333') = 0.166$ ksf

CONCRETE SELF WEIGHT = 0.15 kcf $\times 15.3333' \times 35.3333' \times 2'$ (assumed)
= 162.53k
⇒ 0.15 kcf $\times 2' = 0.3$ ksf

LIVE LOAD = 0.15 ksf $\times 34' \times 14' = 71.4^k$
⇒ 0.15 ksf

IMPACT LIVE LOAD (due to machinery) = $0.7 \times$ dead load $= 0.7(90^k + 162.53^k)$
= 176.8k
⇒ $176.8^k/(15.3333' \times 35.3333') = 0.326$ ksf

DL = 0.466 ksf = 0.166 ksf + 0.3 ksf
LL = 0.476 ksf = 0.15 ksf + 0.326 ksf

ALLOWABLE STRESS LOAD COMBINATIONS (ASCE 7-10 Chapter 2.4)

1. DL Does not control
2. DL+LL
3. DL+SNOW does not control
4. DL+0.75LL+0.75 SNOW Does not control
5. DL+0.6WIND Does not control
6. DL+0.7SEISMIC
7. DL+0.75LL+0.75(0.6 WIND)+0.75(SNOW) Does not control
8. DL+0.75LL+0.75(0.7SEISMIC)+0.75(SNOW)
9. 0.6DL+0.6 WIND
10. 0.6DL+0.7SEISMIC

CALCULATE ALLOWABLE STRESS ON SPREAD FOOTING.

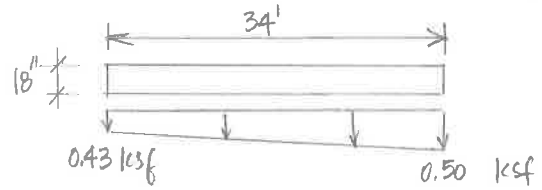
ALLOWABLE BEARING PRESSURE = 3 ksf (Geotech Report from Golder Associates, February 2003)

ALLOWABLE LOAD COMBINATION 2 STRESS : DL+LL = 0.466 ksf + 0.476 ksf = 0.942 ksf < 3 ksf OK

ALLOWABLE LOAD COMBINATION 6 : DL + 0.7 SEISMIC

$$\sigma = \frac{P}{A} \pm \frac{Mc}{I} = 0.466\ \text{ksf} \pm \frac{75.3\ \text{k-ft} \times 0.7 \times \frac{1}{2} \times 15.33' \times 12}{35.33 \times (15.33')^3} = (0.466 \pm 0.038)\ \text{ksf} < 3\ \text{ksf OK}$$

= 0.50 ksf, 0.43 ksf (compressive)



Moment about y-y axis (page 2 diagram)

ALLOWABLE LOAD COMBINATION 8 : DL + 0.75LL + 0.75 (0.7SEISMIC) + 0.75 SNOW

P/A = 0.466 ksf + 0.476 ksf × 0.75 + 0.75 (0.0166 ksf) = 0.835 ksf

M = 75.3 k-ft × 0.75 × 0.7 = 39.533 k-ft (Moment about y-y axis)

$$\sigma = \frac{P}{A} \pm \frac{Mc}{I} = 0.835\ \text{ksf} \pm \frac{39.533\ \text{k-ft} \times 12 \times \frac{15.3333'}{2}}{(35.3333') \times (15.3333')^3} = (0.835 \pm 0.0286)\ \text{ksf} < 3\ \text{ksf}$$

= 0.864 ksf, 0.806 ksf (compressive)

ALLOWABLE LOAD COMBINATION 9 : 0.6DL + 0.6 WIND

P/A = 0.6 × 0.466 ksf = 0.28 ksf

M_y = 0.6 × 12.7 k-ft = 7.62 k-ft

M_x = 0.6 × 58.7 k-ft = 35.22 k-ft

$$\sigma_y = \frac{P}{A} \pm \frac{M_y c}{I_y} = 0.28\ \text{ksf} \pm \frac{7.62\ \text{k-ft} \times 12 \times \left(\frac{35.3333'}{2}\right)}{(15.3333') \times (35.3333')^3} = (0.28 \pm .0024)\ \text{ksf} = 0.28\ \text{ksf}, 0.28\ \text{ksf} < 3\ \text{ksf OK}$$

