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ENGINEERING CALCULATION SHEET

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E-mail: dlfeng@dlfengineers.com

CUSTOMER Field Vineyards

LOCATION

SHEET NO. Cover
JOB NO. 3133.1.1.15
DATE 3/26/2015
BY RB CHK'D

Metal Building Foundation
Field Vineyards
27801 River Road
Cloverdale, CA

CONTENTS	SHEETS
Calculations	1-27
TRASH ENCLOSURE	1-5

REGISTERED PROFESSIONAL ENGINEER
DAVID PLUM
C 45688
CIVIL
STATE OF CALIFORNIA

Exp: 12/31/2016

Shts: 1-27
1-5

STAMP CA

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CUSTOMER Field Vineyards

LOCATION

SHEET NO. 1 of 27
JOB NO. 3133.1.1.15
DATE 3/27/2015
BY RB CHK'D

Metal Building Foundation

Frame Loads from CBC Buildings: Job: Field Winery Cloverdale, CA

Job #: C14C0564

Date: 1/27/2015

Foundation per 2013 CBC: Standard Spread Footing Bearing = 1500 psf D+L
2000 psf D+L+W/E
Lateral Bearing = 100 psf/ft x 1.33 = 133 psf/ft
Sliding Friction = 130 psf x 1.33 = 173 psf

Geotech Report: Giblin Associates
Job # 4095.1.1 May 19, 2009
Supplemental Letter: Reese & Associates
Job # 209.1.13 Jan 25, 2010
Job # 209.1.13 Dec 21, 2012

Spread Ftg Bearing = 2000 psf D+L
= 3000 psf D+L+W/E
Passive Pressure = 300 psf
Sliding Friction = 0.3

Frame Loads: Lines 1, 6

		Line C		Line A	
		Vertical	Horizontal	Vertical	Horizontal H2
	Dead	3.0	1.1	3.1	-1.1
	Co-Lat	3	1.2	3	-1.2
	Live	5.2	2.1	6.1	-2.1
	Snow				
USD	Wind	-9.3	-5	-10.4	-1
			1.0		5.0
USD	EQ ±	2.5	4.1	2.5	4.1

Wind Post Line 1 & 6

D = 1.0
C =
L =

W = ±8.2
W horiz=
E =
E horiz=

Frame Loads: Lines 3, 4

		Line C		Line A	
		Vertical	Horizontal	Vertical	Horizontal
	Dead	5.8	1.6	5.1	-1.6
	Co-Lat	4.7	1.9	4.7	-1.9
	Live	15.4	2.1	8.3	-2.1
	Snow				
USD	Wind	-34.2	-6.3	-16.1	-3.4
			2.8		6.4
USD	EQ ±	3.9	5.4	3.9	6.2

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ENGINEERING CALCULATION SHEET

CUSTOMER Field Vineyards

LOCATION

SHEET NO. 2
JOB NO. 3133.1.1.15
DATE 3/26/2015
BY RB CHK'D

Frame Loads: Lines 2, 5

		Line C		Line A	
		Vertical	Horizontal	Vertical	Horizontal H2
	Dead	5.5	1.8	5.2	-1.8
	Co-Lat	5.8	1.9	4.7	-1.9
	Live	10.7	3.2	9.4	-3.2
	Snow				
USD	Wind	-19.2	-8.7	-17.2	-0.8
			0.2		7.0
USD	EQ ±	3.9	5.4	3.9	6.2

Post Line 2 & 5

	Vertical	Horizontal
D =	1.3	
C =	1.2	
L =	4.3	-0.2
W =	-6.2, +0.2	-0.4
W horiz=		
E =	±0.6	
E horiz=		

Frame Loads: Lines 1A, 5A

		Line D		Line C1	
		Vertical	Horizontal	Vertical	Horizontal
	Dead	0.7		0.8	
	Co-Lat	0.7	0.1	0.6	-0.1
	Live	1.8	0.1	1.7	-0.1
	Snow				
USD	Wind	-2.9	-2.6	-5.6	-0.8
		3.2	3.1	2.5	1.3
USD	EQ ±	1.1	0.7	1.1	0.3

Side Wall Bracing Line A

		Line 2, 3, 4, 5	
USD	Wind ±	3.21	3.9
USD	EQ ±	11.3	13.8

	Line A-B	Portal Frame		Line C	
		Line 4		Line 5	
	Dead	1.6	0.1	1.6	-0.1
USD	Wind ±	9.1	4.6	9.1	4.6
USD	EQ ±	34	17.1	34	17.1

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CUSTOMER ______________________ SHEET NO. 3
JOB NO. ______
LOCATION ______________________ DATE ______
BY ______ CHK'D ______

FRAME FTGS

MIN SIZE VERTICAL LOAD DOWN

LINES 1, 6

$$P = D + C + L = 3.1 + 3 + 6.1 = 12.2^k$$

$$D + C + EQ = 3.1 + 3 + .7(2.5) = 7.8^k$$

$$D + C + .75L + .75(.7)EQ = 3 + 3 + .75(6.1) + .75(.7)2.5 = 11.9^k$$

$$\text{MIN SIZE} = \sqrt{\frac{12.2}{2.0}} = 2.5^{\phi}$$

TRY 3.0 SQ × 18" DEEP

SIZE REQUIRED FOR VERTICAL UP WIND

$$0.6(10.4) = .6\Big[\underbrace{(3.1 + 3)}_{D+C} + \underbrace{1.5(L)^2(.15)}_{FTG} + \overset{SLAB}{\frac{5}{12}(.15)5 \times 5}\Big] + \overset{FRICTION}{4(1)L(.173)}$$

$$0 = 0.135L^2 + 0.692L - 1.6$$

$$L_{MIN} = 1.8' \text{ SQ}$$

USE 3'-0" SQ × 18" DEEP
w/(4) #4 E.W. T&B.

HORIZONTAL MAX

$$H = D + C + L = 1.1 + 1.2 + 2.1 = 4.4^k$$

$$D + C \pm W = 1.1 + 1.2 + 1(.6) = 2.9$$

$$= 1.1 + 1.2 - 5(.6) = -0.7$$

$$D + C \pm EQ = 1.1 + 1.2 + 4.1(.7) = 5.2^k$$

* $$D + C + .75L + .75(.7)EQ = 1.1 + 1.2 + .75(2.1) + .75(.7)4.1 = 6.1^k$$

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SHEET NO. 4

CUSTOMER

JOB NO.

DATE

LOCATION

BY

CHK'D

FRAME LINE 1, 5 CONT.

HORIZONTAL LOAD

PASSIVE LAT BEARING = 300(1.5)3 = 1.35
FRICTION ON SIDES = 173(1)(3.5)2 = 1.0
FRICTION ON BOT = 10.3(6.1) = 1.83
FRICTION ON SLAB = 173(7)(7) = 8.5

Σ 12.7^k > 6.1^k

D+L+ FTG
3.1 + 3 + 2 = 6.1^k

∴ ok

USE 3'-6 SQ × 18" DEEP FTG
w/ (4) #4 T & B

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SHEET NO. 5
CUSTOMER
JOB NO.
DATE
LOCATION
BY CHK'D

FRAME LINES 3 & 4

MIN SIZE VERTICAL DOWN — LINE C — LINE A

P = D+C+L = 5.8 + 4.7 + 15.4 = 25.9^k — 18.1^k

D+C+EQ = 5.8 + 4.7 + 3.9(.7) = 13.3^k — 12.6^k

D+C+.75L+.75EQ = 5.8+4.7+.75(15.4)+.75(.7) 3.9
= 24.1^k — 18.0^k

MIN SIZE = $\sqrt{\frac{25.9}{2.0}}$ = 3.6' SQ — 3.5' SQ

TRY 4'-0" SQ

SIZE FOR UPLIFT. @ PORTAL DOWN 5.8+4.7+1.6+34(.7) = 35.9
q_s = 35.9/5.25×6.33 = 1081 PSF
WIND

LINE C $0.6(34.1) = .L[(5.8+4.7) + 1.5(L)^2.15 + \frac{.5}{2}(.15)10(10)] + 4(1)L(.173)$

$0 = 0.135L^2 + 0.692L - 10.4$

L = 6.6' SQ × 18" DEEP

LINE 4
5'-3" × 6'-4" @ PORTAL

LINE C TRY 24" DEEP

PORTAL FRAME UPLIFT = 34^k
USE SAME FTG 2'-7½" EA WAY OF COL @ PORTAL

$0 = 0.18L^2 + 1.038L - 10.4$

L = 5.25' × 24"

USE 5'-3" SQ × 24" DEEP
@ LINE C
w/ (6) #5 E.W. T.&B.

