[redacted]

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STRUCTURAL CALCULATIONS

FOR

[redacted]

AT

Pleasanton, CA

PREPARED FOR:

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

10/19/2018 [redacted] 7-185

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GOVERNING CODES & DESIGN CRITERIA

California Building Code, 2016 Edition
*ASCE/SEI 7-10 Minimum Design Loads for Buildings and Other Structures

SEISMIC COEFFICIENTS

Site Criteria and Seismic Design Category

Importance Factor -->	I =	1.00 (*Table 11.5-1)
Occupancy Category = II (*Table 1-1)		
Site Class: = D (Table 1613.5.2)		
Mapped Short Term Response Spectral Acceleration ---------------->	S_S =	1.81 (*Fig. 22-3)
Mapped 1s period Spectral Response Acceleration ---------------->	S_1 =	0.67 (*Fig. 22-4)
Site Coefficients -->	F_a =	1.00 (Table 1613.5.3(1))
	F_v =	1.50 (Table 1613.5.3(2))
Modified MCE Spectral Response Acceleration at 0.2s ----------->	$S_{MS} = F_a\ S_S$ =	1.81 (Eqn 16-37)
Modified MCE Spectral Response Acceleration at 1s ------------->	$S_{M1} = F_v\ S_1$ =	1.01 (Eqn 16-38)
Design Spectral Acceleration at 0.2s --------------------------->	$S_{DS} = (2/3)\ S_{MS}$ =	1.21 (Eqn 16-39)
Design Spectral Acceleration at 1s ----------------------------->	$S_{D1} = (2/3)\ S_{M1}$ =	0.67 (Eqn 16-40)

Seismic Design Category: D (*Table 11.6-1 and 11.6-2)

BY [redacted] DATE 10/18/18 [redacted] SHEET NO.____OF ______

CHKD. BY ____ DATE ______ JOB NO. **17-185**

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JWK version 1.1

Seismic Base Shear (Equivalent Lateral Force Procedure Section 12.8*)

Structural System Light-Framed Walls With Wood Shear Panels

Response Modification Factor --> R = 6.50 (*Table 12.2-1)

Fundamental Period Worksheet (*Section 12.8.2)

Using Approximate Method:

Structural Type ---------------------------------> All other structural Systems

	Ct =	0.02 (*Table 12.8-2)
	x =	0.75 (*Table 12.8-2)
Height of structure --------------------------------->	h_n =	30 ft
Number of Stories --------------------------------->	N =	2
Approximate Fundamental Period	$Ta = C_t h_n^x$ =	0.256 (*Eqn 12.8-7)
Approximate Fundamental Period	Ta = 0.1N =	n.a. (*Eqn 12.8-8)
Using properly substantiated analysis:	T =	not used
Fundamental Period of the Structure:	T =	0.256 sec

Long Period Transition Period		T_L =	8 (*Figure 22-16)
Seismic Response Coefficient, Cs =		S_{DS} / (R / I) =	0.19 (*Eqn 12.8-2)
	Upper Limit (for T <= TL):	S_{D1} / (T (R / I)) =	0.40 (*Eqn 12.8-3)
	Upper Limit (for T > TL):	$S_{D1}T_L$ / (T^2 (R / I)) =	na (*Eqn 12.8-4)
	Lower Limit:		0.01 (*Eqn 12.8-5)
	Lower Limit (S_1 >= 0.6):	0.5 x S_1 / (R / I) =	0.05 (*Eqn 12.8-6)
	T<0.5s; N<=5 stories; using S_s = 1.5max	=	na (*Section 12.8.1.3)
	☐ Regular Structure		(*Section 12.3.2)
Seismic Base Shear (Strength):		$V = C_s W$ =	**0.19** W (*Eqn 12.8-1)
Redundancy Factor:	For Diaphragm Design --------------------------->	ρ =	1.00 (*Section 12.3.4.1)
	For SDC D through F --------------------------->	ρ =	1.00 (*Section 12.4.3.2)

Earthquake Load:

Strength Design:	$E_h = \rho V$ =	0.185 W
	$E_v = .2 {}^* S_{DS}$ D =	0.241 D
Allowable Stress Design:	E_h/1.4 =	0.132 W
	E_v/1.4 =	0.172 D

[redacted] # Design Maps Detailed Report

ASCE 7-10 Standard (37.66676°N, 121.86446°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 1.808$ g

From Figure 22-2 [2] $S_1 = 0.673$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1		See Section 20.3.1	

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

[redacted]

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.808 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	$\boxed{1.5}$
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.673 g, F_v = 1.500

[redacted]

1.005

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 1.808 = 1.808$ g

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Equation (11.4–2): $S_{M1} = F_vS_1 = 1.500 \times 0.673 = 1.009$ g

