

VINCI & ASSOCIATES

Structural Engineers

STRUCTURAL CALCULATIONS

FOR

**PHOTOVOLTAIC
ARRAY**

**Buccola Residence
383 N Ashwood
Ventura, CA 93003**



CLIENT: Coastal Solar

JOB NO: 09-3457

DATE: 6/27/2019
 PROJECT: Buccola Residence
 383 N Ashwood
 Ventura, CA 93003
 CLIENT: Coastal Solar

VINCI & ASSOCIATES

Structural Engineers
 175 E WILBUR ROAD, STE 103 • THOUSAND OAKS, CA 91360
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SHEET: 1
 JOB NO: 19-3457
 ENGR: JRV

DESIGN DATA:



ROOF DESIGN LOAD:

ROOFING (Concrete Tile)	=	12	psf
SHEATHING	=	2	
ROOF RAFTERS / TRUSS	=	2	
MISCELLANEOUS	=	1	
ROOF RAFTER/TRUSS DESIGN DEAD LOAD	=	17.0	psf
CEILING & INSULATION	=	3.25	
MISCELLANEOUS	=	1.25	
DESIGN DEAD LOAD	=	21.5	psf
DESIGN LIVE LOAD	=	20	psf
DESIGN SNOW LOAD	=	0	psf

WIND DESIGN CRITERIA:

BASIC WIND SPEED	=	110	mph
WIND EXPOSURE	=	C	
NUMBER OF STORIES	=	1	

SOLAR MODULE

Solar Module Width:	=	39.4	in
Solar Module Length:	=	65.95	in
Solar Module Area	=	18.04	ft ²
Solar Module Dead Load	=	2.63	psf
Number of PV Modules	=	28	
Number of Mounts to Roof	=	48	
Maximum Spacing of Mounts	=	6	ft o.c.
Tributary Area to Mounts			
18.04 ft ² x (28 / 48)	=	10.53	ft ² (Average)
(6 ft oc x 65.95 in / 12"/ft) / 2	=	16.49	ft ² (Worst Case)
Solar Module Vertical Support Point Load			
16.49 ft ² x 2.63 psf	=	43.4	lbs

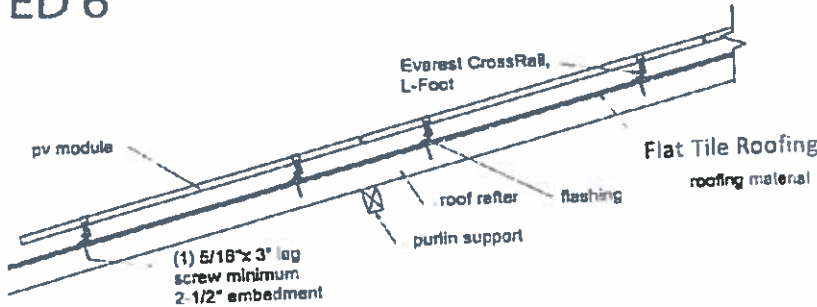
TOTAL AREA OF ADDED SOLAR PANELS

(3.28 ft x 5.50 ft) X (28 Modules) = **505.3** ft²

DESIGN CONNECTION TO ROOF TO RESIST WIND UPLIFT

-

ED 6'



REFER TO NEXT SHEET FOR
 UPLIFT FORCE CALCULATIONS

ZONE 1 UPLIFT = 24.0 psf -0.6(2.63psf)

Design Force = (Uplift x (0.6)) -0.6(2.63psf)

Net Uplift = 12.8 psf

1.71 Connections per Module (MIN)

28 Modules, 48 Connections

Max Trib Area = 16.49 ft²

UPLIFT / CONN. = 16.5 ft² x 12.8 psf
 = 211.1 lbs/conn

PER 2015 NDS (TABLE 12.2A)

P_{ALLOW} = 256 lbs/in x C_D x C_t

for (1) 5/16 in screw

WITH 2.00" PENETRATION INTO JOIST

P_{ALLOW} = 2.00 " x 410 lbs/in
 = 819.2 lbs > 211 lbs

Therefore O.K.

FACTOR OF SAFETY:

F.O.S. = 3.88 > 1.5

Therefore O.K.

