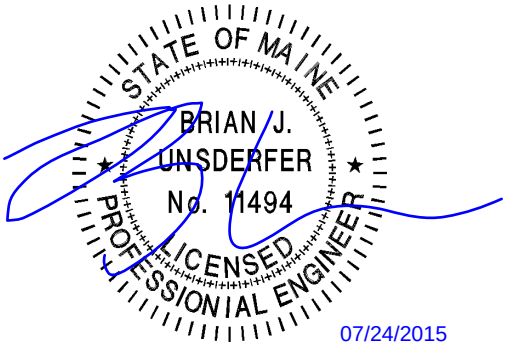
ARMOUR UNSDERFER ENGINEERING INC., P.S.
555 116th Ave. NE, Suite. 118
Bellevue, WA 98004 (425) 614-0949 Fax (425) 614-0950

STRUCTURAL CALCULATIONS FOR:

Fat Face Portland
Portland ME
04104

Client:

Design Republic
501 Madison Ave, 11th Floor
New York, NY



By
Edgar Del Rio
Brian Unsderfer, SE, SECB

AUE No. 15231
Date 7/20/2015

IBC Seismic		
Non-Structural Systems	IBC	2009

Occ. Category:	II	*Standard-Occupancy Building*		Fa =	1.55	Sms =	0.485
Ss=	0.313	hn =	15 ft	Fv =	2.40	Sm1 =	0.185
S1=	0.077	Ct =	0.02				
Ie =	1.00	SDC:	B	Sds=	0.323		
				Sd1=	0.123		
Site Class =	D						

Zipcode:	04104

Seismic Variables from USGS			Seismic Variables from User Input	
	Max	Loc		Max
Ss =	-	-	**Ss =**	0.3130
S1 =	-	-	**S1 =**	0.0770
PGA =	-	-		
	Min	Loc		
Ss =	-	-		
S1 =	-	-		
PGA =	-	-		

See ASCE 13.3 Non Structural Systems

MEC1. Air-Side HVAC, Fans, Air Handlers, Air Conditioning Units, Cabinet Heaters, Air Distribution Boxes, and Other Mechanical Components Constructed of Sheet Metal Framing

Ap = 2.5 z = 15.00 ft Ip = 1.0
Rp = 6.0 h = 15.00 ft
Ωo = 2.5

Fp = 0.16 * Wp — **Fp = 0.162 * Wp**
Fp max = 0.52 * Wp
Fp min = 0.10 * Wp
With Overstrength factor: ***Fp = 0.404 * Wp***

-

A14. Ceilings - All

Ap = 1.0 z = 15.00 ft Ip = 1.5
Rp = 2.5 h = 15.00 ft
Ωo = 2.5

Fp = 0.23 * Wp — **Fp = 0.233 * Wp**
Fp max = 0.78 * Wp
Fp min = 0.15 * Wp
With Overstrength factor: ***Fp = 0.582 * Wp***

Client Name

Client Address

Client Address

Ap = #N/A z = 15.00 ft Ip = 1.0
Rp = #N/A h = 15.00 ft
Ωo = #N/A

Fp = #N/A * Wp — **Fp = #N/A * Wp**
Fp max = 0.52 * Wp
Fp min = 0.10 * Wp
With Overstrength factor: ***Fp = #N/A * Wp***

-

-

Ap = - z = 15.00 ft Ip = 1.0
Rp = - h = 15.00 ft
Ωo = -

Fp = #VALUE! * Wp — **Fp = #VALUE! * Wp**
Fp max = 0.52 * Wp
Fp min = 0.10 * Wp
With Overstrength factor: ***Fp = #VALUE! * Wp***

Armour Unsderfer Eng Inc., P.S., Job Number

Project:	Fat Face Portland	Pg. No:	C&C Seis 1
Address:	Portland ME	Date:	7/20/15
Client:	Design Republic	Job No.:	15231

Snow Drift -Typical for Side Elevations

Building Occupancy =	II	
Ground Snow (P_g) =	50	psf
Terrain Category =	B	
Exposure =	Fully	
Exposure Factor (C_e) =	0.9	
Thermal Factor (C_t) =	1.20	
Importance Factor (I) =	1.00	