---

Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 1.808 = 1.205$ g

---

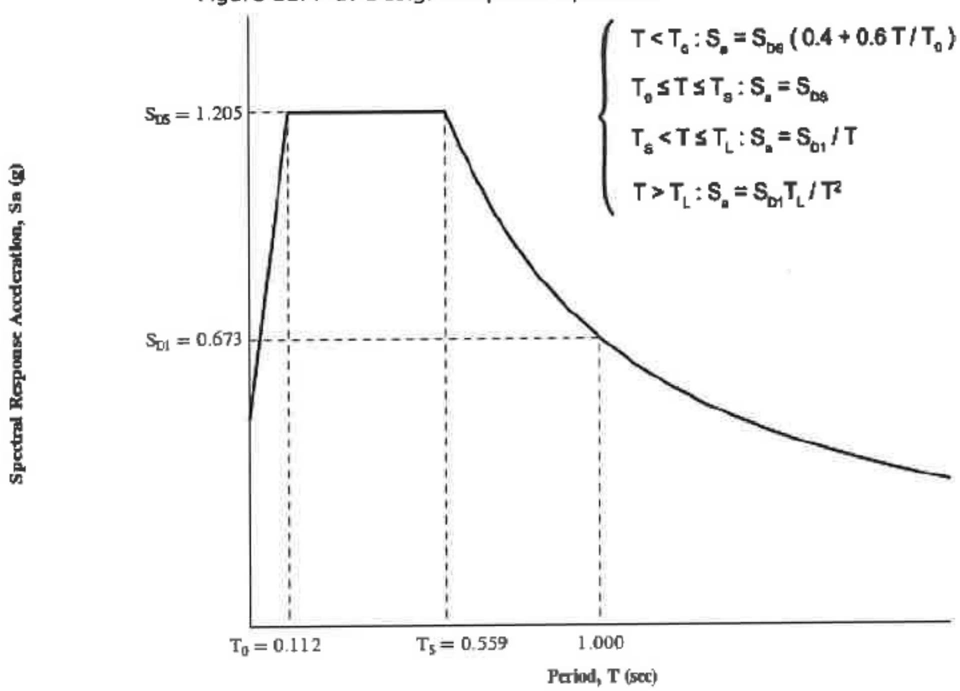
Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 1.009 = 0.673$ g

---

Section 11.4.5 — Design Response Spectrum

From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4–1: Design Response Spectrum

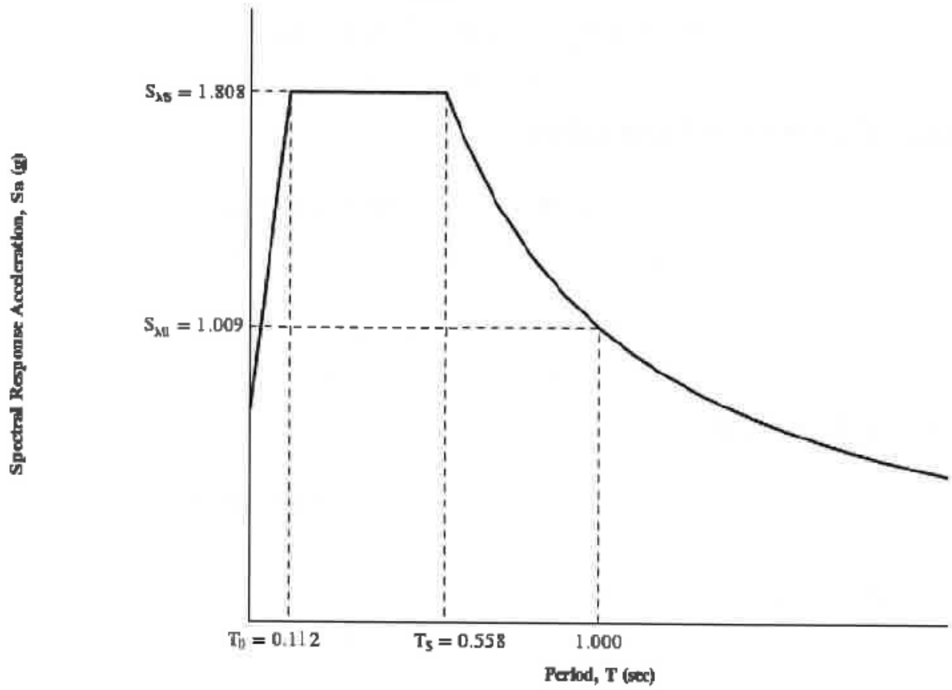


[redacted]

1.006

Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.691

Equation (11.8-1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.691 = 0.691$ g

Table 11.8-1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.691 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 1.025

From Figure 22-18 [6] C_{R1} = 1.005

[redacted]

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.205 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.673 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

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Date	: 8/7/2018	Project No.	: JobNo
Company Name	: True	Designed By	: Engineer
Address	: Address	Description	: Description
City	: City	Customer Name	: Customer
State	: State	Proj Location	: Location

File Location: C:\Users\toan\AppData\Roaming\MecaWind\Default.wnd

Input Parameters: Directional Procedure All Heights Building (Ch 27 Part 1)