ZONE 3 UPLIFT = 53.3 psf -0.6 (2.63 psf)

Design Force = (Uplift x (0.6)) -0.6 (2.63 psf)

Net Uplift = 30.4 psf

1.71 Connections per Module (MIN)

28 Modules, 48 Connections

Max Trib Area = 16.49 ft²

UPLIFT / CONN. = 16.49 ft² x 30.4 psf
 = 501.5 lbs/conn

PER 2015 NDS (TABLE 12.2A)

P_{ALLOW} = 256 lbs/in x C_D x C_t

for (1) 5/16 in screw

WITH 2.00" PENETRATION INTO JOIST

P_{ALLOW} = 2.00 " x 409.6 lbs/in
 = 819.2 lbs > 501 lbs

Therefore O.K.

FACTOR OF SAFETY:

F.O.S. = 1.63 > 1.5

Therefore O.K.

ZONE 2 UPLIFT = 33.8 psf -0.6(2.63psf)

Design Force = (Uplift x (0.6)) -0.6(2.63psf)

Net Uplift = 18.7 psf

1.71 Connections per Module (MIN)

28 Modules, 48 Connections

Max Trib Area = 16.49 ft²

UPLIFT / CONN. = 16.49 ft² x 18.7 psf
 = 307.9 lbs/conn

PER 2015 NDS (TABLE 12.2A)

P_{ALLOW} = 256 lbs/in x C_D x C_t

for (1) 5/16 in screw

WITH 2.00" PENETRATION INTO JOIST

P_{ALLOW} = 2.00 " x 410 lbs/in
 = 819.2 lbs > 308 lbs

Therefore O.K.

FACTOR OF SAFETY:

F.O.S. = 2.66 > 1.5

Therefore O.K.



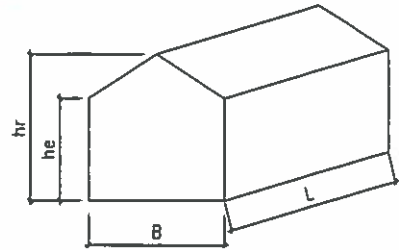
Wind Analysis for Low-rise Building, Based on ASCE 7-2010

2016 CBC

INPUT DATA

Exposure category (B, C or D, ASCE 7-10 26 7.3)
 Importance factor (ASCE 7-10 Table 1.5-2)
 Basic wind speed (ASCE 7-10 26 5.1 or 2012 IBC)
 Topographic factor (ASCE 7-10 26 8 & Table 26 8-1)
 Building height to eave
 Building height to ridge
 Building length
 Building width
 Effective area of components (or Solar Panel area)

C
 $I_w = 1.00$ for all Category
 $V = 110$ mph
 $K_{zt} = 1$ Flat
 $h_e = 10$ ft
 $h_r = 15$ ft
 $L = 65.28302$ ft
 $B = 37.09688$ ft
 $A = 505.2503$ ft²



DESIGN SUMMARY

Max horizontal force normal to building length, L, face = 15.66 kips, SD level (LRFD level), Typ
 Max horizontal force normal to building length, B, face = 8.49 kips
 Max total horizontal torsional load = 99.74 ft-kips
 Max total upward force = 46.17 kips

ANALYSIS

Velocity pressure

$q_h = 0.00256 K_h K_{zt} K_d V^2 = 24.46$ psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 28.3-1 page 298 & Eq. 30.3-1 page 316)

K_h = velocity pressure exposure coefficient evaluated at height, h. (Tab. 28.3-1, pg. 299) = 0.85

K_d = wind directionality factor (Tab. 26.6-1, for building, page 250) = 0.85

h = mean roof height = 12.50 ft

< 60 ft, [Satisfactory] (ASCE 7-10 26 2.1)

< Min (L, B), [Satisfactory] (ASCE 7-10 26 2.2)

Design pressures for MWFRS

$p = q_h [(G C_{pf}) - (G C_{pi})]$

where: p = pressure in appropriate zone (Eq. 28.4-1, page 298) $p_{min} = 16$ psf (ASCE 7-10 28.4.4)

$G C_{pf}$ = product of gust effect factor and external pressure coefficient, see table below (Fig. 28.4-1, page 300 & 301)

$G C_{pi}$ = product of gust effect factor and internal pressure coefficient (Tab. 26.11-1, Enclosed Building, page 258)

= 0.18 or -0.18

a = width of edge strips, Fig. 28.4-1, note 9, page 301, MAX[MIN(0.1B, 0.1L, 0.4h), MIN(0.04B, 0.04L), 3] = 3.71 ft