LINE A

$0.6(16.1) = .16[(5.1+4.7) + 1.5(L)^2(.15) + \frac{.5}{2}(.15)10(10)] + 4(1)L(.173)$

$0 = 0.135L^2 + 0.692L - 0.1$

MIN 1'-0" SQ

USE 4'-0" SQ × 18" DEEP
w/ (5) #4 EW T&B
@ LINE A

NOTE SIDE WALL
BRACING UPLIFT 11.3(.7) = 8^K < .6(16.1) = 9.7^k
∴ FRAME LOAD GOVERNS

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SHEET NO. 6
CUSTOMER
JOB NO.
DATE
LOCATION
BY CHK'D

FRAMES LINES 2 & 5

MIN SIZE DOWN

$P = D+C+L = 5.5 + 5.8 + 10.7 = 22^k$

$D+C+EQ = 5.5 + 5.8 + 3.9(.7) = 14^k$

@ PORTAL WIND $= 5.5 + 5.8 + 1.6 + .75(10.7) + .75(.6)9.1 = 25.1^k$

EQ $= 5.5 + 5.8 + 1.6 + 34(.7) = 36.7^k$

MIN PAD LINE A/C

$\sqrt{\frac{22}{2}} = 3.3^{\#}$

USE 4'-0" SQ x 18"

LINE C @ PORTAL

$\sqrt{\frac{36.7}{3}} = 3.5^{\#}$

L SAME AS LINE 4 @ PORTAL
5'-3" x 6'-4"

UPLIFT LINE 2 (A & C) LINE 5 A

$.6(19.2) = .6\left[(5.5 + 5.8) + 1.5(L)^2 .15 + \frac{5}{12}(.15)10(10)\right] + 4(1)(L).173$

$0 = 0.135L^2 + 0.692L - 1.0$

$L_{min} = 1.2$ MIN

USE 4'-0" SQ x 18" DEEP
w/ (5) #4 EW T & B

UPLIFT @ PORTAL
USE SAME FTG AS LINE 4 C

5'-3" x 6'-4" x 2'-0" DEEP
w/ (7) #4 LONG T&B
(6) #4 TRANV T&B

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SHEET NO. 7
CUSTOMER
JOB NO.
DATE
LOCATION
BY CHK'D

LEAN TO FRAME @ LINE 2 & 5

VERTICAL LOAD DESIGN

$P = D + C + L = 1.3 + 1.2 + 4.3 = 6.8^k$

$\text{MIN PAD} = \sqrt{\frac{6.8}{2.0}} = 1.8'\ \text{SQ}$

UPLIFT $= 6.2^k$ WIND USD

$\therefore .6(6.2) = 3.72^k$

FTG 3'-0" SQ x 18"

$3 \times 3 \times 1.5 \times .15 = 2^k$

RESISTING DEAD LOAD

$$= 0.6\Big(\underbrace{1.3 + 1.2}_{D+C} + \underbrace{2.0}_{FTG} + \underbrace{\tfrac{5}{12}(.15)\,7 \times 7}_{SLAB}\Big)$$

$= 4.5^k > 3.72^k \therefore$ ok

USE 3'-0" SQ x 18" FTG
w/ (4) #4 E.W. T & B

HORIZ LOAD $= 0.6^k \therefore$ ok BY INSPECTION

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SHEET NO. 8

CUSTOMER

JOB NO.

DATE

LOCATION

BY CHK'D

FRAME LINES 1A & 5A

VERTICAL LOAD DOWN

P = D+C+L = .7 + .7 + 1.8 = 3.2^k
or .8 + .6 + 1.7 = 3.1^k

WITH WIND DOWN = .7+.7 + 3.2(.6) = 3.32^k

MIN FTG $\sqrt{\frac{3.4}{2.0}} = 1.4'$ Ø   3×3×18"

UPLIFT MAX = WIND 5.6^k USD
= 5.6×.6 = 3.4^k

FTG WT = 2^k

SLAB WT = $\frac{5}{12}$(.15) 7×7 = 3.1^k

Σ RESISTING R = 0.6(.7+.7 + 2 + 3.1) = 3.9^k > 3.4^k
ok

USE 3'-0" SQ × 18" DEEP
w/ (4) #4 EW T&B

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SHEET NO. 9

CUSTOMER

JOB NO.

DATE

LOCATION

BY CHK'D

ENDWALL POST LINE 1 & 6

$D = 1.0^k$

HORIZ $= 8.2^k (.6) = 4.92^k$ ASD

LATERAL BEARING $300(1.5)3 = 1.35^k$

FRICTION ON SLABS $.173(7)(7) = 8.4^k$

$9.75^k > 4.92^k$

ok

USE 3'-0" SQ x18" PAD
w/ (4) #4 EW T&B

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SHEET NO. 10
CUSTOMER
JOB NO.
DATE
LOCATION
BY CHK'D

HORIZONTAL LOADS

LINES 3 & 4

$D+C+L = 1.6+1.9+2.1 = 5.6^k$

$D+C+EQ = 1.6+1.9+6.2(.7) = 7.84^k$ GOVERNS

$D+C+W = 1.6+1.9-6.4(.6) = 0.34^k$

PASSIVE ON 4' FTG $300(1.5)4 = 1.8^k$

FRICTION ON SLAB $173(7)(7) = 8.5^k$

$10.3^k > 7.9^k$ ok ✓

LINES 2, 5

$D+C+L = 1.8+1.9+3.2 = 6.9^k$

$D+C+EQ = 1.8+1.9+6.2(.7) = 8.1^k$ GOVERNS

$D+C+W = 1.8+1.9-8.7(.6) = -1.52$

$1.8+1.9-7(.6) = -0.5$

SAME AS ABOVE ∴ ok

SIDE WALL BRACING LINE A

$H_{MAX\ EQ} = 13.8(.7) = 9.9^k$ ok LESS THAN ABOVE ∴ ok ✓

PORTAL FRAME

$H_{MAX} = 17.1(.7) = 12.0^k$

FTG PASSIVE $= 300(2)5.25 = 3.1^k + 8.5^k = 11.6^k \approx 12.0$

ACTUAL FULL SLAB TO RESIST. ∴ ok

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SHEET NO. 11

CUSTOMER

JOB NO.

DATE

LOCATION

BY CHK'D

CHECK A.B.

MAX UPLIFT.

WIND @ LINE C FRAME 3/4

$T_u = 0.9(5.8 + 4.7) - 34.2 = -24.8^k$ ON (4) BOLTS

$V_u = 0.9(1.6 + 1.9) - 6.3 = -3.2^k$ ON (4) BOLTS

∴ ON (2) BOLTS

$T_u = 12.4^k$

$V_u = 1.6^k$

USE 3/4"ϕ F1554 A.BS

PORTAL FRAME

$EQ = 34^k$ UP

$T_u = .9(1.6) - 34^k = -32.6$ ON (4) BOLTS

$V_u = 17.1^k$ ON (4) BOLTS

∴ ON (2) BOLTS $T_u = 16.3^k$

$V_u = 8.6^k$

V_u IN PEDESTAL $= 17.1^k$

$\phi V_c = \underline{.85}\sqrt{2500}(22)(31)$

$= 29^k > 17.1^k$ ∴ OK

USE 1"ϕ F1554 ABS @ PORTAL FRAME.

$M_u = 17.1(.5) = 8.6^{1-k}$

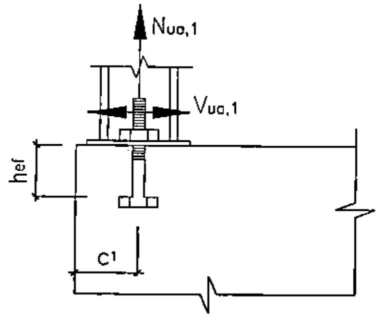
$As_{MIN} = 0.14 in^2$ #4 DOWELS OK

PROJECT : Metal Building Foundation
CLIENT : Field Vineyards
JOB NO. : 3333.1.1.14 DATE : 3/30/2015

PAGE : 12
DESIGN BY : RB
REVIEW BY :

Group of Tension and Shear Fasteners Near Two Edges Based on ACI 318-08 — Wind uplift @ Frames

INPUT DATA & DESIGN SUMMARY



CONCRETE STRENGTH f_c' = 2.5 ksi

SPECIFIED STRENGTH OF FASTENER f_{uta} = 58 ksi

(The strength of most fastenings is likely to be controlled by the embedment strength rather than the steel strength, so it is usually economical to use ASTM A307 Grade A fastener.)

FACTORED DESIGN TENSION LOAD $N_{ua,1}$ = 12.4 k

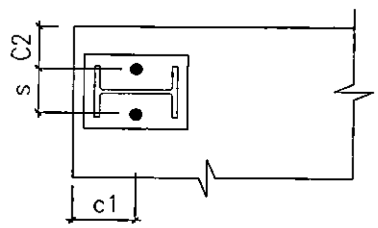
FACTORED DESIGN SHEAR LOAD $V_{ua,1}$ = 1.6 k

EFFECTIVE EMBEDMENT DEPTH h_{ef} = 12 in

FASTENER DIAMETER d = 0.75 in

FASTENER HEAD TYPE 3 Hex

(1=Square, 2=Heavy Square, 3=Hex, 4=Heavy Hex, 5=Hardened Washers)



FASTENER CENTER-TO-CENTER SPACIN(s = 4 in

DIST. BETWEEN THE FASTENER AND ED(c1 = 8.5 in

DIST. BETWEEN THE FASTENER AND ED(c2 = 6 in

SEISMIC LOAD ? (ACI 318 D3.3) n

[THE FASTENER DESIGN IS ADEQUATE.]