Basic Wind Speed(V)	= 110.00 mph		
Structural Category	= II	Exposure Category	= B
Natural Frequency	= N/A	Flexible Structure	= No
Importance Factor	= 1.00	Kd Directional Factor	= 0.85
Alpha	= 7.00	Zg	= 1200.00 ft
At	= 0.14	Bt	= 0.84
Am	= 0.25	Bm	= 0.45
Cc	= 0.30	l	= 320.00 ft
Epsilon	= 0.33	Zmin	= 30.00 ft
Pitch of Roof	= 3.5 : 12	Slope of Roof(Theta)	= 16.26 Deg
h: Mean Roof Ht	= 28.60 ft	Type of Roof	= GABLED
RHt: Ridge Ht	= 37.21 ft	Eht: Eave Height	= 20.00 ft
OH: Roof Overhang at Eave	= 3.50 ft	Overhead Type	= Overhang
Bldg Length Along Ridge	= 159.00 ft	Bldg Width Across Ridge	= 118.00 ft

Gust Factor Calculations

Gust Factor Category I Rigid Structures - Simplified Method
Gust1: For Rigid Structures (Nat. Freq.>1 Hz) use 0.85 = 0.85

Gust Factor Category II Rigid Structures - Complete Analysis

Zm:	0.6*Ht	= 30.00 ft
lzm:	Cc*(33/Zm)^0.167	= 0.30
Lzm:	l*(Zm/33)^Epsilon	= 309.99 ft
Q:	(1/(1+0.63*((B+Ht)/Lzm)^0.63))^0.5	= 0.85
Gust2:	0.925*((1+1.7*lzm*3.4*Q)/(1+1.7*3.4*lzm))	= 0.83

Gust Factor Summary
Not a Flexible Structure use the Lessor of Gust1 or Gust2 = 0.83

Table 26.11-1 Internal Pressure Coefficients for Buildings, GCpi

GCPi : Internal Pressure Coefficient = +/-0.18

Wind Pressurs Main Wind Force Resisting System (MWFRS) - Ref Figure 27.4-1

Kh:	2.01*(Ht/Zg)^(2/Alpha)	= 0.69
Kht:	Topographic Factor (Figure 6-4)	= 1.00
Qh:	.00256*(V)^2*I*Kh*Kht*Kd	= 10.92 psf
Cpww:	Windward Wall Cp(Ref Fig 6-6)	= 0.80
Roof Area		= 20703.10 ft^2
Reduction Factor based on Roof Area		= 0.80

MWFRS-Wall Pressures for Wind Normal to 159 ft Wall (Normal to Ridge)

All pressures shown are based upon ASD Design, with a Load Factor of .6

Wall	Cp	Pressure +GCpi (psf)	Pressure -GCpi (psf)
Leeward Wall	-0.50	-6.52	-2.59
Side Walls	-0.70	-8.35	-4.42

Wall	Elev ft	Kz	Kzt	Cp	qz psf	Press +GCpi	Press -GCpi	Total +/-GCpi
Windward	20.00	0.62	1.00	0.80	9.86	4.62	8.55	11.14
Windward	10.00	0.57	1.00	0.80	9.08	4.10	8.03	10.62

Roof Location	Cp	Pressure +GCpi (psf)	Pressure -GCpi (psf)
Windward - Min Cp	-0.45	-6.07	-2.14
Windward - Max Cp	0.05	-1.51	2.42
Leeward Norm to Ridge	-0.53	-6.80	-2.87

Overhang Top (Windward)	-0.45	-4.10	-4.10
Overhang Top (Leeward)	-0.53	-4.83	-4.83
Overhang Bot (Windward only)	0.80	6.58	6.58

Normal to Ridge - Base Reactions - Walls+Roof +GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Leeward Wall	-6.52	3180	.00	20.74	.00	207.4	.0	.0
Side Wall	-8.35	2360	-19.70	.00	.00	.0	197.0	.0
Side Wall	-8.35	2360	19.70	.00	.00	.0	-197.0	.0
Windward Wall	4.62	1590	.00	7.34	.00	110.1	.0	.0
Windward Wall	4.10	1590	.00	6.52	.00	32.6	.0	.0
Roof Windward	-6.07	9772	.00	-16.60	56.92	1204.2	.0	.0
Roof Leeward	-6.80	9772	.00	18.60	63.76	-1348.9	.0	.0
OH Top Windward	-4.10	580	.00	-0.67	2.28	125.7	.0	.0
OH Top Leeward	-4.83	580	.00	0.78	2.69	-148.0	.0	.0
OH Bot Windward	6.58	580	.00	-1.07	3.66	201.8	.0	.0
Side Wall	-8.35	1015	-8.47	.00	.00	.0	218.1	.0
Side Wall	-8.35	1015	8.47	.00	.00	.0	-218.1	.0
Total	.00	34393	.00	35.65	129.31	384.8	.0	.0

Normal to Ridge - Base Reactions - Walls Only +GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Leeward Wall	-6.52	3180	.00	20.74	.00	207.4	.0	.0
Side Wall	-8.35	2360	-19.70	.00	.00	.0	197.0	.0
Side Wall	-8.35	2360	19.70	.00	.00	.0	-197.0	.0
Windward Wall	4.62	1590	.00	7.34	.00	110.1	.0	.0
Windward Wall	4.10	1590	.00	6.52	.00	32.6	.0	.0
Side Wall	-8.35	1015	-8.47	.00	.00	.0	218.1	.0
Side Wall	-8.35	1015	8.47	.00	.00	.0	-218.1	.0
Total	.00	13111	.00	34.60	.00	350.2	.0	.0