(compressive)

$$\sigma_x = \frac{P}{A} \pm \frac{M_x c}{I_x} = 0.28\ \text{ksf} \pm \frac{35.22\ \text{k-ft} \times 12 \times \left(\frac{15.3333'}{2}\right)}{(35.3333') \times (15.3333')^3} = (0.28 \pm 0.025)\ \text{ksf} = 0.305\ \text{ksf}, 0.25\ \text{ksf} < 3\ \text{ksf OK}$$

(compressive)

CALCULATE ALLOWABLE STRESS ON SPREAD FOOTING

ALLOWABLE LOAD COMBINATION 10: 0.6DL + 0.7SEISMIC

$P/A = 0.6 \times 0.466 \text{ ksf} = 0.28 \text{ ksf}$

$M_y = 75.3 \text{ k-ft} \times 0.7 = 52.71 \text{ k-ft}$

$$\sigma = \frac{P}{A} \pm \frac{M_y c}{I_y} = 0.28 \text{ ksf} \pm \frac{52.71 \text{ k-ft} \times \left(\frac{15.3333'}{2}\right) \times 12}{35.3333' \times (15.3333')^3} = 0.32 \text{ ksf}, 0.24 \text{ ksf} < 3 \text{ ksf} \quad \underline{\text{OK}}$$

NOTE: I_x is selected when moment due to seismic forces is used since it yields a greater overall stress.

ALLOWABLE LOAD COMBINATION 2: DL+LL CONTROLS

Check kern location.

Load combination 8. $P = 0.835 \text{ ksf} \times 35.3333' \times 15.3333' = 452.4 \text{ k}$

$M = 39.533 \text{ k-ft}$

$$e = \frac{M}{P} = \frac{39.533 \text{ k-ft}}{452.4 \text{ k}} = 0.087' < \frac{15.3333'}{6} = 2.6' \quad \underline{\text{OK}}$$

Load combination 10: $P = 0.28 \text{ ksf} \times 35.3333' \times 15.3333' = 151.7 \text{ k}$

$M = 52.71 \text{ k-ft}$

$$e = \frac{M}{P} = \frac{52.71 \text{ k-ft}}{151.7 \text{ k}} = 0.35' < \frac{15.3333'}{6} = 2.6' \quad \underline{\text{OK}}$$

DESIGN REINFORCEMENT IN SPREAD FOOTING

Check reinforcement recommended by SAFE model (TRRF Generator Foundation.fdb)
Design moments in both directions are negligible. Use a nominal value of $7^{k\text{-}ft}$ to design.

$d \approx 18'' - 3'' - 5/8'' \times 1/2 = 14.69''$ assume 3" cover

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{A_s (60\,ksi)}{0.85 (4\,ksi)\, 12 \times 7'} = 0.21 A_s$$

$$\phi M_n = 0.9 A_s f_y (d - a/2) = 0.9 \times A_s \times 60\,ksi\,(14.69'' - 0.21 A_s (1/2)) = 54(14.69 - 0.105 A_s) A_s$$
$$= -5.6723 A_s^2 + 793.26 A_s \geq M_u = 7\ k\text{-}ft \times 12 = 84\ k\text{-}in$$

$$0.106\ in^2 \leq A_s^{req'd} \leq 139.74\ in^2$$

Provide #5 @ 12". $0.31\ in^2 \times \left(\frac{84''}{12} + 1\right) = A_s = 2.48\ in^2 > 0.106\ in^2$ OK