ANALYSIS

NUMBER OF FASTENERS n = 2

EFFECTIVE AREA OF FASTENER A_{se} = 0.334 in^2

BEARING AREA OF HEAD A_b = 0.654 in^2, (or determined from manufacture's catalogs.)

CHECK THE FASTENERS TENSILE STRENGTH : (ACI 318, D.5.1.2 & ASCE 14.2.2.17)

$$\phi N_s = \phi n A_{se}\left(f_{uta}\right) = 29.058 \text{ k} > N_{ua} = 12.400 \text{ k} \quad \text{[Satisfactory]}$$

where : ϕ = 0.75 x 1 = 0.75 , (ACI 318-08 D.4.4 & D.3.3.3)

CHECK CONCRETE BREAKOUT STRENGTH : (ACI 318, D.5.2.1)

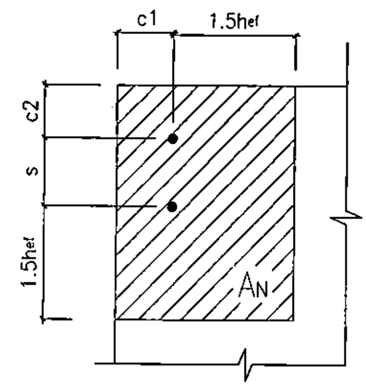
$$\phi N_{cbg} = \phi \frac{A_N}{A_{No}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} N_b = \phi \frac{A_N}{\left(9h_{ef}^2\right)} \psi_{ec,N} \left(0.7 + \frac{0.3c_{\min}}{1.5h_{ef}}\right) \psi_{c,N} \left(24\sqrt{f_c'}h_{ef}^{1.5}\right)$$

= 17.136 k > N_{ua} [Satisfactory]

where : ϕ = 0.75 x 1 = 0.75

$\psi_{ec,N}$ term is 1.0 for no eccentricity in the connection.

$\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur.



CHECK PULLOUT STRENGTH : (ACI 318, D.5.3.1)

$$\phi N_{pn} = \phi n \psi_{cp,N}\left(A_b 8 f_c'\right) = 19.620 \text{ k} > N_{ua} \quad \text{[Satisfactory]}$$

where : ϕ = 0.75 x 1 = 0.75

$\psi_{cp,N}$ term is 1.0 for location where concrete cracking is likely to occur.

CHECK SIDE-FACE BLOWOUT STRENGTH : (ACI 318, D.5.4.1)

c_{min} > 0.4 h_{ef} [Satisfactory]

Since this fastener is located far from a free edge of concrete (c > 0.4 hef) this type of failure mode is not applicable.

DETERMINE DESIGN TENSILE STRENGTH :

$$\phi N_n = \min\left(\phi N_s \ , \phi N_{cb} \ , \phi N_{pn}\right) = 17.136 \text{ K}$$

CHECK Fasteners SHEAR STRENGTH : (ACI 318, D.6.1.2b & ASCE 14.2.2.17)

$$\phi V_s = \phi n 0.6 A_{se} f_{ut} = 15.110 \text{ k} > V_{ua} = 1.600 \text{ k} \quad \text{[Satisfactory]}$$

where : ϕ = 0.65 x 1 = 0.65

(for built-up grout pads, first factor shall be multiplied by 0.8, ACI 318 D6.1.3)

CHECK CONCRETE BREAKOUT STRENGTH FOR SHEAR LOAD : (ACI 318, D.6.2.1b) (Cont'd)

$$\phi V_{cbg} = \phi \frac{A_V}{A_{Vo}} \psi_{ec,V}\psi_{ed,V}\psi_{c,V} V_b = \phi \frac{(1.5c1)(1.5c1+s+c2)}{4.5c1^2}\psi_{ec,V}\left(0.7+0.3\frac{c2}{1.5c1}\right)\psi_{c,V}\left(7\left(\frac{l}{d}\right)^{0.2}\sqrt{d}\sqrt{f'_c}\,c1^{1.5}\right)$$

= 6.408 k > V_{ua} [Satisfactory]

where : ϕ = 0.75 x 1 = 0.75

$\psi_{cp,N}$ term is 1.0 for no eccentricity in the connection.

$\psi_{c,V}$ term is 1.0 for location where concrete cracking is likely to occur.

l term is load bearing length of the anchor for shear, not to exceed 8d.

CHECK PRYOUT STRENGTH FOR SHEAR LOAD : (ACI 318, D.6.3.1)

$$\phi V_{cpg} = \phi k_{cp}\frac{A_N}{A_{No}}\psi_{ed,N}\psi_{c,N}N_b = \phi k_{cp}\frac{A_N}{(9h_{ef}^2)}\left(0.7+\frac{0.3c_{min}}{1.5h_{ef}}\right)\psi_{c,N}\left(24\sqrt{f'_c}\,h_{ef}^{1.5}\right)$$

= 34.272 k > V_{ua} [Satisfactory]

where : ϕ = 0.75 x 1 = 0.75

$\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur.

k_{cp} = 2.0 for $h_{ef} > 2.5$ in.

DETERMINE DESIGN SHEAR STRENGTH :

$$\phi V_n = \min\left(\phi V_s\ ,\phi V_{cb}\ ,\phi V_{cp}\right)$$ = 6.408 K

REQUIRED EDGE DISTANCES AND SPACING TO PRECLUDE SPLITTING FAILURE :

Since headed cast-in-place fasteners are not like to be highly torqued, the minimum cover requirements of ACI 318 Sec. 7.7 apply.

$Cover_{Provd}$ > $Cover_{Reqd}$ [Satisfactory]

CHECK TENSION AND SHEAR INTERACTION : (ACI 318, D.7)

Since $N_{ua,1}$ > $0.2\,\phi\,N_n$ and

$V_{ua,1}$ > $0.2\,\phi\,V_n$ the full design strength is not permitted.

The interaction equation must be used

$$\frac{N_{ua,1}}{\phi N_n}+\frac{V_{ua,1}}{\phi V_n}=$$ 0.97 < 1.2 [Satisfactory]

Summary of Dimensional Properties of Fasteners

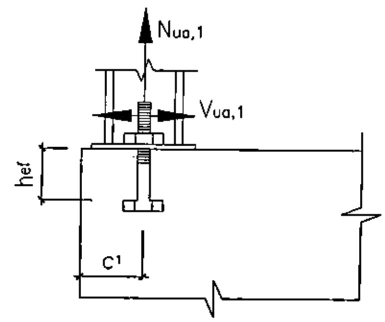
Fastener Diameter (in)		Gross Area of Fastener (in^2)	Effective Area of Threaded Fastener (in^2)	Bearing Area of Heads, Nuts, and Washers (A_b) (in^2)				
				Square	Heavy Square	Hex	Heavy Hex	Hardened Washers
0.250	1/4	0.049	0.032	0.142	0.201	0.117	0.167	0.258
0.375	3/8	0.110	0.078	0.280	0.362	0.164	0.299	0.408
0.500	1/2	0.196	0.142	0.464	0.569	0.291	0.467	0.690
0.625	5/8	0.307	0.226	0.693	0.822	0.454	0.671	1.046
0.750	3/4	0.442	0.334	0.824	1.121	0.654	0.911	1.252
0.875	7/8	0.601	0.462	1.121	1.465	0.891	1.188	1.804
1.000	1	0.785	0.606	1.465	1.855	1.163	1.501	2.356
1.125	1 1/8	0.994	0.763	1.854	2.291	1.472	1.851	2.982
1.250	1 1/4	1.227	0.969	2.288	2.773	1.817	2.237	3.682
1.375	1 3/8	1.485	1.160	2.769	3.300	2.199	2.659	4.455
1.500	1 1/2	1.767	1.410	3.295	3.873	2.617	3.118	5.301
1.750	1 3/4	2.405	1.900	-	-	-	4.144	6.541
2.000	2	3.142	2.500	-	-	-	5.316	7.903

Technical Reference:

1. Ronald Cook, "Strength Design of Anchorage to Concrete," PCA, 1999.

PROJECT : Metal Building Foundation		PAGE : 14
CLIENT : Field Vineyards		DESIGN BY : RB
JOB NO. : 3333.1.1.14	DATE : 3/30/2015	REVIEW BY :
Group of Tension and Shear Fasteners Near Two Edges Based on ACI 318-08		EQ uplift @ Portal Frames

INPUT DATA & DESIGN SUMMARY



CONCRETE STRENGTH f_c' = 2.5 ksi

SPECIFIED STRENGTH OF FASTENER f_{uta} = 58 ksi

(The strength of most fastenings is likely to be controlled by the embedment strength rather than the steel strength, so it is usually economical to use ASTM A307 Grade A fastener.)

