McPherson Engineering

Structural Analysis Report

For

Ground Mount Photovoltaic System

At

The Logan Residence
8405 Mill Station Rd
Sebastopol, CA 95472

REGISTERED PROFESSIONAL ENGINEER
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Digitally sealed by RJM on 2/22/22

Designed in Accordance with CBC 2019, ASCE 7-16, NDS 2018

Date: 2/22/22
Job #: S22-1228
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THESE ATTACHMENTS ARE PART OF THE APPROVED PLANS.
* DO NOT REMOVE THEM *
02/24/2022
PERMIT AND RESOURCE MANAGEMENT DEPARTMENT
BUILDING PLAN CHECK
PERMIT # BLD21-7982

Table Of Contents

Project Info

Location:

8405 Mill Station Rd
Sebastopol, CA 95472

Ground Mount Information:

L_{column} =	7.666667	ft.	column to column length
W_{column} =	9.00	ft.	column to column width
$L_{overhang}$ =	1.33	ft.	max. length of overhang
H_{column} =	68.00	in.	max. height of column
$\varnothing_{tilt}$ =	20.00	deg.	max. angle of frame tilt

P.V. Array Info:

Model =	JKM405M-72HL-V	
Height of Panel =	79.06	in.
Width of Panel =	39.45	in.
Area of Panel =	21.7	$ft.^2$
Number of Panels =	30	
Total Area of Array =	649.8	$ft.^2$
rows of racks per panel =	2	

Site Properties:

Lattitude =	38.411260	
Longitude =	-122.853642	
S_s =	1.500	
S_1 =	0.600	
Exposure Category =	C	
Wind Speed =	100	m.p.h.

Gravity Loads

Snow Load (if applicable):

P_g =	0	p.s.f.	snow load per AHJ	
C_t =	1.1	thermal factor		ASCE 7 Table 7.3-2
C_e =	1.0	exposure factor		ASCE 7 Table 7.3-1
C_{s_N} =	0.9	slope factor with solar (slippery)		
I =	1.0	importance factor		ASCE 7 Table 1.5-1
$SL_N = 0.7*C_e*C_t*I*P_g*C_{s\_N}$ =	0	p.s.f.		ASCE 7 Eq. 7.3-1

Proposed Loads:

Ground Mount Design Loads:

Panels =	2.79	p.s.f.	weight of panels including rack system
Misc =	1.2	p.s.f.	
DL =	4.00	p.s.f.	maximum weight of solar modules and rack system
LL =	0.0	p.s.f.	Live load
SL_N =	0.00	p.s.f.	Snow load on ground mount (if applicable)
WL=	20.3	p.s.f.	Wind load

PIPE BEAM CHECK

Beam properties

Beam Size = 2" STD Steel Pipe

Trib. =	5.83 ft	A =	1.02 in^2
L_{beam} =	7.67 ft	E =	29000 ksi
		Z =	0.71 in^3
F_y =	35 ksi	I =	0.63 in^4
W_{self} =	3.66 plf		

Load Combinations:

LC1 =	Trib. * 1.4DL + 1.4W_{self} =	37.77 plf
LC3 =	Trib. * (1.2DL + 0.5WL + 1.6SL) + 1.2W_{self} =	91.44 plf
LC4 =	Trib. * (1.2DL + WL + 0.5SL) + 1.2W_{self} =	150.50 plf
LC5 =	Trib. * (0.9DL + WL) + 0.9W_{self} =	142.41 plf
	W_u =	150.50 plf (governs)

Beam Calculations:

$$V_u = \frac{w_u * L_{beam}}{2} = 576.9 \text{ lbs (simply supported)}$$

$$M_u = \frac{w_u * L_{beam}^2}{8} = 1105.8 \text{ lb-ft (simply supported)}$$

Check for Flexure:

$M_n = F_y$*Z = 2080 lb-ft yielding

$\varnothing_b$ = 0.9

$M_c = M_n$*$\varnothing_b$ = 1872 lb-ft

Flange Local Buclking: Compact Section; does not apply

Web Local Buckling: Compact Section does not apply

M_c > M_u

$$\frac{M_u}{M_c} = \mathbf{0.59} \quad \mathbf{< 1.00}$$

2" STD Steel Pipe is OK!