Check section is tension controlled (ACI 318-11)
$c = a/\beta_1 = (0.21 \times 2.48\ in^2)/0.85 = 0.6127\ in$ (10.2.7.1 & 10.2.7.3)

$$\frac{\varepsilon_t}{d-c} = \frac{\varepsilon_{cu}}{c} \Rightarrow \varepsilon_t = \varepsilon_{cu}\left(\frac{d-c}{c}\right) = 0.003\left(\frac{14.69'' - 0.6127''}{0.6127''}\right) = 0.06893 > 0.005$$
(10.3.4)

∴ Section is tension controlled. $\phi = 0.9$

Provide temperature & shrinkage reinforcement
$\rho = 0.0018$ (Section 7.12)
$A_s = 0.0018 \times 84'' \times 18'' = 2.72\ in^2$
Distribute #4 @ 12" top in both directions. $A_s = 0.2\ in^2 \times (7+1) \times 2 = 3.2\ in^2$

Check shear
$\phi V_n = 0.75\,(2\sqrt{f'_c})\,bd = 1.5\sqrt{4000\,psi} \times 84'' \times 14.69'' = 117.^k > V_u = 7.5 \times 10^{-7}\,k$ OK

Shear reinforcement is not required.
USE THE SAME FOOTING DESIGN FOR THE PEDESTAL FOOTING.