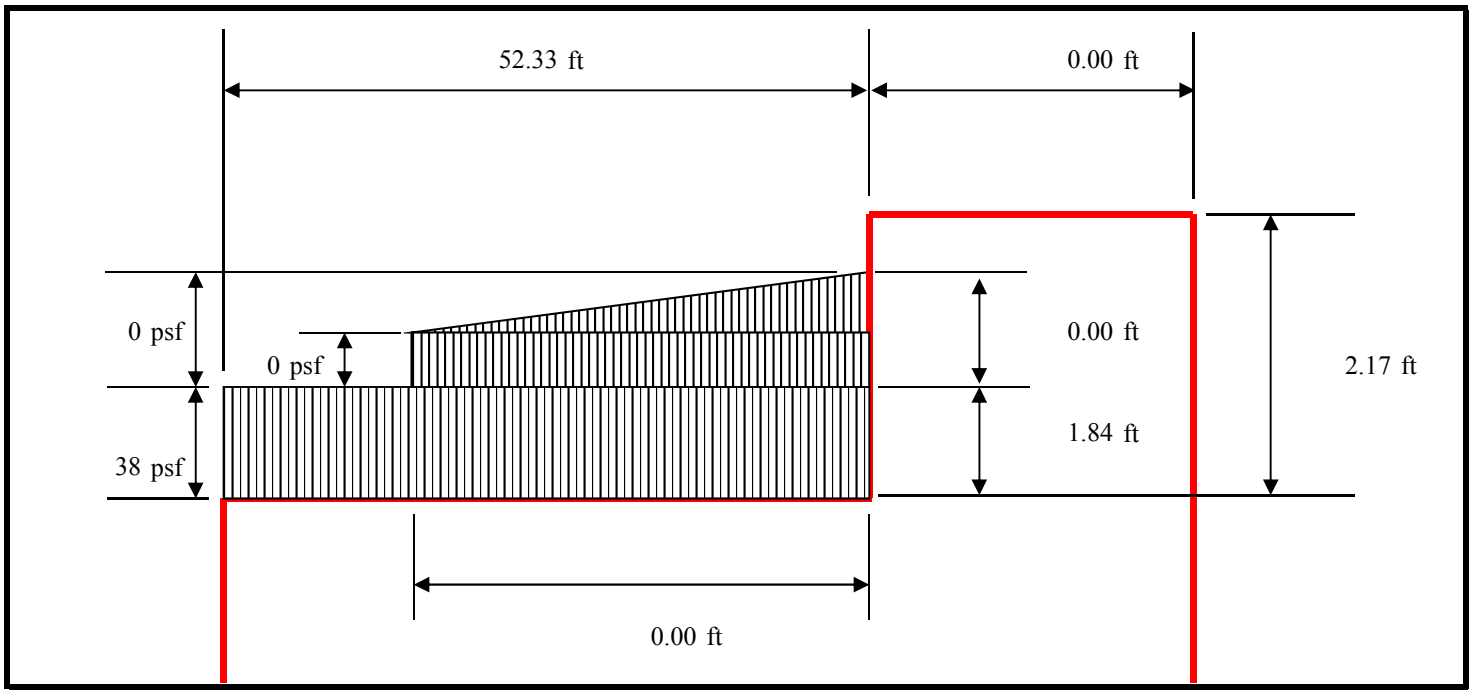
Flat Roof Snow Load (P_f) =	37.80	psf
Code Minimum Flat Roof Snow Load ($P_{f,min}$) =	25	psf
Horizontal Dimension ($l_{u,upper\ roof}$) =	0.00	ft
Horizontal Dimension ($l_{u,lower\ roof}$) =	52.33	ft
Roof Elevation Change (h_r) =	2.17	ft

Leeward Drift Surcharge (h_{d1}) =	2.00	ft
Windward Drift Surcharge (h_{d2}) =	2.23	ft

Drift Height (h_d) =	0.00	ft
Snow Density =	20.50	pcf
Base Snow Height (h_b) =	1.84	ft

Drift Need not be Considered

Drift Intensity (Pm1) =	0	psf
Truncated Drift Intensity (Pm2) =	0	psf
Width of Drift (Wd) =	0.00	ft