SEISMIC DESIGN

Seismic Design Parameters

Risk Category =	2	Per Table 1-5-1	$R =$	2	Response Modification Factor (Table 12-2.1)
Site Class =	D	Per 11.4.2	$I_e =$	1	Seismic Importance Factor (Table 1.5-2)
$S_s =$	1.50	Short Period P.G.A.	$p =$	1.0	Redundancy Factor
$S_1 =$	0.60	1-Sec Period P.G.A.			
S.D.C. =	D	Seismic Design Category Per 11.6			
$F_a =$	1.00	Site Coefficient per Table 11.4-1	ASCE 7-16 Supplement #1 Tables used for Fa and Fv		
$F_v =$	1.50	Site Coefficient per Table 11.4-2	(note: section 11.4.8 Exception #2 calc used)		
$S_{MS} = S_{DS}\,F_a =$	1.50	Short Period M.C.E. Per Eq. 11.4-1			
$S_{M1} = S_{D1}\,F_v =$	0.90	1-Sec Period M.C.E. Per Eq. 11.4-2			
$S_{DS} = 2/3\,(S_{MS}) =$	1.00	Short Period Design Parameter Per Eq 11.4-3			
$S_{D1} = 2/3\,(S_{M1}) =$	0.60	1-Sec Period Design Parameter Per Eq 11.4-4			

Approximate Fundamental Period (Per 12.8.2.1)

$T = C_t\,(h_n)^x$		per Eq. 12.8-7
$h_n =$	6.15 ft	
$C_t =$	0.02	(per table 12.8-2)
$x =$	0.75	(per table 12.8-2)
$T =$	0.08 sec	Approximate Fundamental Period
$T_L =$	8 sec	Long-Period Transition Period per Figure 22-12

Seismic Response Coefficient (Per 12.8.1.1)

$C_S = (S_{DS}\,I_e)/R =$	0.50	Seismic Response Coefficient Per Eq. 12.8-2
$C_{s1} = 1.5*(S_{D1}\,I_e)/R\,T =$	5.76	Max. Seismic Response Coefficient Per Eq. 12.8-3 if $T \leq T_L$
$C_{s2} =$	0.01	Min. Seismic Response Coefficient Per Eq. 12.8-5
$C_{s2} = 0.044\,S_{DS}\,I_e =$	0.04	Min. Seismic Response Coefficient Per Eq. 12.8-5
$C_{s3} = 0.5\,S_1\,I_e/R =$	0.15	Min. Seismic Response Coefficient Per Eq. 12.8-6 if $S_1 \geq 0.6g$
$C_S =$	0.50	Design Seismic Coefficient
$S_f = p*C_s =$	0.50	seismic load factor

WIND DESIGN

Rigid Determination (ASCE 7-16 26.2)

$f = 1/T_{max.} =$	12.80	Hz	approximate fundamental frequency of structure
	12.80	> 1.0	Structure is rigid in accordance with ASCE 7-16

Wind Loads on Other Structures (ASCE 7-16 Section 26 & 29)

$V_{ult} =$	100 mph		$K_{zt} =$	1.00	(sec 26.8.2)
Exposure Category =	C		$K_z =$	0.85	(sec 26.10.1)
G =	0.85		$K_d =$	0.85	(sec 26.6)
$C_f =$	1.3		$K_e =$	0.99	(sec 26.9)
$q_z = 0.00256\ K_z\ K_{zt}\ K_d\ K_e\ V^2$			$q_z =$	18.34	(eq. 29.3-1) velocity pressure
$P = q_z*G*C_f =$	20.26 psf	design wind pressure			

Check Load Combinations

$Trib_{column} =$ 44.7 ft^2

Wind Loads		**Seismic Loads**	
$A_f = \sin(\varnothing_{tilt})*Trib_{column} =$	15.3 ft^2	$F_s = S_f*Trib_{column}*DL =$	89.4 lb
$F_w = A_f*p =$	309.7 lb	$V_{EQ} = F_s =$	89.4 lb
$V_w = F_w =$	309.7 lb	$M_{EQ} = V_{EQ}*H_{column} =$	506.6 lb-ft
$M_w = V_w*h_{column} =$	1755 lb-ft		
$P_{uplift} = p - 0.9DL =$	16.66 psf		

Load Combos:

LFRD:

$V_{max} = \max.$ of $(V_w, V_{EQ}) =$	310 lb	Wind Governs
$M_{max} = \max.$ of $(M_{EQ}, M_w) =$	1755 lb-ft	Wind Governs

ASD Alt. Basic Load Combos: (for soil pressure calculations)

$V_{alt} = \max.$ of $(V_{EQ}/1.4, 0.6V_w) =$	186 lb	
$M_{alt} = V_{alt}*H_{column} =$	1053 lb-ft	moment on tall pipe column (at rear)

STEEL COLUMN

Per LRFD, AISC 360, Inverted Pendulum:

Column Size =	2" STD			Column Type =	Pipe	
$L = H_{column}$ =	5.67	ft	actual height of column	F_y =	35.00	ksi
K =	2.1		buckling length coeff. (Table C-C2.2)	E_{column} =	29000	ksi
∅ =	0.9		resistance factor			
				c =	1.19	in
$P_{DL} = Trib_{column}*DL$ =	178.8	lb		A =	1.02	in^2
$P_{SL} = Trib_{column}*SL$ =	0.0	lb		I_{col} =	0.63	in^4
Load Combinations:				R =	0.79	in
				S =	0.53	in^3
Pu = 1.4*PDL	250	lb	vertical load on column	Z =	0.71	in^3