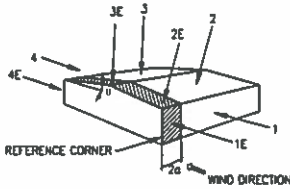
Net Pressures (psf), Basic Load Cases

Surface	Roof angle $\theta = 15.09$			Roof angle $\theta = 0.00$		
	$G C_{pf}$	Net Pressure with		$G C_{pi}$	Net Pressure with	
		(+ $G C_{pi}$)	(- $G C_{pi}$)		(+ $G C_{pi}$)	(- $G C_{pi}$)
1	0.49	7.52	16.33	-0.45	-15.41	-6.60
2	-0.69	-21.28	-12.48	-0.69	-21.28	-12.48
3	-0.44	-15.26	-6.46	-0.37	-13.45	-4.65
4	-0.38	-13.80	-4.99	-0.45	-15.41	-6.60
5				0.40	5.38	14.19
6				-0.29	-11.50	-2.69
1E	0.74	13.64	22.45	-0.48	-16.14	-7.34
2E	-1.07	-30.58	-21.77	-1.07	-30.58	-21.77
3E	-0.64	-20.00	-11.19	-0.53	-17.37	-8.56
4E	-0.57	-18.38	-9.57	-0.48	-16.14	-7.34
5E				0.61	10.52	19.32
6E				-0.43	-14.92	-6.12

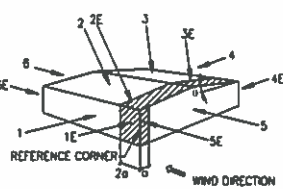
Net Pressures (psf), Torsional Load Cases

Surface	Roof angle $\theta = 15.09$		
	$G C_{pf}$	Net Pressure with	
		(+ $G C_{pi}$)	(- $G C_{pi}$)
1T	0.49	1.88	4.08
2T	-0.69	-5.32	-3.12
3T	-0.44	-3.82	-1.61
4T	-0.38	-3.45	-1.25

Surface	Roof angle $\theta = 0.00$		
	$G C_{pi}$	Net Pressure with	
		(+ $G C_{pi}$)	(- $G C_{pi}$)
5T	0.40	1.35	3.55
6T	-0.29	-2.87	-0.67

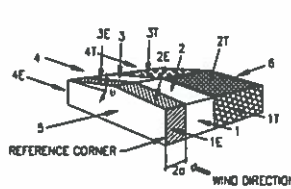


Load Case A (Transverse)

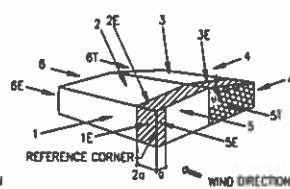


Load Case B (Longitudinal)

Basic Load Cases



Load Case A (Transverse)



Load Case B (Longitudinal)

Torsional Load Cases

Basic Load Case A (Transverse Direction)

Surface	Area (ft ²)	Pressure (k) with	
		(+GC _{p1})	(-GC _{p1})
1	578	4.35	9.44
2	1111	-23.65	-13.86
3	1111	-16.96	-7.17
4	578	-7.98	-2.89
1E	74	1.01	1.67
2E	143	-4.36	-3.10
3E	143	-2.85	-1.60
4E	74	-1.36	-0.71
Σ	Horiz.	12.57	12.57
	Vert.	-46.17	-24.85
Min. wind	Horiz.	15.66	15.66
28.4.4	Vert.	-38.74	-38.74

Basic Load Case B (Longitudinal Direction)

Surface	Area (ft ²)	Pressure (k) with	
		(+GC _{p1})	(-GC _{p1})
2	1111	-23.65	-13.86
3	1111	-14.95	-5.16
5	386	2.08	5.47
6	386	-4.44	-1.04
2E	143	-4.36	-3.10
3E	143	-2.48	-1.22
5E	78	0.82	1.51
6E	78	-1.16	-0.48
Σ	Horiz.	8.49	8.49
	Vert.	-37.60	-17.38
Min. wind	Horiz.	7.42	7.42
28.4.4	Vert.	-38.74	-38.74

Torsional Load Case A (Transverse Direction)