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Project:	Fat Face Portland	Pg. No:	Snow 1
Address:	Portland ME	Date:	7/20/15
Client:	Design Republic	Job No.:	15231

IBC Seismic	
Main Seismic Force Resisting Systems	**IBC 2009**

Occ. Category:	II	*Standard-Occupancy Building*		Fa =	1.55	Sms =	0.485
Ss=	0.313	hn =	15 ft	Fv =	2.40	Sm1 =	0.185
S1=	0.077	Ct =	0.02				
Ie =	1.00	SDC:	B	Sds=	0.323		
				Sd1=	0.123		
Site Class =	D						

Zipcode:	4104

Seismic Variables from USGS			Seismic Variables from User Input	
	Max	Loc		Max
Ss =	-	-	**Ss =**	**0.3130**
S1 =	-	-	**S1 =**	**0.0770**
PGA =	-	-		
	Min	Loc		
Ss =	-	-		
S1 =	-	-		
PGA =	-	-		

See ASCE 12.8 Equiv. Lateral Force Procedure

A11. Ordinary Plain Masonry Shear Walls

R =	1.5	Ωo =	2.5	Cd =	1.3

Cs = 0.216

T = 0.152 Seconds

Cs max = 0.539

Cs min = 0.010 S1 < 0.6

Cs min = Not Applicable S1 >= 0.6

Cs min = 0.010

V = 0.216 * W

Armour Unsderfer Eng Inc., P.S., 555 116th Ave. NE, Suite 118, Bellevue, WA 98004 (425) 614-0949

Project:	Fat Face Portland	Pg. No:	MSFRS 1
Address:	Portland ME	Date:	7/20/15
Client:	Design Republic	Job No.:	15231

IBC Seismic

Vertical Shear Distribution of Seismic Forces

Level	Floor		Walls			Misc. Weight	Total Weight		Mass
	Area (ft^2)	Weight (psf)	Total L (ft)	Trib Ht (ft)	Weight (psf)	(lb)	(lb)	Elevation (ft)	Irregularity
1	0	0	0.00	11.04	0	0	0	0.00	N/A
2	10,487	130	361.85	17.94	112	500	2,087,720	22.08	OK
3	10,487	130	361.85	13.90	97	500	1,849,861	35.87	OK
4	10,487	130	361.85	14.00	97	500	1,853,534	49.87	OK
5	10,487	130	361.85	7.00	97	0	1,608,179	63.87	N/A
6	0	50	0.00	0.00	0	0	0	0.00	N/A
7	0	50	0.00	0.00	0	0	0	0.00	N/A
8	0	0	0.00	0.00	0	0	0	0.00	N/A
9	0	0	0.00	0.00	0	0	0	0.00	N/A
10	0	0	0.00	0.00	10	0	0	0.00	N/A

- Note: Tributary Weights on Level 1 are ignored because level 1 is considered to be Grade Level

Base Shear:	V =	**1595.04** k	LRFD

Level	h^x	wh^x	Cvx	Fx (k) (LRFD)
1	0	0	0.000	0.00
2	22.08	46,096,847	0.150	239.03
3	35.87	66,354,503	0.216	344.08
4	49.87	92,435,717	0.301	479.32
5	63.87	102,714,385	0.334	532.62
6	0	0	0.000	0.00
7	0	0	0.000	0.00
8	0	0	0.000	0.00
9	0	0	0.000	0.00
10	0	0	0.000	0.00
	Total:	307,601,452	1.000	1595.04

Wall On Grid A: (Prior to Removal)

Pier	Length	Height		h/L	Df	RF	Shear Load:
1	0.00	0.00		0	0	0	0.00 k
2	8.83	14.37		1.627	0.919	1.088	384.04 k
3	8.99	14.37		1.599	0.888	1.126	397.25 k
4	2.46	14.37		5.846	21.73	0.046	16.23 k

Wall On Grid A: (After Removal)

Pier	Length	Height		h/L	Df	RF	Shear Load:	% Change:
1	0.00	0.00		0	0	0	0.00 k	#DIV/0!
2	8.83	14.37		1.627	0.919	1.088	392.02 k	2%
3	8.99	14.37		1.599	0.888	1.126	405.50 k	2%
4	0.00	0.00		0	0	0	0.00 k	-100%