Normal to Ridge - Base Reactions - Walls+Roof -GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Leeward Wall	-2.59	3180	.00	8.24	.00	82.4	.0	.0
Side Wall	-4.42	2360	-10.42	.00	.00	.0	104.2	.0
Side Wall	-4.42	2360	10.42	.00	.00	.0	-104.2	.0
Windward Wall	8.55	1590	.00	13.59	.00	203.9	.0	.0
Windward Wall	8.03	1590	.00	12.77	.00	63.8	.0	.0
Roof Windward	2.42	9772	.00	6.62	-22.71	-480.5	.0	.0
Roof Leeward	-2.87	9772	.00	7.84	26.88	-568.8	.0	.0
OH Top Windward	-4.10	580	.00	-0.67	2.28	125.7	.0	.0
OH Top Leeward	-4.83	580	.00	0.78	2.69	-148.0	.0	.0
OH Bot Windward	6.58	580	.00	-1.07	3.66	201.8	.0	.0
Side Wall	-4.42	1015	-4.48	.00	.00	.0	115.4	.0
Side Wall	-4.42	1015	4.48	.00	.00	.0	-115.4	.0
Total	.00	34393	.00	48.12	12.81	-519.7	.0	.0

Normal to Ridge - Base Reactions - Walls Only -GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Leeward Wall	-2.59	3180	.00	8.24	.00	82.4	.0	.0
Side Wall	-4.42	2360	-10.42	.00	.00	.0	104.2	.0
Side Wall	-4.42	2360	10.42	.00	.00	.0	-104.2	.0
Windward Wall	8.55	1590	.00	13.59	.00	203.9	.0	.0
Windward Wall	8.03	1590	.00	12.77	.00	63.8	.0	.0
Side Wall	-4.42	1015	-4.48	.00	.00	.0	115.4	.0
Side Wall	-4.42	1015	4.48	.00	.00	.0	-115.4	.0
Total	.00	13111	.00	34.60	.00	350.2	.0	.0

Normal to Ridge - Base Reactions - Walls+Roof MIN

Description	Press psf	Area* ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Windward Wall	9.60	1590	.00	15.26	.00	229.0	.0	.0
Windward Wall	9.60	1590	.00	15.26	.00	76.3	.0	.0
Roof Windward	4.80	2736	.00	13.13	.00	375.7	.0	.0
Roof Leeward	4.80	2736	.00	13.13	.00	375.7	.0	.0
OH Top Windward	4.80	162	.00	0.78	.00	15.2	.0	.0
OH Top Leeward	4.80	162	.00	0.78	.00	15.2	.0	.0
Total	.00	8977	.00	58.35	.00	1087.0	.0	.0

Notes - Normal to Ridge

Note (1) Per Fig 27.4-1 Note 7, Since Theta > 10 Deg base calcs on Mean Ht
Note (2) Wall & Roof Pressures = Qh*(G*Cp - GCPi)
Note (3) +GCpi = Positive Internal Bldg Press, -GCPi = Negative Internal Bldg Press
Note (4) Total Pressure = Leeward Press + Windward Press (For + or - GCPi)
Note (5) Ref Fig 27.4-1, Normal to Ridge (Theta>=10), Theta= 16.3 Deg, h/l= 0.18
Note (6) No internal pressure considered (GCpi = 0) for Overhang
Note (7) Overhang bottom based upon windward wall Cp and GCpi = 0.
Note (8) X= Along Building ridge, Y = Normal to Building Ridge, Z = Vertical
Note (9) MIN = Minimum pressures on Walls = 9.6 psf and Roof = 4.8 psf
Note (10) Area* = Area of the surface projected onto a vertical plane normal to wind.

MWFRS-Wall Pressures for Wind Normal to 118 ft wall (Along Ridge)

All pressures shown are based upon ASD Design, with a Load Factor of .6

Wall	Cp	Pressure +GCpi (psf)	Pressure -GCpi (psf)
Leeward Wall	-0.43	-5.89	-1.96
Side Walls	-0.70	-8.35	-4.42

Wall	Elev ft	Kz	Kzt	Cp	qz psf	Press +GCpi	Press -GCpi	Total +/-GCpi
Windward	37.21	0.75	1.00	0.80	11.77	5.90	9.83	11.79
Windward	20.00	0.62	1.00	0.80	9.86	4.62	8.55	10.51
Windward	10.00	0.57	1.00	0.80	9.08	4.10	8.03	9.99

Roof - Dist from Windward Edge	Cp	Pressure +GCpi (psf)	Pressure -GCpi (psf)
Roof: 0.0 ft to 14.3 ft	-0.90	-10.17	-6.24
Roof: 14.3 ft to 28.6 ft	-0.90	-10.17	-6.24
Roof: 28.6 ft to 57.2 ft	-0.50	-6.52	-2.59
Roof: 57.2 ft to 159.0 ft	-0.30	-4.70	-0.77
OH Top : 0.0 ft to 14.3 ft	-0.90	-12.13	-12.13
Overhang Top : 14.3 ft to 28.6 ft	-0.90	-12.13	-12.13
Overhang Top : 28.6 ft to 57.2 ft	-0.50	-8.49	-8.49
Overhang Top : 57.2 ft to 159.0 ft	-0.30	-6.67	-6.67