Column Calculations:

Service Level Loads:

$V_u = V_{max}$ =	310	lb	base shear on column	$M = M_u/1.4$ =	1254	lb-ft
$M_u = M_{max}$ =	1755	lb-ft	bend moment on column due to base shear	$V = V_u/1.4$ =	221	lb

$$F_e = \frac{\pi^2 E_{col}}{\left(\frac{KL}{r}\right)^2} =$$ 8782 psi — elastic critical buckling stress (Eq. E3-4)

$$F_{cr\_a} = 0.658^{\frac{F_y}{F_e}} * F_y =$$ 34999.9 psi — (Eq. E3-2)

$$F_{cr\_b} = 0.877 * F_e =$$ 7701.8 psi

F_{cr} = 7701.8 psi — buckling stress (chapter E, section E3)

$P_c = \varnothing * F_{cr} * A$ = 7070.3 lb — design compressive stress in column (chapter H, section H1, and chapter E, section E3)

M_c = min. of $(\varnothing F_y Z, \varnothing 1.6 F_y S)$ = 1871.6 lb-ft

Interaction Formula:

since P_u / P_c = 0.04 — Eq. H1-1b per AISC 13, chapter H, section 1

case 1: Eq. H1-1a
for $P_r / P_c \geq 0.2$ **case 1 is not applicable**

$$\frac{P_u}{P_c} + \frac{8}{9} * \left(\frac{M_u}{M_c}\right) =$$ N/A **< 1.00**

case 2: Eq. H1-1b
for $P_r / P_c < 0.2$ **Use Eq. H1-1b**

$$\frac{P_u}{2 * P_c} + \left(\frac{M_u}{M_c}\right) =$$ 0.96 **< 1.00** **OK!**

Column OK! Use 2" STD Steel Pipe

POLE FOOTING DESIGN

Footing Design Loads:

Pmax = PDL + PLL	178.8 lb	maximum axial loading from above
q_{allow} =	1500 psf	allowable soil bearing capacity
q =	1500 psf	soil bearing capacity
γ_p =	200 pcf	passive earth pressure
γ_c =	150 pcf	unit weight of reinforced concrete

Footing Calculations:

Footing shape =	round	
w =	12 in	diameter of footing
d =	48 in	depth of footing
d_{ig} =	0 in	ignore depth of passive pressure

$A_{req} = P_{max}/q_{allow}$ = 0.12 ft² — area of footing required for soil bearing

$A_{proivded}$ = 0.79 ft²

A_{req} = 0.12 ft² < $A_{proivded}$ OK! **VERTICAL BEARING OK!**

non constrained at ground level (see CBC/IBC 1807.3.2.1)

V_{alt} = 186 lb

$M_u = M_{alt}+V_{alt}*d_{ig}$ = 1053 lb-ft — service level base shear and moment

b =	12.0 in	diagonal dimension of footing
$h = M_u/V_{alt}$ =	5.7 ft	effectve height of lateral loading
$S_1 = 2\gamma_p(d/3)$ =	533 psf	
$A = 2.34*V/S_1*b$ =	11.65 in	allowable lateral soil bearing pressure (per CBC/IBC table 1806.2)

$$d_{req} = \frac{A}{2}\left(1 + \sqrt{4.36\left(\frac{h}{A}\right) + 1}\,\right)$$

depth required to resist lateral loading

d_{req} = 35.8 in < d = 48.0 in **DEPTH OK!**

check uplift:

$P_{uplift} = Trib_{column}*P_{uplift}$ = 745 lb　　μ_{skin} = 250.0 psf

$$W_{ftg} = \left(\frac{w}{2}\right)^2 \pi * d * \gamma_{conc} =$$ 471 lb　　$F_{skin\_\mu} = w\pi d\mu_{skin}$ = 3142 lb

$$\frac{0.9 * W_{ftg} + F_{skin\mu}}{P_{uplift}}$$ 4.79 **> 3.00** **UPLIFT IS OK!**

USE FOOTING SIZE: 12 IN. DIAM. ROUND x 48 IN. DEEP

Summary

PIPE BEAM CHECK

2" STD Steel Pipe is OK!

STEEL COLUMN

Column OK! Use 2" STD Steel Pipe

POLE FOOTING DESIGN

VERTICAL BEARING OK!
DEPTH OK!
UPLIFT IS OK!
USE FOOTING SIZE: 12 IN. DIAM. ROUND x 48 IN. DEEP