Surface	Area (ft ²)	Pressure (k) with		Torsion (ft-k)	
		(+GC _{p1})	(-GC _{p1})	(+GC _{p1})	(-GC _{p1})
1	252	1.90	4.12	27	60
2	484	-10.31	-6.04	-39	-23
3	484	-7.39	-3.13	28	12
4	252	-3.48	-1.26	50	18
1E	74	1.01	1.67	29	48
2E	143	-4.36	-3.10	-33	-23
3E	143	-2.85	-1.60	21	12
4E	74	-1.36	-0.71	39	21
1T	326	0.61	1.33	-10	-22
2T	627	-3.34	-1.96	14	8
3T	627	-2.39	-1.01	-10	-4
4T	326	-1.13	-0.41	-18	-7
Total Horiz. Torsional Load, M _T				100	100

Torsional Load Case B (Longitudinal Direction)

Surface	Area (ft ²)	Pressure (k) with		Torsion (ft-k)	
		(+GC _{p1})	(-GC _{p1})	(+GC _{p1})	(-GC _{p1})
2	1111	-23.65	-13.86	-11	-7
3	1111	-14.95	-5.16	7	2
5	154	0.83	2.18	6	15
6	154	-1.77	-0.41	12	3
2E	143	-4.36	-3.10	35	25
3E	143	-2.48	-1.22	-20	-10
5E	78	0.82	1.51	14	25
6E	78	-1.16	-0.48	19	8
5T	232	0.31	0.82	-3	-7
6T	232	-0.67	-0.16	-6	-1
Total Horiz. Torsional Load, M _T				53.7	53.7

Design pressures for components and cladding

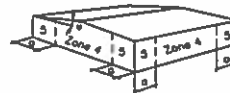
$p = q_h [(G C_p) - (G C_{pi})]$

where: p = pressure on component. (Eq. 30.4-1, pg 318)

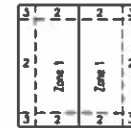
p_{min} = 16.00 psf (ASCE 7-10 30.2.2)

G C_p = external pressure coefficient.

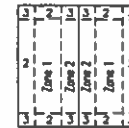
see table below (ASCE 7-10 30.4.2)



Walls



Roof 0-7*



Roof 0-7*

	Effective Area (ft ²)	Zone 1		Zone 2		Zone 3		Zone 4		Zone 5	
		GC _p	-GC _p	GC _p	-GC _p	GC _p	-GC _p	GC _p	-GC _p	GC _p	-GC _p
Comp.	505.2503	0.30	-0.80	0.30	-1.20	0.30	-2.00	0.70	-0.80	0.70	-0.80

Comp. & Cladding Pressure (psf)	Zone 1		Zone 2		Zone 3		Zone 4		Zone 5	
	Positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative	Positive	Negative
	16.00	-23.97	16.00	-33.76	16.00	-53.32	21.53	-23.97	21.53	-23.97

Note: If the effective area is roof Solar Panel area, the only zone 1, 2, or 3 apply.

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 PROJECT: Buccola Residence
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SHEET: 5
 JOB NO.: 19-3457
 ENGR: JRV

LATERAL FORCES: (Base Shear Calculations - 2016 CBC)

SEISMIC:

Site Class: **D** (Use D if unknown)

$S_1 = 0.937$ (USGS Software) $S_S = 2.46$ (USGS Software)

$F_v = 1.5$ (Table 1613.3.3(2)) $F_a = 1.0$ (Table 1613.3.3 (1))

$I = 1.0$ (Table 11.5-1 ASCE 7-10)

$R = 6.5$ (Table 12.2-1 ASCE 7-10)

$S_{MS} = F_a * S_S = 2.463$ (Eq. 16-37)

$S_{DS} = \frac{2}{3} * S_{MS} = 1.64$ (Eq. 16-39)

$S_{M1} = F_v * S_1 = 1.406$ (Eq. 16-38)

$S_{D1} = \frac{2}{3} * S_{M1} = 0.94$ (Eq. 16-40)

$T_L = 8$ s (Figure 22-12 ASCE 7-10)

$T = C_t * (h_n)^x = 0.112$ s (Eq. 12.8-7) $C_t = 0.02$ (Table 12.8-2 ASCE 7-10)

$h_n = 10.0$ ft $x = 0.75$ (Table 12.8-2 ASCE 7-10)

****GOVERNS****

$V_{MIN} = (S_{DS}) / (R / I) = 0.25$ W	(Eq. 12.8-2 ASCE 7-10)
---	------------------------

NEED NOT EXCEED:

IF $T \leq T_L$ USE:

$V = (S_{D1}) / T * (R / I) = 1.28$ W (Eq. 12.8-3 ASCE 7-10)