Along Ridge - Base Reactions - Walls+Roof +GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Leeward Wall	-5.89	2360	13.90	.00	.00	.0	-139.0	.0
Side Wall	-8.35	3180	.00	26.54	.00	265.4	.0	.0
Side Wall	-8.35	3180	.00	-26.54	.00	-265.4	.0	.0
Windward Wall	4.62	1180	5.45	.00	.00	.0	-81.7	.0
Windward Wall	4.10	1180	4.84	.00	.00	.0	-24.2	.0
Roof (0 to h/2)	-10.17	879	.00	-2.50	8.58	181.5	-620.8	-181.1
Roof (0 to h/2)	-10.17	879	.00	2.50	8.58	-181.5	-620.8	181.1
OH Top	-12.13	52	.00	-0.18	0.61	33.4	-43.9	-12.8
OH Top	-12.13	52	.00	0.18	0.61	-33.4	-43.9	12.8
Roof (h/2 to h)	-10.17	879	.00	-2.50	8.58	181.5	-498.1	-145.3
Roof (h/2 to h)	-10.17	879	.00	2.50	8.58	-181.5	-498.1	145.3
OH Top	-12.13	52	.00	-0.18	0.61	33.4	-35.3	-10.3
OH Top	-12.13	52	.00	0.18	0.61	-33.4	-35.3	10.3
Roof (h to 2h)	-6.52	1758	.00	-3.21	11.01	232.9	-402.8	-117.5
Roof (h to 2h)	-6.52	1758	.00	3.21	11.01	-232.9	-402.8	117.5
OH Top	-8.49	104	.00	-0.25	0.85	46.8	-31.1	-9.1

OH Top	-8.49	104	.00	0.25	0.85	-46.8	-31.1	9.1
Roof (>2h)	-4.70	6256	.00	-8.23	28.23	597.2	807.4	235.5
Roof (>2h)	-4.70	6256	.00	8.23	28.23	-597.2	807.4	-235.5
OH Top	-6.67	371	.00	-0.69	2.37	130.8	67.9	19.8
OH Top	-6.67	371	.00	0.69	2.37	-130.8	67.9	-19.8
Leeward Wall	-5.89	1015	5.98	.00	.00	.0	-153.9	.0
Windward Wall	5.90	1015	5.99	.00	.00	.0	-154.1	.0
Total	.00	33814	36.15	.00	121.67	.0	-2066.3	.0

Along Ridge - Base Reactions - Walls Only +GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Leeward Wall	-5.89	2360	13.90	.00	.00	.0	-139.0	.0
Side Wall	-8.35	3180	.00	26.54	.00	265.4	.0	.0
Side Wall	-8.35	3180	.00	-26.54	.00	-265.4	.0	.0
Windward Wall	4.62	1180	5.45	.00	.00	.0	-81.7	.0
Windward Wall	4.10	1180	4.84	.00	.00	.0	-24.2	.0
Leeward Wall	-5.89	1015	5.98	.00	.00	.0	-153.9	.0
Windward Wall	5.90	1015	5.99	.00	.00	.0	-154.1	.0
Total	.00	13111	36.15	.00	.00	.0	-552.9	.0

Along Ridge - Base Reactions - Walls+Roof -GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Leeward Wall	-1.96	2360	4.62	.00	.00	.0	-46.2	.0
Side Wall	-4.42	3180	.00	14.04	.00	140.4	.0	.0
Side Wall	-4.42	3180	.00	-14.04	.00	-140.4	.0	.0
Windward Wall	8.55	1180	10.09	.00	.00	.0	-151.3	.0
Windward Wall	8.03	1180	9.47	.00	.00	.0	-47.4	.0
Roof (0 to h/2)	-6.24	879	.00	-1.54	5.26	111.4	-380.9	-111.1
Roof (0 to h/2)	-6.24	879	.00	1.54	5.26	-111.4	-380.9	111.1
OH Top	-12.13	52	.00	-0.18	0.61	33.4	-43.9	-12.8
OH Top	-12.13	52	.00	0.18	0.61	-33.4	-43.9	12.8
Roof (h/2 to h)	-6.24	879	.00	-1.54	5.26	111.4	-305.6	-89.1
Roof (h/2 to h)	-6.24	879	.00	1.54	5.26	-111.4	-305.6	89.1
OH Top	-12.13	52	.00	-0.18	0.61	33.4	-35.3	-10.3
OH Top	-12.13	52	.00	0.18	0.61	-33.4	-35.3	10.3
Roof (h to 2h)	-2.59	1758	.00	-1.28	4.37	92.6	-160.1	-46.7
Roof (h to 2h)	-2.59	1758	.00	1.28	4.37	-92.6	-160.1	46.7
OH Top	-8.49	104	.00	-0.25	0.85	46.8	-31.1	-9.1
OH Top	-8.49	104	.00	0.25	0.85	-46.8	-31.1	9.1
Roof (>2h)	-0.77	6256	.00	-1.35	4.62	97.8	132.2	38.5
Roof (>2h)	-0.77	6256	.00	1.35	4.62	-97.8	132.2	-38.5
OH Top	-6.67	371	.00	-0.69	2.37	130.8	67.9	19.8
OH Top	-6.67	371	.00	0.69	2.37	-130.8	67.9	-19.8
Leeward Wall	-1.96	1015	1.99	.00	.00	.0	-51.2	.0
Windward Wall	9.83	1015	9.98	.00	.00	.0	-256.8	.0
Total	.00	33814	36.15	.00	47.92	.0	-2066.3	.0