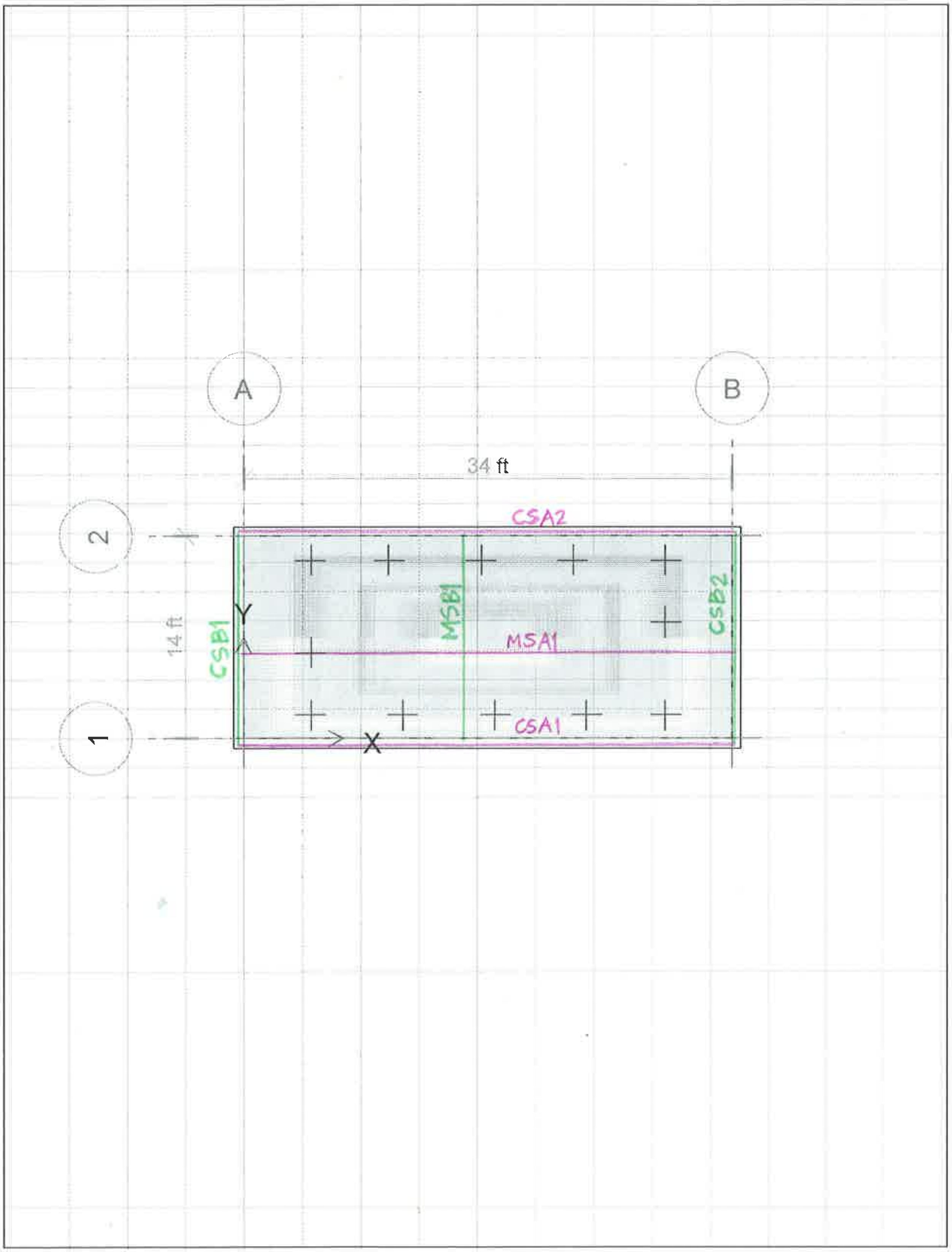

A
B
34 ft
CSA2
2
14 ft
CSB1
Y
MSB1
MSA1
CSB2
1
X
CSA1

TABLE: Strip Forces - Summary

Strip	SpanID	Location	OutputCase	CaseType	AbsMaxP	AbsMaxV2	AbsMaxT	MaxM3	MinM3
Text	Text	Text	Text	Text	kip	kip	kip-ft	kip-ft	kip-ft
CSA1	Span 1	Start	DCONU1	Combination	0	-1.991E-07	-9.985E-08	1.697E-07	2.404E-08
CSA1	Span 1	Middle	DCONU1	Combination	0	1.386E-07	-2.296E-07	0.000001271	6.805E-07
CSA1	Span 1	End	DCONU1	Combination	0	2.399E-07	1.062E-07	1.075E-07	2.333E-08
CSA2	Span 1	Start	DCONU1	Combination	0	-1.368E-07	6.93E-08	2.799E-07	1.804E-08
CSA2	Span 1	Middle	DCONU1	Combination	0	1.023E-07	-7.298E-08	0.000001151	6.529E-07
CSA2	Span 1	End	DCONU1	Combination	0	0.000000168	-1.028E-07	2.353E-07	2.406E-08
CSB1	Span 1	Start	DCONU1	Combination	0	3.761E-08	7.541E-08	2.624E-08	9.51E-09
CSB1	Span 1	Middle	DCONU1	Combination	0	-4.653E-08	-4.598E-08	6.035E-08	3.526E-08
CSB1	Span 1	End	DCONU1	Combination	0	-2.439E-08	-6.828E-08	4.757E-08	1.354E-08
CSB2	Span 1	Start	DCONU1	Combination	0	4.758E-08	-9.411E-08	6.971E-08	3.485E-08
CSB2	Span 1	Middle	DCONU1	Combination	0	-7.365E-08	7.746E-08	1.301E-07	8.794E-08
CSB2	Span 1	End	DCONU1	Combination	0	-3.087E-08	1.119E-07	9.285E-08	2.087E-08
MSA1	Span 1	Start	DCONU1	Combination	0	-1.542E-07	4.93E-08	0.00000068	-1.107E-08
MSA1	Span 1	Middle	DCONU1	Combination	0	-1.472E-07	-1.165E-07	0.000001974	0.000001132
MSA1	Span 1	End	DCONU1	Combination	0	1.616E-07	-9.901E-08	6.958E-07	-2.192E-08
MSB1	Span 1	Start	DCONU1	Combination	0	-3.309E-07	2.447E-07	8.508E-07	-3.785E-07
MSB1	Span 1	Middle	DCONU1	Combination	0	1.246E-07	1.609E-07	0.000001182	0.000001069
MSB1	Span 1	End	DCONU1	Combination	0	2.415E-07	8.975E-08	6.269E-07	-0.00000023
CSA1	Span 1	Start	DCONU2	Combination	0	-4.478E-07	-2.245E-07	3.818E-07	5.44E-08
CSA1	Span 1	Middle	DCONU2	Combination	0	3.112E-07	-5.163E-07	0.000002859	0.00000153
CSA1	Span 1	End	DCONU2	Combination	0	5.392E-07	2.389E-07	2.411E-07	5.372E-08
CSA2	Span 1	Start	DCONU2	Combination	0	-3.079E-07	1.559E-07	6.292E-07	3.994E-08
CSA2	Span 1	Middle	DCONU2	Combination	0	0.00000023	-1.641E-07	0.000002587	0.000001468
CSA2	Span 1	End	DCONU2	Combination	0	3.777E-07	-2.312E-07	5.292E-07	5.354E-08
CSB1	Span 1	Start	DCONU2	Combination	0	8.459E-08	1.696E-07	5.924E-08	2.252E-08
CSB1	Span 1	Middle	DCONU2	Combination	0	-1.044E-07	-1.034E-07	1.358E-07	7.928E-08
CSB1	Span 1	End	DCONU2	Combination	0	-5.418E-08	-1.535E-07	1.067E-07	3.009E-08
CSB2	Span 1	Start	DCONU2	Combination	0	1.067E-07	-2.116E-07	1.569E-07	7.742E-08
CSB2	Span 1	Middle	DCONU2	Combination	0	-1.656E-07	1.741E-07	0.000000293	1.977E-07
CSB2	Span 1	End	DCONU2	Combination	0	-6.86E-08	2.514E-07	2.084E-07	4.587E-08
MSA1	Span 1	Start	DCONU2	Combination	0	-3.469E-07	1.108E-07	0.000001529	-2.525E-08
MSA1	Span 1	Middle	DCONU2	Combination	0	-0.000000332	-2.618E-07	0.000004437	0.000002544
MSA1	Span 1	End	DCONU2	Combination	0	3.624E-07	-2.226E-07	0.000001565	-5.043E-08
MSB1	Span 1	Start	DCONU2	Combination	0	-7.453E-07	5.503E-07	0.000001913	-8.526E-07
MSB1	Span 1	Middle	DCONU2	Combination	0	2.774E-07	3.617E-07	0.000002656	0.000002405
MSB1	Span 1	End	DCONU2	Combination	0	5.429E-07	2.016E-07	0.000001409	-5.169E-07