FACTORED DESIGN TENSION LOAD $N_{ua,1}$ = 16.3 k

FACTORED DESIGN SHEAR LOAD $V_{ua,1}$ = 8.6 k

EFFECTIVE EMBEDMENT DEPTH h_{ef} = 12 in

FASTENER DIAMETER d = 1 in

FASTENER HEAD TYPE 3 Hex

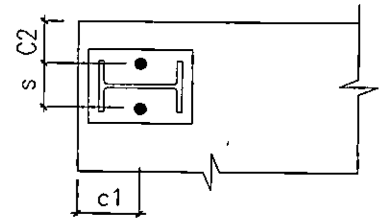
(1=Square, 2=Heavy Square, 3=Hex, 4=Heavy Hex, 5=Hardened Washers)

FASTENER CENTER-TO-CENTER SPACIN(s = 4 in

DIST. BETWEEN THE FASTENER AND ED(c1 = 12 in

DIST. BETWEEN THE FASTENER AND ED(c2 = 18 in

SEISMIC LOAD ? (ACI 318 D3.3) n



[THE FASTENER DESIGN IS ADEQUATE.]

ANALYSIS

NUMBER OF FASTENERS n = 2

EFFECTIVE AREA OF FASTENER A_{se} = 0.606 in^2

BEARING AREA OF HEAD A_b = 1.163 in^2, (or determined from manufacture's catalogs.)

CHECK THE FASTENERS TENSILE STRENGTH : (ACI 318, D.5.1.2 & ASCE 14.2.2.17)

$$\phi N_s = \phi n A_{se}(f_{uta}) = 52.722 \text{ k} > N_{ua} = 16.300 \text{ k} \quad \text{[Satisfactory]}$$

where : ϕ = 0.75 x 1 = 0.75 , (ACI 318-08 D.4.4 & D.3.3.3)

CHECK CONCRETE BREAKOUT STRENGTH : (ACI 318, D.5.2.1)

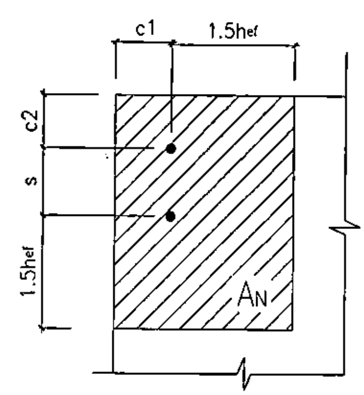
$$\phi N_{cbg} = \phi \frac{A_N}{A_{No}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} N_b = \phi \frac{A_N}{(9h_{ef}^2)} \psi_{ec,N}\left(0.7 + \frac{0.3c_{min}}{1.5h_{ef}}\right)\psi_{c,N}\left(24\sqrt{f_c'}h_{ef}^{1.5}\right)$$

$$= 31.177 \text{ k} > N_{ua} \quad \text{[Satisfactory]}$$

where : ϕ = 0.75 x 1 = 0.75

$\psi_{ec,N}$ term is 1.0 for no eccentricity in the connection.

$\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur.



CHECK PULLOUT STRENGTH : (ACI 318, D.5.3.1)

$$\phi N_{pn} = \phi n \psi_{cp,N}\left(A_b 8 f_c'\right) = 34.890 \text{ k} > N_{ua} \quad \text{[Satisfactory]}$$

where : ϕ = 0.75 x 1 = 0.75

$\psi_{cp,N}$ term is 1.0 for location where concrete cracking is likely to occur.

CHECK SIDE-FACE BLOWOUT STRENGTH : (ACI 318, D.5.4.1)

c_{min} > 0.4 h_{ef} [Satisfactory]

Since this fastener is located far from a free edge of concrete (c > 0.4 hef) this type of failure mode is not applicable.

DETERMINE DESIGN TENSILE STRENGTH :

$$\phi N_n = \min\left(\phi N_s, \phi N_{cb}, \phi N_{pn}\right) = 31.177 \text{ K}$$

CHECK Fasteners SHEAR STRENGTH : (ACI 318, D.6.1.2b & ASCE 14.2.2.17)

$$\phi V_s = \phi n 0.6 A_{se} f_{ut} = 27.415 \text{ k} > V_{ua} = 8.600 \text{ k} \quad \text{[Satisfactory]}$$

where : ϕ = 0.65 x 1 = 0.65

(for built-up grout pads, first factor shall be multiplied by 0.8, ACI 318 D6.1.3)

CHECK CONCRETE BREAKOUT STRENGTH FOR SHEAR LOAD : (ACI 318, D.6.2.1b) (Cont'd)

$$\phi V_{cbg} = \phi \frac{A_V}{A_{Vo}} \psi_{ec,V} \psi_{ed,V} \psi_{c,V} V_b = \phi \frac{(1.5c1)(1.5c1+s+c2)}{4.5c1^2} \psi_{ec,V} \left(0.7+0.3\frac{c2}{1.5c1}\right) \psi_{c,V} \left(7\left(\frac{l}{d}\right)^{0.2} \sqrt{d}\sqrt{f'_c} c1^{1.5}\right)$$

= 18.377 k > V_{ua} [Satisfactory]

where : ϕ = 0.75 x 1 = 0.75

$\psi_{cp,N}$ term is 1.0 for no eccentricity in the connection.

$\psi_{c,V}$ term is 1.0 for location where concrete cracking is likely to occur.

l term is load bearing length of the anchor for shear, not to exceed 8d.

CHECK PRYOUT STRENGTH FOR SHEAR LOAD : (ACI 318, D.6.3.1)

$$\phi V_{cpg} = \phi k_{cp} \frac{A_N}{A_{No}} \psi_{ed,N} \psi_{c,N} N_b = \phi k_{cp} \frac{A_N}{(9h_{ef}^2)} \left(0.7+\frac{0.3c_{min}}{1.5h_{ef}}\right) \psi_{c,N} \left(24\sqrt{f'_c} h_{ef}^{1.5}\right)$$

= 62.354 k > V_{ua} [Satisfactory]

where : ϕ = 0.75 x 1 = 0.75

$\psi_{c,N}$ term is 1.0 for location where concrete cracking is likely to occur.

k_{cp} = 2.0 for h_{ef} > 2.5 in.

DETERMINE DESIGN SHEAR STRENGTH :

$$\phi V_n = \min(\phi V_s \ , \phi V_{cb} \ , \phi V_{cp})$$ = 18.377 K

REQUIRED EDGE DISTANCES AND SPACING TO PRECLUDE SPLITTING FAILURE :

Since headed cast-in-place fasteners are not like to be highly torqued, the minimum cover requirements of ACI 318 Sec. 7.7 apply.

$Cover_{Provd}$ > $Cover_{Reqd}$ [Satisfactory]

CHECK TENSION AND SHEAR INTERACTION : (ACI 318, D.7)

Since $N_{ua,1}$ > 0.2 ϕ N_n and

$V_{ua,1}$ > 0.2 ϕ V_n the full design strength is not permitted.

The interaction equation must be used

$$\frac{N_{ua,1}}{\phi N_n} + \frac{V_{ua,1}}{\phi V_n} =$$ 0.99 < 1.2 [Satisfactory]

Summary of Dimensional Properties of Fasteners

Fastener Diameter (in)		Gross Area of Fastener (in^2)	Effective Area of Threaded Fastener (in^2)	Bearing Area of Heads, Nuts, and Washers (A_b) (in^2)				
				Square	Heavy Square	Hex	Heavy Hex	Hardened Washers
0.250	1/4	0.049	0.032	0.142	0.201	0.117	0.167	0.258
0.375	3/8	0.110	0.078	0.280	0.362	0.164	0.299	0.408
0.500	1/2	0.196	0.142	0.464	0.569	0.291	0.467	0.690
0.625	5/8	0.307	0.226	0.693	0.822	0.454	0.671	1.046
0.750	3/4	0.442	0.334	0.824	1.121	0.654	0.911	1.252
0.875	7/8	0.601	0.462	1.121	1.465	0.891	1.188	1.804
1.000	1	0.785	0.606	1.465	1.855	1.163	1.501	2.356
1.125	1 1/8	0.994	0.763	1.854	2.291	1.472	1.851	2.982
1.250	1 1/4	1.227	0.969	2.288	2.773	1.817	2.237	3.682
1.375	1 3/8	1.485	1.160	2.769	3.300	2.199	2.659	4.455
1.500	1 1/2	1.767	1.410	3.295	3.873	2.617	3.118	5.301
1.750	1 3/4	2.405	1.900	-	-	-	4.144	6.541
2.000	2	3.142	2.500	-	-	-	5.316	7.903

Technical Reference:

1. Ronald Cook, "Strength Design of Anchorage to Concrete," PCA, 1999.

Moment Capacity of Concrete Section

Beam ID.	Width b in.	Depth d in.	F'c (ksi)	Fy (ksi)	A_s (in^2)	Mu (ft-k)
Pedestal	20	27	2.5	40	**0.14**	8.60

As req = 0.1064 in^2
As min = 2.7000 in^2
$A_{s\ req}$*4/3 = 0.1418 in^2
As max = 12.5282 in^2 ro= 0.000197
ro max = 0.0232
B1 = 0.85
phi = 0.9
Stress Block
a = 0.10 in

Moment Capacity of Concrete Section

Beam ID.	Width b in.	Depth d in.	F'c (ksi)	Fy (ksi)	A_s (in^2)	Mu (ft-k)

As req = in^2
As min = in^2
$A_{s\ req}$*4/3 = in^2
As max = in^2 ro=
ro max = #DIV/0!
B1 = 0.85
phi = 0.9
Stress Block
a = in

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SHEET NO. 17
CUSTOMER
JOB NO.
DATE
LOCATION
BY CHK'D

CEILING FRAME TYP.