IF $T > T_L$ USE:

$V = (S_{D1}) * (T_L) / T^2 * (R / I) = 91.17$ W (Eq. 12.8-4 ASCE 7-10)

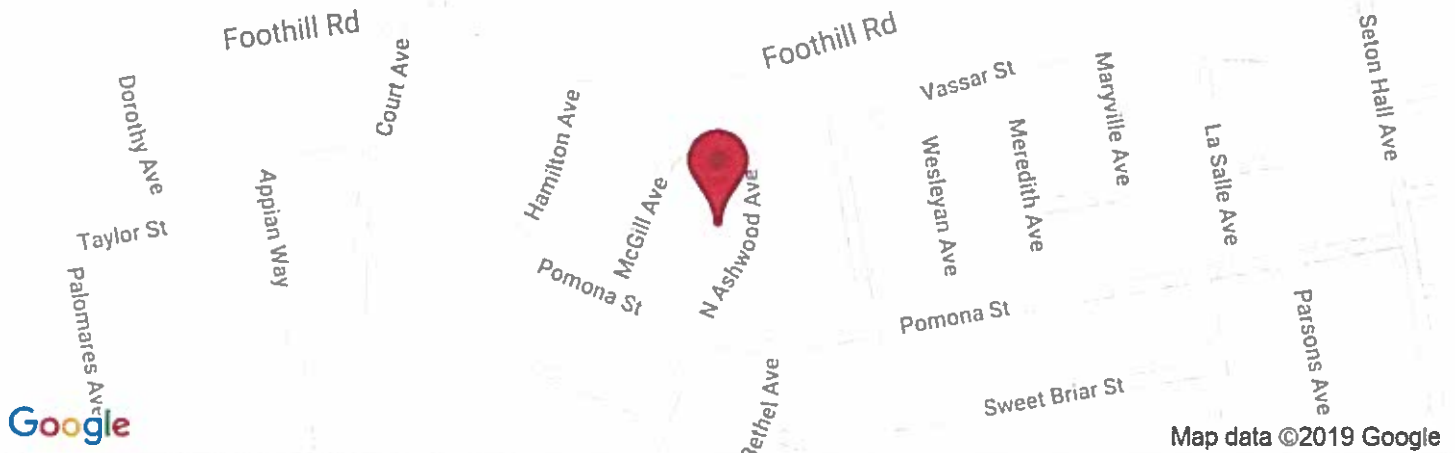
SHALL NOT BE LESS THAN:

$V = 0.01$ W (Eq. 12.8-5 ASCE 7-10)



383 N Ashwood Ave, Ventura, CA 93003, USA

Latitude, Longitude: 34.2812053, -119.23922920000001



Date	6/27/2019, 12:31:12 PM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	2.463	MCE_R ground motion. (for 0.2 second period)
S_1	0.937	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.463	Site-modified spectral acceleration value
S_{M1}	1.405	Site-modified spectral acceleration value
S_{DS}	1.642	Numeric seismic design value at 0.2 second SA
S_{D1}	0.937	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	E	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.943	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.943	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
S_{sRT}	2.463	Probabilistic risk-targeted ground motion. (0.2 second)
S_{sUH}	2.628	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
S_{sD}	2.836	Factored deterministic acceleration value. (0.2 second)
S_{1RT}	0.937	Probabilistic risk-targeted ground motion. (1.0 second)
S_{1UH}	1.012	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S_{1D}	1.108	Factored deterministic acceleration value. (1.0 second)
$PGAd$	1.089	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.937	Mapped value of the risk coefficient at short periods
C_{R1}	0.925	Mapped value of the risk coefficient at a period of 1 s

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SHEET: 7
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LATERAL FORCES 2016 CBC:

(BASE SHEAR - Conservative)

SEISMIC BASE SHEAR:

$V = 0.25 W$ (SEE PREVIOUS)



F_1	ROOF =	22	PSF
	WALLS =	5	
		<hr/>	
		26.8	
	x	0.253	
		<hr/>	
		6.76	PSF

WIND LOADS:

$p_s = \lambda k_{zt} I_w p_{s30}$ (Eq. 6-1)

$\lambda =$	1.21	$p_{s30A} =$	19.20	psf	p_{sA}	23.27	psf
$I_w =$	1.00	$p_{s30B} =$	-10.00	psf	p_{sB}	-12.12	psf
$k_{zt} =$	1.00	$p_{s30C} =$	12.70	psf	p_{sC}	15.39	psf
		$p_{s30D} =$	-5.90	psf	p_{sD}	-7.15	psf