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TABLE: Load Combinations								
Combo	Load	SF	Type	DSStrength	DSServInit	DSServNorm	DSServLong	AutoDesign
Text	Text	Unitless	Text	Yes/No	Yes/No	Yes/No	Yes/No	Yes/No
DCONU1	DEAD	1.4	Linear Add	Yes	No	No	No	Yes
DCONU2	DEAD	1.2	Linear Add	Yes	No	No	No	Yes
DCONU2	LIVE	1.6						

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TABLE: Load Assignments - Surface Loads						
Area	LoadPat	Dir	UnifLoad	A	B	C
Text	Text	Text	lb/ft2	lb/ft3	lb/ft3	lb/ft2
MAT	LIVE	Gravity	476	0	0	0
MAT	DEAD	Gravity	166	0	0	0

TABLE: Concrete Slab Design Summary 02 - Span Definition Data								
Strip	SpanID	SpanLength	StartDist	EndDist	GlobalX1	GlobalY1	GlobalX2	GlobalY2
Text	Text	ft	ft	ft	ft	ft	ft	ft
CSA1	Span 1	35.3334	0	0	-0.6667	0	34.6667	0
CSA2	Span 1	35.3334	0	0	-0.6667	14	34.6667	14
CSB1	Span 1	15.3334	0	0	0	-0.6667	0	14.6667
CSB2	Span 1	15.3334	0	0	34	-0.6667	34	14.6667
MSA1	Span 1	35.3334	0	0	-0.6667	7	34.6667	7
MSB1	Span 1	15.3334	0	0	17	-0.6667	17	14.6667