CEILING JOIST @ 16 OC MAX

SPAN QC LAB L= 11'-6"
SPAN ENTRY / RESTROOMS 13'-9"

$\omega = 1.33(6\text{ PSF DL} + 10\text{ PSF LL})$
$= 8\text{ PLF DL} + 14\text{ PLF LL}$
$= 22\text{ PLF}$

USE 6" x 1 3/8" x 18GA @ 16" OC
STRONG BACK @ MID-SPAN.

FULL HT WALL STUDS

INTERIOR LOADING = 5 PSF
STUDS @ 16" OC ∴ $\omega = 1.33 \times 5 = 7$ PLF
MAX FULL HT STUDS = 27' ±

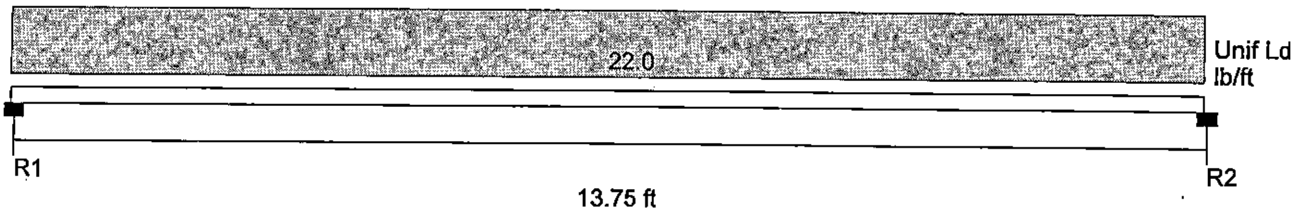
USE 6" x 1 5/8" x 18 GA @ 16" OC

STUDS @ 18' HT. → USE 6" x 1 3/8" x 18 GA @ 16" OC

2007 NASPEC

Project: Ceiling Joist
Model:

Date: 3/27/2015



Section : 600S137-43 Single C Stud (X-X Axis)
Maxo = 1061.4 Ft-Lb
Moment of Inertia, I = 2.041 in^4
Fy = 33.0 ksi
Va = 1415.7 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	519.9	0.490	519.9	Mid-Pt	676.9	0.768	0.294	L/561

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac (in)	Ma-d Ft-Lb	Mmax/ Ma-d
Center Span	0.00	165.0	985.0	0.528

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	151.3	1.00	259.1	453.4	0.0	0.30	No
R2	151.3	1.00	259.1	453.4	0.0	0.30	No

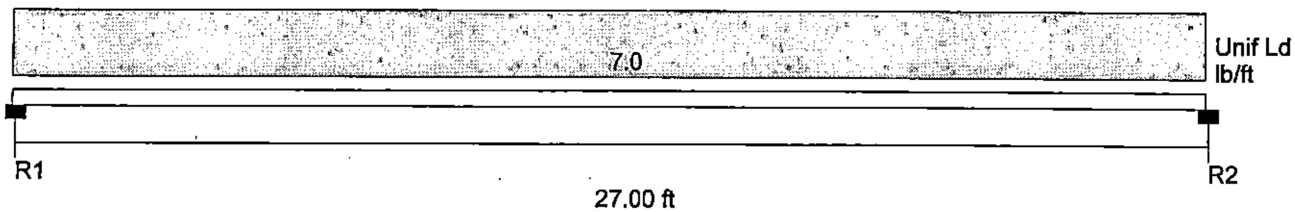
Combined Bending and Shear

Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	151.3	0.0	1.00	0.11	0.00	0.01	NA
R2	151.3	0.0	1.00	0.11	0.00	0.01	NA

19

2007 NASPEC

Project: Full Ht wall studs 27' **Date:** 3/27/2015
Model:



Section : 600S162-43 Single C Stud (X-X Axis) **Fy =** 33.0 ksi
Maxo = 1390.0 Ft-Lb **Moment of Inertia, I =** 2.316 in^4 **Va =** 1415.7 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	637.9	0.459	637.9	Full	1390.0	0.459	1.225	L/264

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac (in)	Ma-d Ft-Lb	Mmax/ Ma-d
Center Span	0.00	324.0	1205.1	0.529

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	94.5	1.00	259.1	453.4	0.0	0.19	No
R2	94.5	1.00	259.1	453.4	0.0	0.19	No

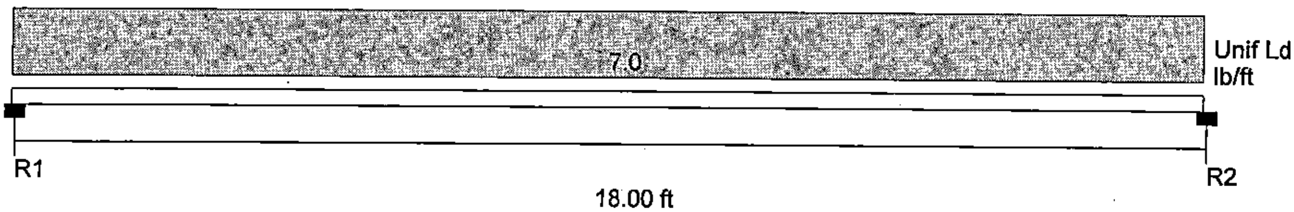
Combined Bending and Shear

Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	94.5	0.0	1.00	0.07	0.00	0.00	NA
R2	94.5	0.0	1.00	0.07	0.00	0.00	NA

20

2007 NASPEC

Project: Full Ht wall studs 18' **Date:** 3/27/2015
Model:



Section : 600S137-43 Single C Stud (X-X Axis) **Fy =** 33.0 ksi
Maxo = 1061.4 Ft-Lb **Moment of Inertia, I =** 2.041 in^4 **Va =** 1415.7 lb

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Flexural and Deflection Check

Span	Mmax Ft-Lb	Mmax/ Maxo	Mpos Ft-Lb	Bracing (in)	Ma(Brc) Ft-Lb	Mpos/ Ma(Brc)	Deflection (in)	Deflection Ratio
Center Span	283.5	0.267	283.5	Full	1061.4	0.267	0.275	L/786

Distortional Buckling Check

Span	K-phi lb-in/in	Lm Brac (in)	Ma-d Ft-Lb	Mmax/ Ma-d
Center Span	0.00	216.0	985.0	0.288

Combined Bending and Web Crippling

Reaction or Pt Load	Load P(lb)	Brng (in)	Pa (lb)	Pn (lb)	Mmax (Ft-Lb)	Intr. Value	Stiffen Req'd ?
R1	63.0	1.00	259.1	453.4	0.0	0.13	No
R2	63.0	1.00	259.1	453.4	0.0	0.13	No

Combined Bending and Shear

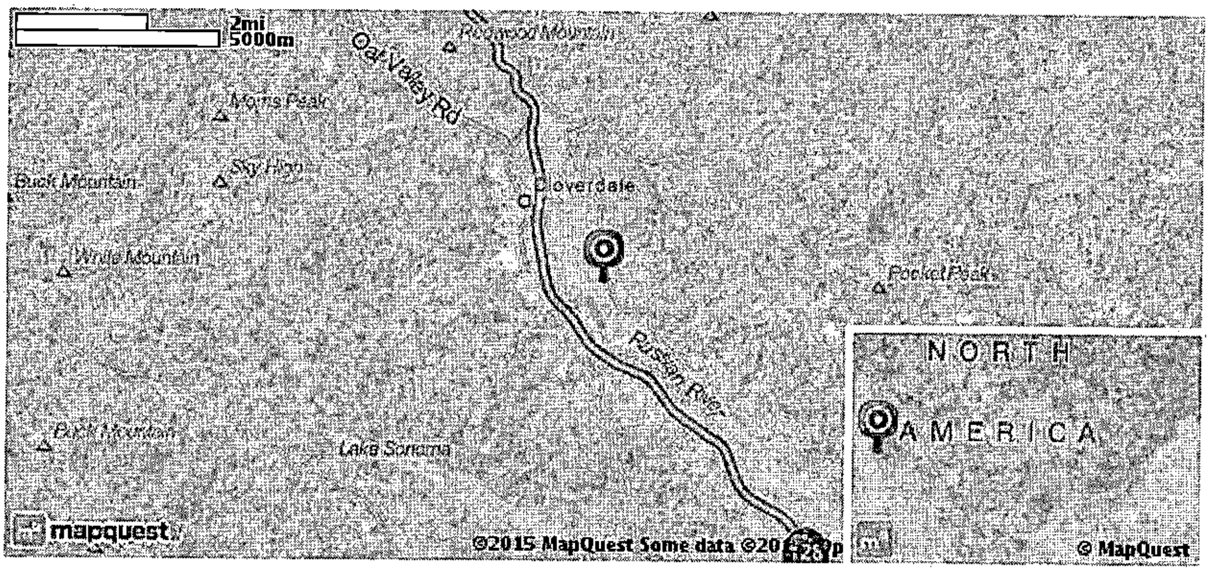
Reaction or Pt Load	Vmax (lb)	Mmax (Ft-Lb)	Va Factor	V/Va	M/Ma	Intr. Unstiffen	Intr. Stiffen
R1	63.0	0.0	1.00	0.04	0.00	0.00	NA
R2	63.0	0.0	1.00	0.04	0.00	0.00	NA