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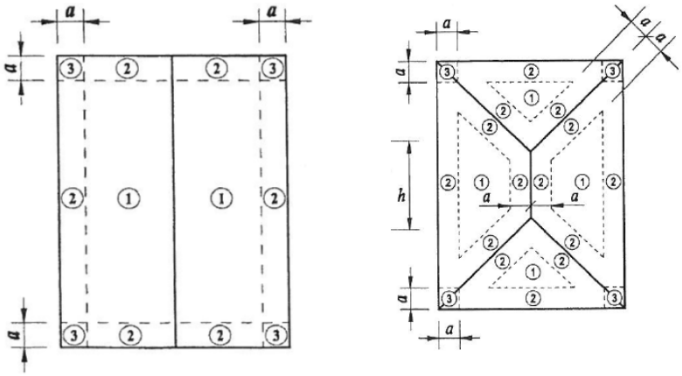
Project:	Fat Face Portland	Pg. No:	MSFRS 2
Address:	Portland ME	Date:	7/20/15
Client:	Design Republic	Job No.:	15231

Wind Loads - Components and Cladding, Roofs

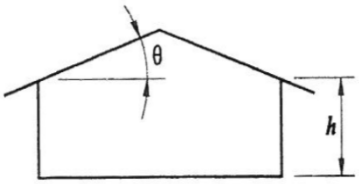
ASCE 7-10 Chapter 30, Part 1 - Low Rise Buildings



Design Criteria

Risk Category:	II	
Basic Wind Speed:	101 mph	
Exposure Category:	B	
Hurricane Prone:	No	
Building Type:	Enclosed Buildings	
Length:	63.00 ft	Longer Dimension
Width:	36.00 ft	Shorter Dimension
Height:	45.00 ft	
Component Exp. Category:	B	

Gable and Hip Roofs, Zone Extents



Roof Angle

Wind Load Calculation - Flat, Gable or Hip Roofs

Wind Directionality Factor, K_d:	0.85	Table 26.6-1	
Gust Effect Factor, G:	0.85	Section 26.9	
Roof Angle:	$\Theta <= 7^0$		
Zg:	1200		
α:	7.00 ft		
Kz:	0.79		
Kzt:	1.00		
qh:	17.46 psf		

Gcpi:	
	0.18
	-0.18

Zone 5 Extent: **3.60** ft

Parapet Height Above Deck: **3.00** ft

Roof External Pressure Coefficients

Area (ft^2)	Location	0	10	20	50	100	200	500
GC_{pf}	1	-1.0	-1.0	-1.0	-1.0	-0.9	-0.9	-0.9
	2	-1.8	-1.8	-1.6	-1.3	-1.1	-1.1	-1.1
	3 [1]	-1.8	-1.8	-1.6	-1.3	-1.1	-1.1	-1.1
	1, 2 & 3 [1]	0.9	0.9	0.9	0.8	0.7	0.7	0.6
	2 & 3	NA	NA	NA	NA	NA	NA	NA

1) 3' or Higher Parapet, For Flat Roof, Negative GCp in Zone 3 = Zone 2, and Positive Zone 2 & 3 = Zone 4 & 5

Overhang External Pressure Coefficients

Area (ft^2)	Location	0	10	20	50	100	200	500
GC_{pf}	1	-1.7	-1.7	-1.7	-1.6	-1.6	-1.4	-1.1
	2	-1.7	-1.7	-1.7	-1.6	-1.6	-1.4	-1.1
	3	-2.8	-2.8	-2.2	-1.4	-0.8	-0.8	-0.8

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Project:	Fat Face Portland	Pg. No:	CCRoof 1
Address:	Portland ME	Date:	7/20/15
Client:	Design Republic	Job No.:	15231