DESIGN PEDESTAL ANCHOR BOLTS (FOR RADIATOR SUPPORT)

$P_{U1} = 1.4(4.5^k + \underbrace{0.49\,kcf \times 13.5' \times 4.43\,in^2/144}_{0.203^k} + \underbrace{(4.43\,in^2/144 \times 0.49\,kcf \times 11/2)}_{0.083^k}) = 6.7^k$ say 7^k (horizontal W6 beam)

$P_{U2} = 1.2\,(4.5^k + 0.203^k + 0.083^k) + 1.6\,(1.0^k) = 7.34^k$ say 7.5^k

Use 7.5^k

USE EQUATION ON SHEET 4 TO DETERMINE SEISMIC LATERAL FORCE

$$F_p = \frac{0.4 a_p S_{DS} W_p}{(R_p/I_p)}\left(1+2\frac{z}{h}\right) = \frac{0.4 \times 2.5 \times 0.224 \times 14^k}{(4/1.0)}(1+2\times1) = 2.352^k \text{ say } 2.4^k$$

ASSUME $z = h$ = height of attachment to component = entire height of column/pedestal

ALL OTHER COEFFICIENTS ASSUMED TO BE SAME AS GENERATOR CONTAINER

ASSUME WEIGHT OF RADIATOR $= 14^k$ (from Smithco manufacturer) $= W_p$

WIND LATERAL FORCE $=$ 600 lb / COL $\sim 2.352^k/4$ COL $\Rightarrow$ USE 0.6^k/column $= V_u$

$M_u = 0.6^k \times 12.5'$ (height of steel column) $= 7.5^{k\text{-}ft}$

TENSION FROM DE-COUPLED MOMENT $= \dfrac{M_u}{5.53'} = \dfrac{7.5^{k\text{-}ft}}{5.53'} = 1.36^k = T_u$ (shortest distance between columns)

NET AXIAL FORCES: $7.5^k + 1.36^k = 8.9^k$ (compression) & 5.04^k (compression) $= 7.5^k - 1.36^k - 1.6\times1^k + 0.5\times1^k$

NOTE: DESIGN FORCES ARE NOT FROM THE SAME LOAD COMBINATION.

USE ACI 318-11 APPENDIX D.

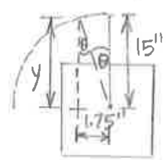
$h_{ef} = 9''$ embedded, $12''$ long

$c_{a1} = 14.897''$

$c_{a2} = 15'' - 1.75'' = 13.25''$

Diameter of Concrete Base $= 2'\text{-}6'' = 30''$

Depth of Concrete Base $= h_a = 6.5'$ (height of pedestal)



$\sin\theta = \dfrac{1.75''}{15''}$

$\theta = 6.7°$

$\tan\theta = \dfrac{1.75''}{y}$

$y = 1.75''/\tan 11.537°$
$= 14.897''$ say $14.9''$

DESIGN AS POST-INSTALLED ANCHOR SINCE IT HAS MORE STRINGENT REQUIREMENTS.

CHECK SHEAR ONLY. NO TENSILE FORCES.

USE ϕ FOR CONDITION B: NO SUPPLEMENTARY REINFORCEMENT

CHECK STEEL STRENGTH OF ANCHOR IN SHEAR (D.6.1)

$V_{sa} = 0.6\, A_{se,v}\, f_{uta} = 0.6[\pi(1/2'')^2]\ 114\ ksi = 53.72^k$

$f_{uta} = \min(1.9 f_{ya}, 125\ ksi) = \min(1.9\times 60\ ksi, 125\ ksi) = 114\ ksi$

grout pad factor

$\phi V_{sa} = 0.65 \times 53.72^k \times 0.8 = 27.9^k/bolt > 0.6^k/2\ bolt = 0.3^k/bolt$ OK