21

USGS Design Maps Summary Report

User–Specified Input

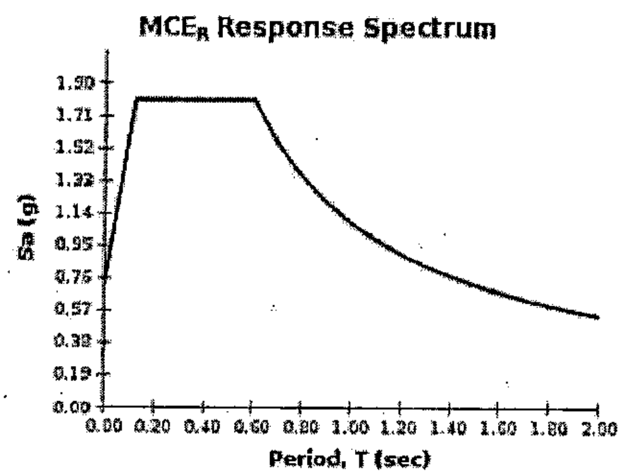
Report Title	Field Vineyards 27801 River Road Cloverdale, CA Mon March 30, 2015 18:17:38 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	38.7833°N, 122.98889°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

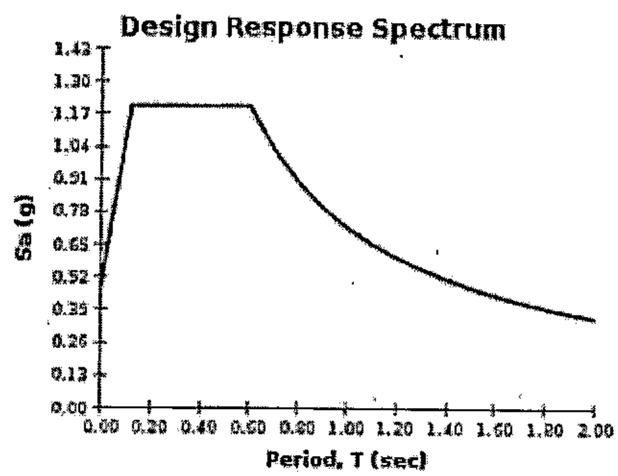


USGS–Provided Output

$S_S =$	1.806 g	$S_{MS} =$	1.806 g	$S_{DS} =$	1.204 g
$S_1 =$	0.722 g	$S_{M1} =$	1.083 g	$S_{D1} =$	0.722 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

MONUMENT SIGN

29.5, Design Wind Load on Others Structures

$F =$	$q_z G C_f A_f$		(29.5-1)
$q_z =$	$.00256 K_z K_{zt} K_d V^2$		(29.3-1)
Ht. z at the centroid of area Af =	5	ft	Exp = C
Exposure coefficient K_z =	0.85		6.5.6.6, T-6-3 for MWFR
Topography factor K_{zt} =	1.00		6.5.7.2
Directionality factor K_d =	0.85		Table 6-4
Building & Structure Risk Category =	II, standard		IBC T-1604.5
Wind Speed V =	110	MPH	Fig. 26.5-1A, MRI = 700 yrs
q_z =	22.38	psf	
Gust Effect factor G =	0.85		26.9
Force coeff C_f =	1.6		Figure 29.5-1 through 29.5-3
Design wind pressure, F/A_f =	30.44	psf	

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23

FREE STANDING WALL

	Wind Speed from Fig. 1609A =	110	MPH			
	Fig. 1609B =	115	MPH			
	Fig. 1609C =	100	MPH			
29.4 Design Wind Load on Solid Freestanding Walls and Solid Signs						
	F =	$q_h\,G\,C_f A_s$			(29.4-1)	
	q_z=	$.00256\,K_z\,K_{zt}\,K_d\,V^2$			(29.3-1)	
	Exposure	C			26.7	
	Topography factor K_{zt} =	1.00			(26.8.1)	
	Directionality factor K_d =	0.85			T-26.6-1	
	Building & Structure Risk Category =	II, standard			IBC T-1604.5	
	Wind Speed V =	110	MPH	Fig. 26.5-1A, MRI = 700 yrs		
	q_z=	26.33	K_z			
	Gust Effect factor G =	0.85			26.9	
	B/s=	2.20				
	s/h =	1.00				
	Case A & B, C_f =	1.40			Fig 29.4-1	
	Since B/s ≥ 2 Case C must also be considered					
	Total # of Segment with width, s =	2				
	Vert. location of resultant force =	3.6685 ft from grade				
	Balance, see Fig 6-20 =	1.33	ft			
	Case C, C_f for Region					
	0 to s, =	2.32				
	s to 2s, =	1.54				
	Case C, Multiples factor (if applicable)					
	Horizontal dim of return corner L_r =	0.0 ft			Free Standing Wall, Case B=	26.54PSF
	when s/h > 0.8, (1.8-s/h) =	0.80			Free Standing Wall, Case C=	28.63PSF
	for Lr/s = 0.00, =	1.00				
	%openning =	0.0%				
	Reduction factor =	1.00				

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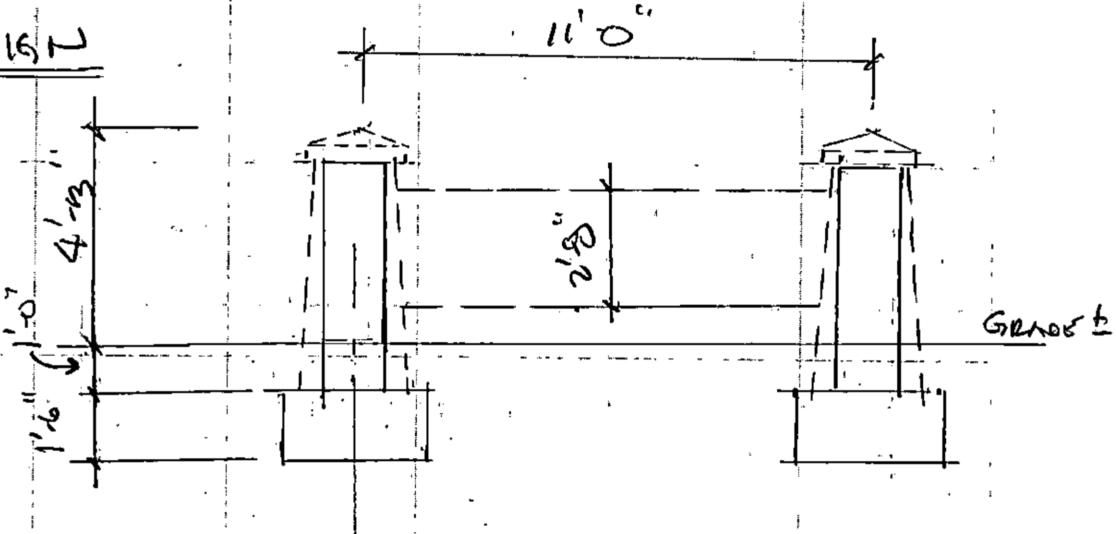
SHEET NO. 24
CUSTOMER
JOB NO.
DATE
LOCATION
BY CHK'D

MONUMENT SIGN



USE 16" CMU BLOCK
W/ STONE VENEER
SIGN IS WOOD

WIND LOAD TO ONE SIDE

FIGURE 29.4-1 $C_f = 1.56$

110MPH EXP C

$K_z = .85$ $G = .85$

$q_z = 26.33$

$q_h = 26.33(0.85) = 22.4$

$w = 22.4(.85)1.6 = 30.4$ PSF ULT

$\therefore 30.4 \times .6 = 18.3$ PSF ASD

$F_p =$ CMU $=$

$= 0.4 S_{DS} I W$

$= 0.4(1.204)1.0$

$= 0.482W$ USD

$\therefore F_p = \frac{0.482}{1.4} W = 0.344W$ ASD

$w =$ 16" CMU: 168 PSF + 40(2) VENEER

$= 248$ PSF

$F_p = 0.344(248) = 86$ PSF GOVERNS

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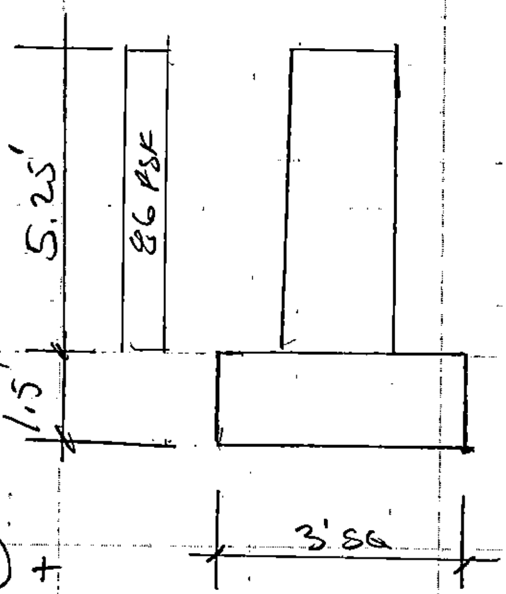
E-mail: dlfeng@dlfengineers.com

SHEET NO. 25
CUSTOMER
JOB NO.
DATE
LOCATION
BY
CHK'D

$F_p = 86$ psf

$w = 86 \times 2 = 172$ plf

$M_{OT} = 172(5.25)\left(\frac{5.25}{2} + 1.5\right)$
$= 3725^{1\text{-}\#}$



$M_{RESIT} = 0.6\left[(248 \times 2)(5.25')(1.5') + 3(3)(1.5)(.15)\left(\frac{3}{2}\right)\right]$
$= 2345^{1\text{-}\#}$

TRY 5'-0" SQ

$M_{RESIT} = 0.6\left[(248 \times 2)(5.25)(2.5) + 5(5)1.5(.15)\frac{5}{2}\right]$
$= 3915^{1\#}$

USE 5'-0" SQ x 1'-6" DEEP
w/ (6) #4 EW BOT.