Governs



$F1_{ULT}:$	=	(5 ft + 10 ft / 2) x (23.2704 psf + 12.12 psf)	=	354 lb/ft
$F1_{ASD}:$	=	353.90 lb/ft x 0.6	=	212 lb/ft

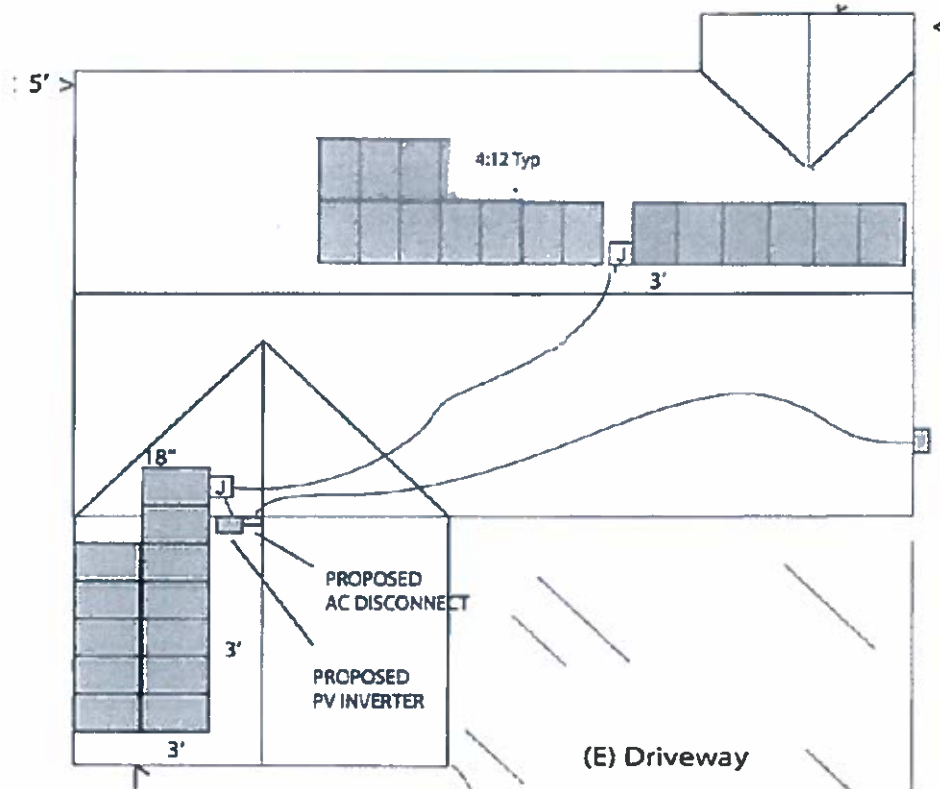
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SHEET: 8
 JOB NO.: 19-3457
 ENGR.: JRV

CHECK IMPACT OF NEW SOLAR PANELS ON SEISMIC LOAD



TOTAL ROOF AREA:

$$\begin{aligned}
 & (65.3 \text{ ft} \times 37.1 \text{ ft}) + (28.9 \text{ ft} \times 21.3 \text{ ft}) + \\
 & (16.5 \text{ ft} \times 4.1 \text{ ft}) + (0.0 \text{ ft} \times 0.0 \text{ ft}) = 3103.4799 \text{ ft}^2
 \end{aligned}$$

EXISTING ESTIMATED SEISMIC LOAD (BASE SHEAR)

$$3103.48 \text{ ft}^2 \times 6.76 \text{ psf} = \underline{\underline{20992.9 \text{ lbs}}}$$

AREA OF ADDED SOLAR PANELS

$$(5.50 \text{ ft} \times 3.28 \text{ ft}) \times (28 \text{ PANELS}) = 505.3 \text{ ft}^2$$