CHECK CONCRETE BREAKOUT STRENGTH IN SHEAR (D.6.2)

$A_{vco} = 4.5(c_{a1})^2 = 4.5(14.897'')^2 = 998.6\ in^2$

(use larger c_a to be conservative)

$A_{vc} = 1.5\, c_{a1}(1.5 c_{a1} + c_{a2}) = 1.5(13.25'')(1.5\times 13.25'' + 14.897'') = 698.5\ in^2$

(use smaller c_a to be conservative)

$\psi_{ed,v} = 0.7 + 0.3\dfrac{c_{a2}}{1.5 c_{a1}} = 0.7 + 0.3 \times \dfrac{13.25''}{14.897''\times 1.5} = 0.88$

$\psi_{c,v} = 1.0$ cracked concrete without supplementary reinforcement

$\psi_{h,v} = 1.0 \quad h_a > 1.5 c_{a1}$

$\to l_e = h_{ef} = 9''$

$V_{b1} = \left(7\left(\dfrac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right)\lambda_a\sqrt{f'_c}\,(c_{a1})^{1.5} = \left(7\left(\dfrac{9''}{1''}\right)^{0.2}\sqrt{1''}\right)0.8\times 1.0\sqrt{4000\ psi}\times(13.25'')^{1.5}$

$= 26.5^k$

$V_{b2} = 9\lambda_a\sqrt{f'_c}\,(c_{a1})^{1.5} = 9\times 0.8\times 1.0\times\sqrt{4000\ psi}\times(13.25'')^{1.5} = 21.96^k$

$V_b = \min(V_{b1}, V_{b2}) = 21.96^k$

$\phi V_{cb} = 0.7\dfrac{A_{vc}}{A_{vco}}\psi_{ed,v}\psi_{c,v}\psi_{h,v}V_b = 0.7\times\dfrac{698.5\ in^2}{998.6\ in^2}\times 0.88\times 1.0\times 1.0\times 21.96^k$

$= 9.5^k/bolt > 0.3^k/bolt$ OK

CHECK CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

$V_{cp} = k_{cp} N_{cp} = 2\times 13.7^k/bolt = 27.4^k/bolt$

$N_{cp} = N_{cb} = \dfrac{A_{Nc}}{A_{Nco}}\psi_{ed,N}\psi_{c,N}\psi_{cp,N}N_b = \dfrac{722.25\ in^2}{729\ in^2}\times 0.994\times 1.0\times 0.6\times 23.2^k = 13.7^k/bolt$

$A_{Nco} = 9\, h_{ef}^2 = 9\times(9'')^2 = 729\ in^2$

$A_{Nc} = (1.5 h_{ef} + c_{a1})(2\times 1.5 h_{ef}) = (1.5\times 9'' + 13.25'')(3\times 9'') = 722.25\ in^2$

ORIGINATOR LC DATE 6/25/12

REVIEWER DATE

SCALE SHEET NO. 17 OF

CHECK CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR

$$\psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} = 0.7 + 0.3 \times \frac{13.25''}{1.5 \times 9''} = 0.994$$

$$\psi_{c,N} = 1.0$$

$c_{ac} = 2.5h_{ef} = 22.5''$ for undercut anchors (D.8.6)

$$\psi_{cp,N} = \frac{c_{a,min}}{c_{ac}} = \frac{13.25''}{22.5''} = 0.59$$

$$\frac{1.5h_{ef}}{c_{ac}} = \frac{1.5 \times 9''}{22.5''} = 0.6$$

Use 0.6

$$N_b = k_c \lambda_a \sqrt{f'_c}\, h_{ef}^{1.5} = 17 \times 0.8 \times 1.0 \times \sqrt{4000\,psi}\,(9'')^{1.5} = 23.2^k$$

$$\phi V_{cp} = 0.7 \times 27.4^k/bolt = 19.2^k/bolt > 0.3^k/bolt \quad \underline{OK}$$

Grid: 4x4 = 1"