M ON MASONRY
$= \frac{172(5.25)^2}{2} = 2371^{1\text{-}\#}$

16" CMU w/ (2) #4
EACH FACE
(4) TOTAL

Company Info	Project Info	
DLF Engineers	Project:	Field Vineyard
420 Hudson Street; Suite F	Location:	
Healdsburg, CA, 95448		Cloverdale, CA
Phone: (707) 838-1505	Client:	
Fax: (707) 838-1970	Job No.:	3133.1.1.14
E-mail: dave@dlfengineers.com	Footing Id:	**F1**

FOUNDATION PARAMETERS

Concrete Ultimate Compressive Strength, f'c........................	2.50 ksi
Concrete Type..	HardRock
Concrete Cover...	3.0 in.
Steel Ultimate Strength, Fy..............................	40.0 ksi
Column Size..	6.00 in. by 6.00 in.
Allowable Soil Bearing Strength..........................	1.500 ksf
Wind Load Soil Bearing Strength, (1.33 increase)..................	1.995 ksf
Seismic Load Soil Bearing Strength, (1.33 increase)...............	1.995 ksf

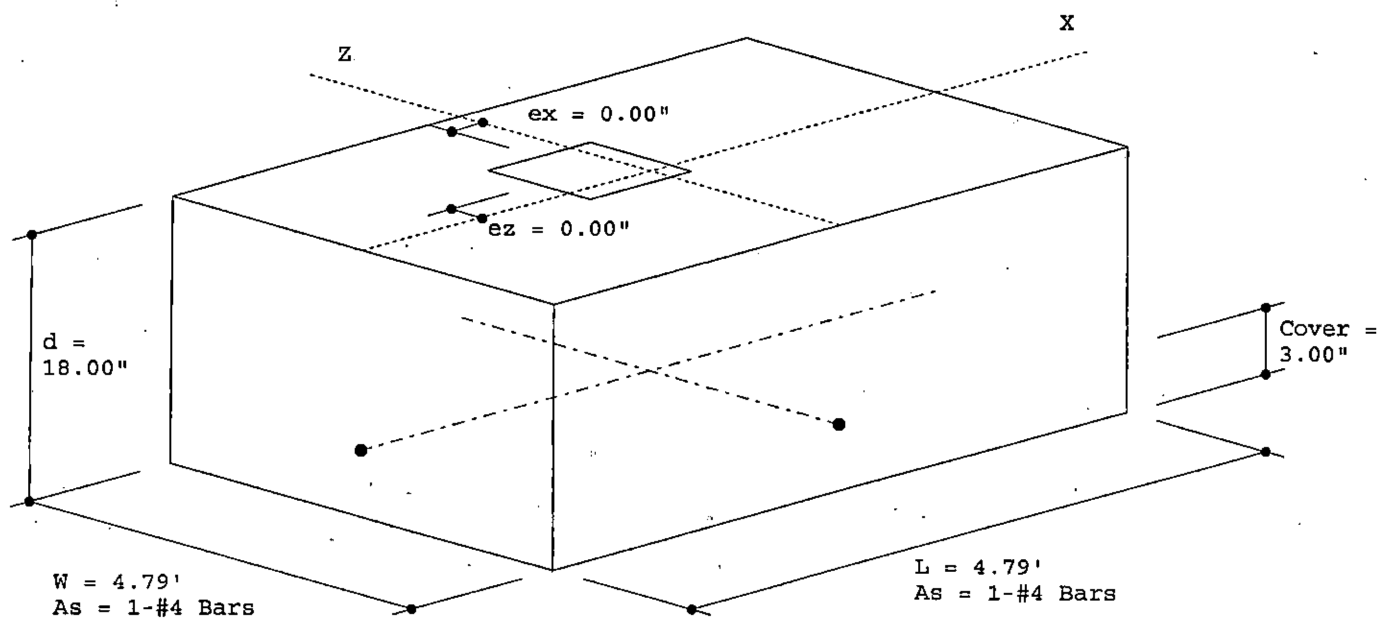
Footing Width...	**4.79 ft.**
Footing Length..	**4.79 ft.**
Footing Depth...	**18.00 in.**
Punching Shear Stress.....................................	**6.50 psi**
Beam Shear Stress...	**1.66 psi**
Reinforcing Standards per.................................	**ASTM-A615**
Longitudinal Bottom Reinforcement Required for Strength............	**.04 in² (1-#4)**
Transverse Bottom Reinforcement Required for Strength..............	**.07 in² (1-#4)**
Gravity Only Soil Bearing.................................	**.208 ksf**
Wind Load Soil Bearing....................................	**.208 ksf**
Seismic Load Soil Bearing.................................	**.416 ksf**

LOADING PARAMETERS - FACTORED LOAD CASES CONSIDERED:

1.4DL | 1.2DL + 1.6LL | 1.2DL + 1.6LL + 1.6SL
1.2DL + 1.0LL + 1.6WL | 1.2DL + 1.0LL + 1.0EQ | 0.9DL + 1.6WL
0.9DL + 1.0EQ

UNFACTORED LOADS:

Load Case	FY, (kips)	MX, (ft-kips)	MZ, (ft-kips)
Dead Load	2.70	0.00	0.00
Live Load	0.00	0.00	0.00
Wind Load	0.00	0.00	0.00
Earthquake	0.00	3.80	0.00
Other Loads	0.00	0.00	0.00



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ENGINEERING CALCULATION SHEET

P.O. Box 717, Windsor, CA 95492 • (707) 838-1505 • FAX # (707) 838-1970

3/30/2015

Masonry Bending Stress Check

Load Duration Factor	1	
Special Inspection	1	1 = SI, 2 = No SI
f'm =	1500	
M =	2400	ft-lbs.
Wall Thk =	15.625	
Edge or Ctr =	1	1 = Edge 2 = center
d =	13.375	in. (Wall Thk - 2 - bar diameter/2)
b =	16	in.
Rebar size =	4	40 = Fy ksi
spacing =	16	
As =	0.15	in.^{2} / ft
As =	0.2	in.^{2} / wall width
Ro =	0.000935	
n =	25.77778	
nRo =	0.024091	
k =	0.196732	
j =	0.934423	
2/jk =	10.87955	

fb =	109.47	psi
fs =	11521.93	psi

Allowable Stress

495.00 = Fb psi

20000.00 = Fs psi

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E-mail: dlfeng@dlfengineers.com

CUSTOMER ____________________

LOCATION ____________________

SHEET NO. 1/5
JOB NO. ____________________
DATE ____________________
BY: ____________ CHK'D: ________

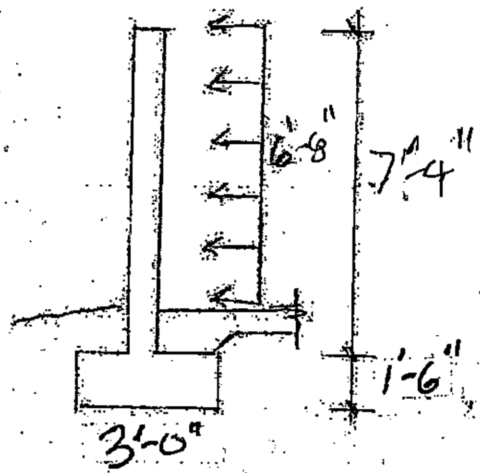
TRASH ENCLOSURE - NO ROOF

ACTUAL WIND
110 MPH EXP C
0-15' W=

WIND = 28.6 PSF × 0.6 = 17.2 PSF

SEISMIC = $\frac{0.482}{1.4}$ (84 PSF)
= 28.9 PSF

USE 29 PSF



BEARING CAPACITY = 1500 PSF   ACTUAL = 2000 PSF ∴ OK

FRICTION = 0.3

PASSIVE = 250 PSF

USE CMU 8×8×16
w/ #5 @ 16" oc V
#4 @ 24" oc H
3'-0" WIDE × 1'-6" THK.