Wind Loads - Components and Cladding, Roofs

ASCE 7-10 Chapter 30, Part 1 - Buildings Less than 60ft

Roof Wind Loads

1.0W Per ASCE 7-10 LRFD Load Cases

Area (ft^2)	Location	0	10	20	50	100	200	500
P (psf)	1	-20.60	-20.60	-20.26	-19.91	-18.86	-18.86	-18.86
	2	-34.57	-34.57	-31.08	-25.84	-22.35	-22.35	-22.35
	3 [1]	-34.57	-34.57	-31.08	-25.84	-22.35	-22.35	-22.35
	1, 2 & 3	18.86	18.86	18.07	16.97	15.87	15.24	14.14
	2 & 3	NA	NA	NA	NA	NA	NA	NA

0.6W Per ASCE 7-10 ASD Load Cases

Area (ft^2)	Location	0	10	20	50	100	200	500
P (psf)	1	-12.36	-12.36	-12.15	-11.94	-11.31	-11.31	-11.31
	2	-20.74	-20.74	-18.65	-15.51	-13.41	-13.41	-13.41
	3 [1]	-20.74	-20.74	-18.65	-15.51	-13.41	-13.41	-13.41
	1, 2 & 3	11.31	11.31	10.84	10.18	9.52	9.15	8.49
	2 & 3	NA	NA	NA	NA	NA	NA	NA

Overhang Wind Loads

1.0W Per ASCE 7-10 LRFD Load Cases

Area (ft^2)	Location	0	10	20	50	100	200	500
P (psf)	1	-32.8	-32.8	-32.0	-31.1	-31.1	-27.6	-22.4
	2	-32.8	-32.8	-32.0	-31.1	-31.1	-27.6	-22.4
	3	-52.0	-52.0	-41.6	-27.6	-17.1	-17.1	-17.1

*Assumes a GC_{pi} of 0.18

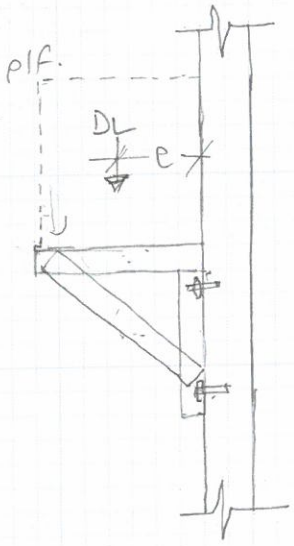
0.6W Per ASCE 7-10 ASD Load Cases

Area (ft^2)	Location	0	10	20	50	100	200	500
P (psf)	1	-19.7	-19.7	-19.2	-18.6	-18.6	-16.6	-13.4
	2	-19.7	-19.7	-19.2	-18.6	-18.6	-16.6	-13.4
	3	-31.2	-31.2	-24.9	-16.6	-10.3	-10.3	-10.3

*Assumes a GC_{pi} of 0.18

MECHANICAL SUPPORT FRAME @ EXTERIOR WALL

UNIT DL = 400 lbs.
SNOW L = 37.8 psf
W_{DL} = (400)/(3.5x3) · 1.5 = 57 plf
W_{SL} = (37.8)(1.5) = 57 plf
F_p = 0.162 (400) = 64 lbs



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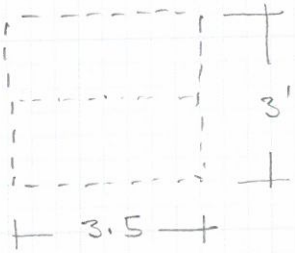
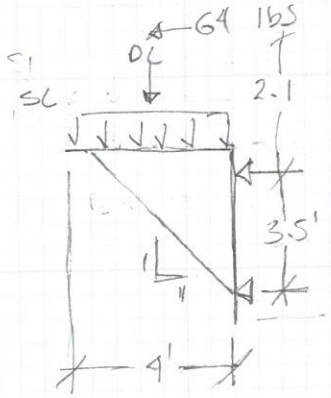
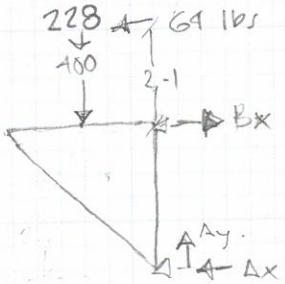