Along Ridge - Base Reactions - Walls Only -GCpi

Description	Press psf	Area ft^2	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Leeward Wall	-1.96	2360	4.62	.00	.00	.0	-46.2	.0
Side Wall	-4.42	3180	.00	14.04	.00	140.4	.0	.0
Side Wall	-4.42	3180	.00	-14.04	.00	-140.4	.0	.0
Windward Wall	8.55	1180	10.09	.00	.00	.0	-151.3	.0
Windward Wall	8.03	1180	9.47	.00	.00	.0	-47.4	.0
Leeward Wall	-1.96	1015	1.99	.00	.00	.0	-51.2	.0
Windward Wall	9.83	1015	9.98	.00	.00	.0	-256.8	.0
Total	.00	13111	36.15	.00	.00	.0	-552.9	.0

Along Ridge - Base Reactions - Walls+Roof MIN

Description	Press	Area*	Fx	Fy	Fz	Mx	My	Mz

	psf	ft^2	Kip	Kip	Kip	K-ft	K-ft	K-ft
Windward Wall	9.60	1180	11.33	.00	.00	.0	-169.9	.0
Windward Wall	9.60	1180	11.33	.00	.00	.0	-56.6	.0
Windward Wall	9.60	1015	9.75	.00	.00	.0	-250.8	.0
Total	.00	3375	32.40	.00	.00	.0	-477.4	.0

Notes - Along Ridge
Note (1) OH = Overhang, no internal pressure considered for Overhang (GCpi=0)
Note (2) Ref Fig 27.4-1, Parallel to Ridge (All), h/l= 0.18
Note (3) X= Along Building ridge, Y = Normal to Building Ridge, Z = Vertical
Note (4) MIN = Minimum pressures on Walls = 9.6 psf and Roof = 4.8 psf
Note (5) Area* = Area of the surface projected onto a vertical plane normal to wind.

Total Base Reaction Summary

Description	Fx Kip	Fy Kip	Fz Kip	Mx K-ft	My K-ft	Mz K-ft
Normal to Ridge Walls+Roof +GCpi	.0	35.6	129.3	384.8	.0	.0
Normal to Ridge Walls Only +GCpi	.0	34.6	.0	350.2	.0	.0
Normal to Ridge Walls+Roof -GCpi	.0	48.1	12.8	-519.7	.0	.0
Normal to Ridge Walls Only -GCpi	.0	34.6	.0	350.2	.0	.0
Normal to Ridge Walls+Roof MIN	.0	58.4	.0	1087.0	.0	.0
Along Ridge Walls+Roof +GCpi	36.2	.0	121.7	.0	-2066.3	.0
Along Ridge Walls Only +GCpi	36.2	.0	.0	.0	-552.9	.0
Along Ridge Walls+Roof -GCpi	36.2	.0	47.9	.0	-2066.3	.0
Along Ridge Walls Only -GCpi	36.2	.0	.0	.0	-552.9	.0
Along Ridge Walls+Roof MIN	32.4	.0	.0	.0	-477.4	.0

Notes Applying to MWFRS Reactions:
Note (1) Per Fig 27.4-1, Note 9, Use greater of Shear calculated with or without roof.
Note (2) X= Along Building ridge, Y = Normal to Building Ridge, Z = Vertical
Note (3) MIN = Minimum pressures on Walls = 9.6 psf and Roof = 4.8 psf
Note (4) MIN area is the area of the surface onto a vertical plane normal to wind.
Note (5) Total Roof Area (incl OH Top) = 20703.1 sq. ft

Wind Pressure on Components and Cladding (Ch 30 Part 1)
All pressures shown are based upon ASD Design, with a Load Factor of .6

Width of Pressure Coefficient Zone "a" = = 11.44 ft

Description	Width ft	Span ft	Area ft^2	Zone	Max GCp	Min GCp	Max P psf	Min P psf
	10.00	1.00	10.0	4	1.00	-1.10	13.06	-14.17
	20.00	1.00	20.0	4	0.95	-1.05	12.47	-13.58
	50.00	1.00	50.0	4	0.88	-0.98	11.69	-12.80
	100.00	1.00	100.0	4	0.82	-0.92	11.11	-12.21
	10.00	1.00	10.0	5	1.00	-1.40	13.06	-17.49
	20.00	1.00	20.0	5	0.95	-1.29	12.47	-16.31
	50.00	1.00	50.0	5	0.88	-1.15	11.69	-14.75
	100.00	1.00	100.0	5	0.82	-1.05	11.11	-13.58