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Title TE -Wind 18" deep ftg x 3' wide
Job # 3133.1.1.1 Dsgnr: DLF
Description....
29. PSF lateral load EQ

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Cantilevered Retaining Wall Design

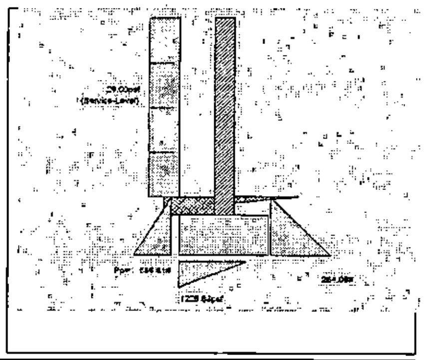
Code: CBC 2013,ACI 318-11,ACI 530-11

Criteria

Retained Height	=	0.67 ft
Wall height above soil	=	6.67 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	8.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	1,500.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
	=	
Passive Pressure	=	250.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	0.00 pcf
Footing||Soil Friction	=	0.350
Soil height to ignore for passive pressure	=	0.00 in



Surcharge Loads

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

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 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		1.00
Wind on Exposed Stem (Service Level)	=	29.0 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	=	1.92 OK
Sliding	=	4.05 OK
Total Bearing Load	=	1,377 lbs
...resultant ecc.	=	9.02 in
Soil Pressure @ Toe	=	1,227 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	1,500 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	1,718 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	5.6 psi OK
Footing Shear @ Heel	=	2.3 psi OK
Allowable	=	75.0 psi

Sliding Calcs

Lateral Sliding Force	=	264.1 lbs
less 100% Passive Force	= -	586.8 lbs
less 100% Friction Force	= -	482.1 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	0.0 lbs OK

Vertical component of active lateral soil pressure IS considered in the calculation of soil bearing pressures.

Load Factors

Building Code	CBC 2013,ACI
Dead Load	1.400
Live Load	1.700
Earth, H	1.700
Wind, W	1.300
Seismic, E	1.000

Stem Construction

			Bottom Stem OK
Design Height Above Ftg	ft	=	0.00
Wall Material Above "Ht"		=	Masonry
Design Method		=	ASD
Thickness		=	8.00
Rebar Size		=	# 5
Rebar Spacing		=	16.00
Rebar Placed at		=	Center
Design Data			
fb/FB + fa/Fa		=	0.793
Total Force @ Section			
Service Level	lbs	=	258.2
Strength Leve	lbs	=	
Moment....Actual			
Service Level	ft-#	=	1,008.6
Strength Leve	ft-#	=	
Moment.....Allowable		=	1,272.4
Shear.....Actual			
Service Level	psi	=	5.7
Strength Leve	psi	=	
Shear.....Allowable	psi	=	45.9
Wall Weight	psf	=	84.0
Rebar Depth 'd'	in	=	3.75
Masonry Data			
f'm	psi	=	1,500
Fs	psi	=	20,000
Solid Grouting		=	Yes
Modular Ratio 'n'		=	21.48
Short Term Factor		=	1.000
Equiv. Solid Thick.	in	=	7.60
Masonry Block Type		=	Normal Weight
Masonry Design Method		=	ASD
Concrete Data			
f'c	psi	=	
Fy	psi	=	

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Title TE -Wind 18" deep ftg x 3' wide
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Description....
29. PSF lateral load EQ

Date: 30 MAR 2015

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Cantilevered Retaining Wall Design

Code: CBC 2013,ACI 318-11,ACI 530-11

Footing Dimensions & Strengths

Toe Width	=	1.17 ft
Heel Width	=	1.83
Total Footing Width	=	3.00
Footing Thickness	=	18.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	1.92 ft

f'c = 2,500 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	=	1,718	0	psf
Mu' : Upward	=	971	9	ft-#
Mu' : Downward	=	367	364	ft-#
Mu: Design	=	604	355	ft-#
Actual 1-Way Shear	=	5.58	2.27	psi
Allow 1-Way Shear	=	75.00	75.00	psi
Toe Reinforcing	=	None Spec'd		
Heel Reinforcing	=	None Spec'd		
Key Reinforcing	=	None Spec'd		

Other Acceptable Sizes & Spacings
Toe: Not req'd: Mu < phi*5*lambda*sqrt(f'c)*Sm
Heel: Not req'd: Mu < phi*5*lambda*sqrt(f'c)*Sm
Key: No key defined

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....

Item		Force lbs	Distance ft	Moment ft-#
Heel Active Pressure	=	70.6	0.72	51.1
Surcharge over Heel	=			
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=	193.4	5.51	1,064.8
	=			
Total		264.1	O.T.M.	1,115.9
	=		=	

Resisting/Overturning Ratio = 1.92
Vertical Loads used for Soil Pressure = 1,377.3 lbs

.....RESISTING.....

		Force lbs	Distance ft	Moment ft-#
Soil Over Heel	=	85.7	2.42	207.3
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=		0.59	
Surcharge Over Toe	=			
Stem Weight(s)	=	616.6	1.50	926.9
Earth @ Stem Transitions	=			
Footing Weight	=	675.0	1.50	1,012.5
Key Weight	=		1.92	
Vert. Component	=			
Total =		1,377.3 lbs	R.M.=	2,146.7

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

Vertical component of active lateral soil pressure IS considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.083	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.

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Cantilevered Retaining Wall Design

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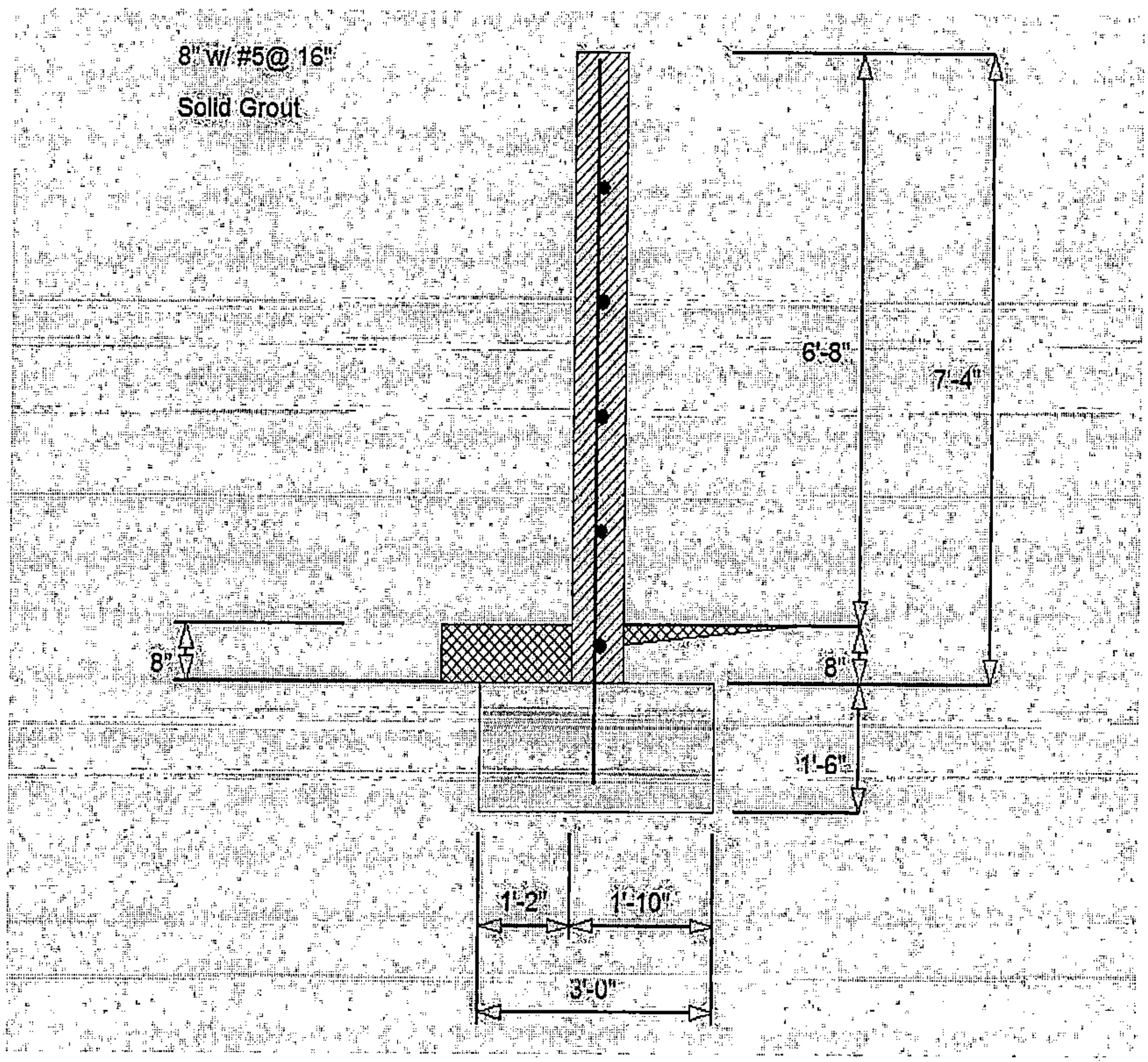

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