Plan view





$\Sigma F_x = A_x + 64 - B_x$ $\quad A_x = B_x - 64$

$\Sigma F_y = A_y - 400 - 228$ $\quad A_y = 628$ lbs

$\Sigma M_A = (400+228)(2) + 64(3.5+2.1) - B_x(4)$

$B_x = \frac{1614.4}{4}$ $\quad B_x = 404$ lbs

$A_x = 340$ lbs

Max Tension = 404 lbs
Max Shear = 628 lbs

> use (2) 3/4" ø Thrd'd Rod in Hilti HY70 screened epoxy grout (8" EMB)

Tallow = 1,200 lbs x2 = 2400 lbs o.k
Sallow = 1000 lbs x2 = 2000 lbs o.k

WELD CONN. CHECK

0.18 x 0.27 x 21 ksi = 1.0 k/in

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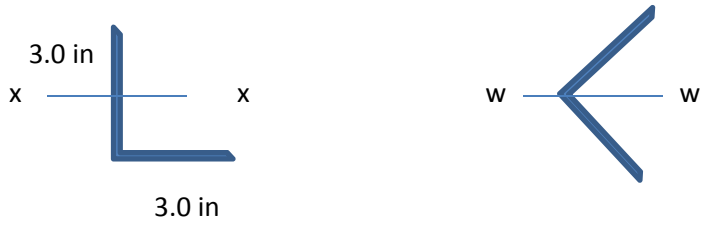
Project: 15231 - Fat Face Portland	Eng. EDR	Pg. 1
Address: Portland, ME, 04104	Ver.	
Client: Design Republic	Date: 7/16/15	Job No. 15231

Bending of Single Angle

Moment=	5.88	k-in	ASD		
Is angle continuously braced:	No				
unbraced length, Lb=	4	ft		E=	29000 ksi
Is angle braced at point of Max Moment::	No			Fy=	36 ksi
Define axis of bending:	w-w Short leg in compression				



Try Angle **L3X3X3/16** **LLV**

Properties		Axis X-X		Axis Y-Y		Axis z-z	
A=	1.09 in^2	Ix=	0.948 in^4	Iy=	0.948 in^4	Iz=	0.373 in^4
t=	0.188 in	Sx=	0.433 in^3	Sy=	0.433 in^3	Sz=	0.326 in^3
J=	0.0136 in^4	rx=	0.933 in	ry=	0.933 in	rz=	0.586 in
Cw=	0.00899 in^6						

1. Yielding

S=	0.326	in^3
My=	9.4	k-in
Mn=	14.1	k-in

2. Lateral Torsional Buckling

	My	Me	Mn		
Equal leg angle Tension at toe	12.5	295.5	18.7	na	
Equal leg angle compression at toe	12.5	42.5	16.0	na	
Equal leg angle moment in either direction			16.0	na	
Equal angle about mpa	9.4	88.4	14.1		16.0
Unequal angle about mpa +beta	9.4	166.6	14.1	na	
Unequal angle about mpa -beta	9.4	166.6	14.1	na	
Unequal angle about mpa moment in either direction			14.1	na	
			Mn=		16.0 k-in

2. Leg Local buckling

λ=	15.96	
λp=	15.3	
λr=	25.8	outstanding leg is: non-compact

Mn, [k-in]= 13.735

Final	Mn, [k-in]=	13.7	
	Omega=	1.67	
	Moment Capacity=	**8.2 k-in**	**OK**

FLOOR JOIST

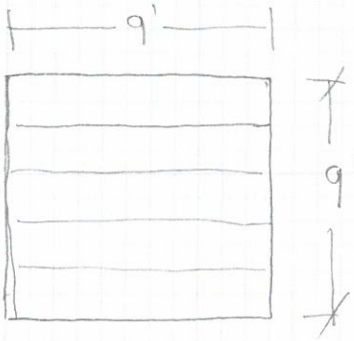
600S162-43 @ 24" O.C

LL = 20 PSF

ACUNIT DL = 360 lbs

FLOOR DL = 15 psf.

MIS. DL = 5 psf.