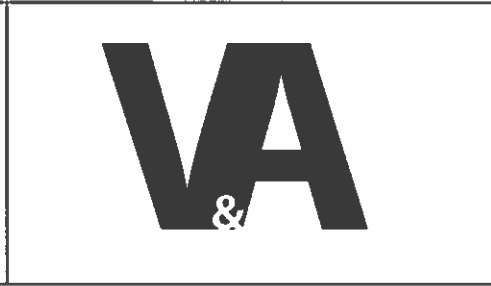
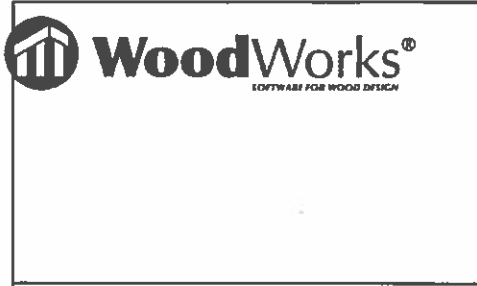
WEIGHT OF PANELS: 2.63 psf

ADDED SEISMIC LOAD:

$$2.63 \text{ psf} \times 505.3 \text{ ft}^2 \times 0.25 = 335.7 \text{ lbs}$$

$$\% \text{ INCREASE} = \frac{335.7}{20992.94509} = 1.6\% < 10\%$$

Therefore O.K.



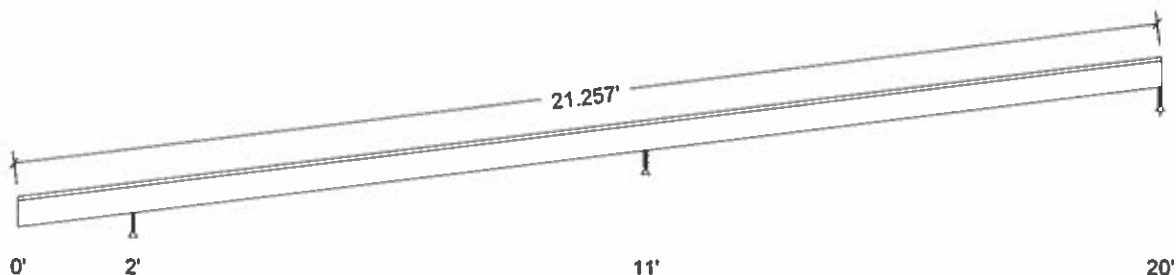
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June 27, 2019 12:51

Design Check Calculation Sheet
WoodWorks Sizer 11.1

Loads:

Load	Type	Distribution	Pat-tern	Location [ft]		Magnitude		Unit
				Start	End	Start	End	
Roof-DL	Dead	Full Area	No			10.00 (24.0")		psf
Roof-LLR	Roof constr.	Full Area	Yes			20.00 (24.0")		psf
PV1	Dead	Point	No	7.00		43		lbs
PV2	Dead	Point	No	11.00		43		lbs
PV3	Dead	Point	No	12.50		43		lbs
PV4	Dead	Point	No	16.50		43		lbs
Self-weight	Dead	Full UDL	No			2.0		plf

Maximum Reactions (lbs) and Bearing Lengths (in) :



Unfactored:						
Dead		143		388		107
Roof Live		250		441		163
Factored:						
Total		392		829		269
Bearing:						
Length		0.50*		0.71		0.50*
Min req'd		0.50*		0.71**		0.50*

*Minimum bearing length setting used: 1/2" for end supports and 1/2" for interior supports

**Minimum bearing length governed by the required width of the supporting member.

Maximum reaction on at least one support is from a different load combination than the critical one for bearing design, shown here, due to Kd factor. See Analysis results for reaction from critical load combination.

Lumber-soft, D.Fir-L, No.2, 2x6 (1-1/2"x5-1/2")

Supports: All - Timber-soft Beam, D.Fir-L No.2

Roof joist spaced at 24.0" c/c; Total length: 21.26'; Clear span: 2.086', 9.434', 9.434'; volume = 1.2 cu.ft.; Pitch: 4/12

Lateral support: top= full, bottom= at all supports; Repetitive factor: applied where permitted (refer to online help);

WARNING: Member length exceeds typical stock length of 18.0 [ft]

Analysis vs. Allowable Stress and Deflection using NDS 2015 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 67	Fv' = 225	psi	fv/Fv' = 0.30
Bending(+)	fb = 843	Fb' = 1682	psi	fb/Fb' = 0.50
Bending(-)	fb = 1146	Fb' = 1328	psi	fb/Fb' = 0.86
Deflection:				
Interior Live	0.14 = L/789	0.47 = L/240	in	0.30
Total	0.22 = L/515	0.63 = L/180	in	0.35
Cantil. Live	-0.11 = L/240	0.21 = L/120	in	0.50
Total	-0.14 = L/184	0.28 = L/90	in	0.49