Khcc:Comp. & Clad. Table 6-3 Case 1 = 0.70
Qhcc:.00256*V^2*Khcc*Kht*Kd = 11.07 psf

BY [redacted]	DATE 10/18/2018	[redacted]	SHEET NO.	OF
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Design Loads - Wood Superstructure

	Dead Load	Live Load
ROOF		
Future PV Panels	3.5 psf (Use 1.5 psf for Seismic)	
Composition Shingle	5.0 psf	
1/2" Plywood	1.5 psf	
Manufactured Roof Trusses @ 24" o.c.	3.0 psf	
Insulation (12 " @ 0.04 psf/in)	0.5 psf	
MEP & Sprinklers	2.0 psf	
2 Layers 5/8" gypboard ceiling	5.6 psf	
Miscellaneous	1.9 psf	
	23.0 psf	20.0 psf (Reducible)
UNIT FLOORS		
Finish (Carpet / Thin Set Tile)	1.5 psf	
1-1/4" Gypcrete	12.0 psf	
Acoustimat	0.5 psf	
3/4" OSB	2.3 psf	
11-7/8" Floor I-Joists @ 16" o.c.	2.3 psf	
Insulation (6.0 " @ 0.2 psf/in)	1.2 psf	
Mech/Elec/Sprklrs	2.0 psf	
2 layers 5/8" Gypboard + RC Channel	5.6 psf	
Miscellaneous	2.6 psf	
	30.0 psf	40.0 psf
	+ 3 psf (At Kitchen and Bath)	
UNIT FLOORS ABOVE COMMON AREAS		
Typical Unit Floor Assembly (See above)	30.0 psf	
Drop Ceiling	5.0 psf	
	35.0 psf	40.0 psf

Wood Beam

\Jobs\17-Jobs\17-185 Sunflower Irby Ranch\Calc\Gravity\Community\17-185_Community Beams & Headers.ec6

Lic. # : KW-06009713 **Licensee : Peoples Associates Structural Engineers**

Description : C.RB.2

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
Length = 13.0 ft	1	0.350	0.306	1.15	0.994	1.00	1.00	1.00	1.00	1.00	16.84	962.02	2744.77	6.53	93.29	304.75
+D+0.750Lr+0.750L+H					0.994	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.437	0.383	1.25	0.994	1.00	1.00	1.00	1.00	1.00	22.80	1,302.96	2983.44	8.88	126.89	331.25
+D+0.750L+0.750S+H					0.994	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.350	0.306	1.15	0.994	1.00	1.00	1.00	1.00	1.00	16.84	962.02	2744.77	6.53	93.29	304.75
+D+0.60W+H					0.994	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.252	0.220	1.60	0.994	1.00	1.00	1.00	1.00	1.00	16.84	962.02	3818.80	6.53	93.29	424.00
+D+0.70E+H					0.994	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.252	0.220	1.60	0.994	1.00	1.00	1.00	1.00	1.00	16.84	962.02	3818.80	6.53	93.29	424.00
+D+0.750Lr+0.750L+0.450W+H					0.994	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.341	0.299	1.60	0.994	1.00	1.00	1.00	1.00	1.00	22.80	1,302.96	3818.80	8.88	126.89	424.00
+D+0.750L+0.750S+0.450W+H					0.994	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.252	0.220	1.60	0.994	1.00	1.00	1.00	1.00	1.00	16.84	962.02	3818.80	6.53	93.29	424.00
+D+0.750L+0.750S+0.5250E+H					0.994	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.252	0.220	1.60	0.994	1.00	1.00	1.00	1.00	1.00	16.84	962.02	3818.80	6.53	93.29	424.00
+0.60D+0.60W+0.60H					0.994	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.151	0.132	1.60	0.994	1.00	1.00	1.00	1.00	1.00	10.10	577.21	3818.80	3.92	55.97	424.00
+0.60D+0.70E+0.60H					0.994	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 13.0 ft	1	0.151	0.132	1.60	0.994	1.00	1.00	1.00	1.00	1.00	10.10	577.21	3818.80	3.92	55.97	424.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr+H	1	0.3322	6.168		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	10.317	5.689
Overall MINimum	3.335	1.785
+D+H	6.981	3.904
+D+L+H	6.981	3.904
+D+Lr+H	10.317	5.689
+D+S+H	6.981	3.904
+D+0.750Lr+0.750L+H	9.483	5.243
+D+0.750L+0.750S+H	6.981	3.904
+D+0.60W+H	6.981	3.904
+D+0.70E+H	6.981	3.904
+D+0.750Lr+0.750L+0.450W+H	9.483	5.243
+D+0.750L+0.750S+0.450W+H	6.981	3.904
+D+0.750L+0.750S+0.5250E+H	6.981	3.904
+0.60D+0.60W+0.60H	4.189	2.343
+0.60D+0.70E+0.60H	4.189	2.343
D Only	6.981	3.904
Lr Only	3.335	1.785
L Only		
S Only		
W Only		
E Only		
H Only		

BY [redacted] DATE 12/28/18

CHKD. BY ______ DATE ________

[redacted]

SHEET NO. ______ OF ______

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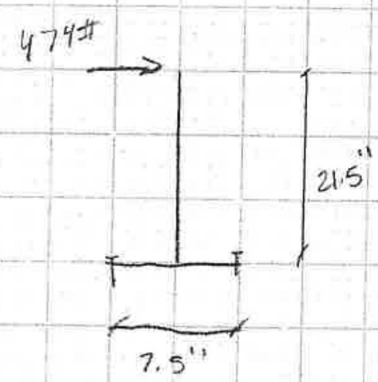
$E_h = 0.603\,w$

$w = 314\ ft^2 \times 10\,psf = 3140\#$

$E_h = 1894\# / 4 = 474\#$ @ EA. Beam support



MAX Tension $= \dfrac{474 \times 21.5}{7.5} = 1360\#$

MAX shear $= \dfrac{474}{2} = 237\#$

Check welded studs

Tension cap : $B_{ab} = 1.25\ A_{pt}\sqrt{f'_m}$

$B_{as} = 0.6\ A_b\ f_y$

$A_{pt} = \pi (L_b)^2$, 8" CMU ∴ use 4" for L_b

$A_{pt} = 50.26\,in^2$

$f'_m = 1500\,psi$

$B_{ab} = 2433\#$ — governs (Tension cap.)

$B_{as} = 9543\#$

BY [redacted] DATE 12/28/18

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SHEET NO. ______ OF ______

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SHEAR CAP:

$A_{pv} = \frac{\pi(4)^2}{2} = 25.13 \text{ in}^2$

$B_{vb} = 1.25 \, A_{pv} \sqrt{f'_m} = 1217\#$ — Governs.

$B_{vc} = 350 \sqrt[4]{f'_m A_b} = 1775\#$

$B_{vpry} = 2.0 \, B_{ab} = 2.5 \, A_{pt} \sqrt{f'_m} = 4866\#$

$B_{vs} = 0.36 \, A_b f_y = 5726\#$

$1217\# \times 2 = 2434\#$ (shear cap)

$\frac{1360}{2433} + \frac{474}{2434} = 0.75 < 1$

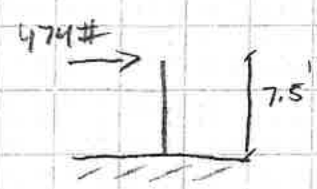
∴ (2) 3/4" Φ welded stud (6" Long) OK!

Check Bolts

5/8" Φ MB

$Z_{//} = 2410\#$ (double shear) > 474 ∴ (2) 5/8" Φ Bolts Ok.

Check Side PLATE (Assume 1/2" plate)



$M_u = 3555$ Lb-in.

$Z = \frac{8" \times (0.5")^2}{4} = 0.5 \text{ in}^3$

$M_n = F_y Z = \Phi \, 36 \text{ ksi} \times 0.5 = 16200$ lb-in $> M_u$ ∴ 1/2" side PL OK

BY [redacted] DATE 1/4/19

CHKD. BY ______ DATE ________

[redacted]

SHEET NO. ______ OF ______

JOB NO. 17.185

[redacted]

CHECK lag screws

• Lag screws resist gravity Forces only

$\frac{765\#}{4} = 192\#$ per screw

$\frac{1}{2}$" Φ Lag screw w/ 4" (min) embed CAP = 378#/inch = 1512#

∴ use $\frac{1}{2}$" Φ Lag screw w 4" min. embed.

CHECK WOOD SCREWS

V per bracket = 165#

2 - #10 wood screws w/ 4" (min.) Embed CAP = (2) 159 = 318#

∴ V CAP is OK.

2 - #10 wood screws w/ 4" (min.) Embed CAP = (2) 578 = 1156#

∴ withdrawal CAP is OK.

∴ use 2 - #10 wood screws w/ 4" (min.) Embed.

BY [redacted] DATE 1/4/19

CHKD. BY ______ DATE ______

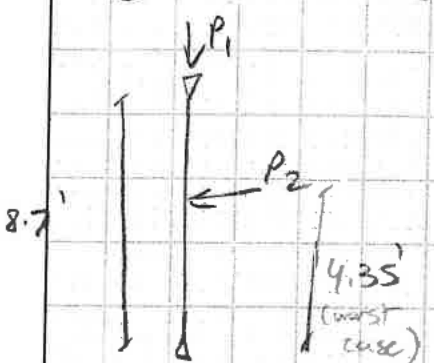
[redacted]

SHEET NO. ______ OF ______

JOB NO. 17-185

[redacted]

V @ each bracket = 165 #



$P_1 = R_{DL} = 2' \times 23\ psf = 46\ plf \times 1.33$

$R_{Lr} = 2' \times 20\ psf = 40\ plf \times 1.33$

$P_{1DL} = 62\#$

$P_{1Lr} = 54\#$

$P_{2(E)} = 165\# \times 1.4 = 231\#$.

See Following for Analysis

2x6 DF #1 @ 16" oc OK.