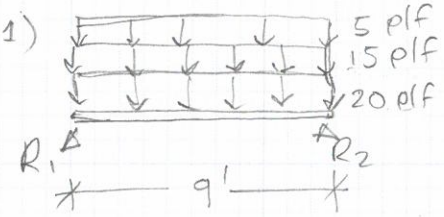


600 S162-43

$M_a = 16.68$ in-K

$V_a = 1416$ lbs

Case 1)

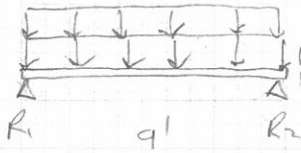


$$M_{max} = \frac{(20+15+5)(9)^2}{8} = 405 \text{ ft-lb} = 4.86 \text{ in-K}$$

$$V_{max} = \frac{(20+15+5)(9)}{2} = 180 \text{ lbs}$$

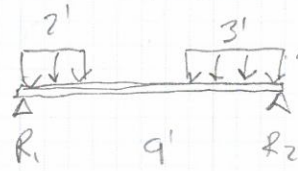
$$R_1 = R_2 = 180 \text{ lbs}$$

Case 2) with UNIT DL.



$$M_{max} = \frac{(15+5)(9^2)}{8} = 203 \text{ ft-lb} = 2.43 \text{ in-K}$$

$$V_{max} = (15+5)(9)/2 = 90 \text{ lbs}$$

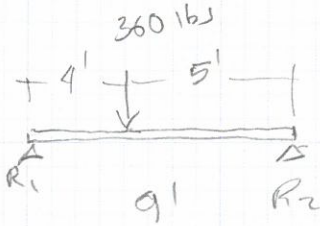


$$V_1 = R_1 = \frac{(20)(2)(2\times9-2)+20(3)^2}{2(9)} = 45 \text{ lbs.}$$

$$V_2 = R_2 = \frac{(20)(3)(18-2)+20(2)^2}{18} = 58 \text{ lbs}$$

$$M_{max_x} = \frac{R_1^2}{2w_1} = 51 \text{ ft-lb} = 0.61 \text{ in-K}$$

$$M_{max_c} = \frac{R_2^2}{2w_2} = 84 \text{ ft-lb} = 1.0 \text{ in-K}$$



$$V_{max} = \frac{Pb}{l} = \frac{360(5)}{9} = 200 \text{ lbs}$$

$R_1 = 200$ lbs $R_2 = 160$

$$M_{max} = \frac{Pab}{l} = \frac{(360)(5)(4)}{9} = 800 \text{ ft-lb} = 9.6 \text{ in-K}$$

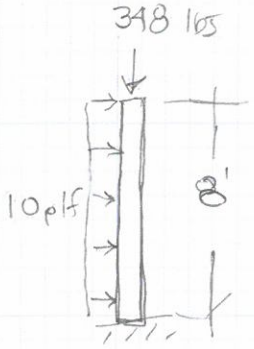
$V_{max} = 348$ lbs. O.K

Total $M_{max} = 2.43 + 1.61 + 9.6$

$= 13.64$ O.K

METAL STUDS

3625162-33 @ 16" O.C



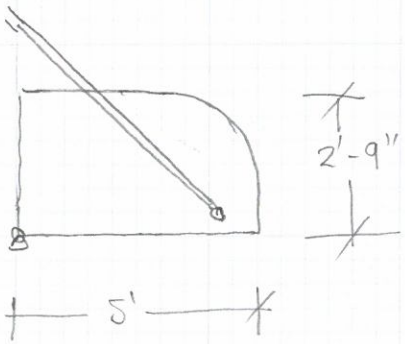
Shear = 348 lbs

Moment = $\frac{wl^2}{8} = \frac{10(8)^2}{8}$ = 80 ft-lb = 0.96 in-K

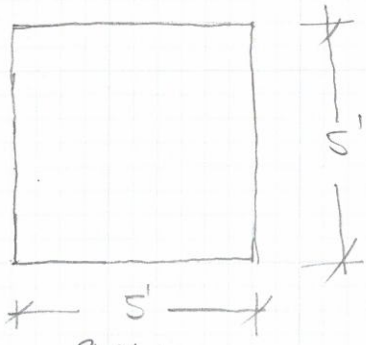
3625162-33 Va = 1.80 in-K O.K.

Ma = 1.64 K O.K

CANOPY CALCS



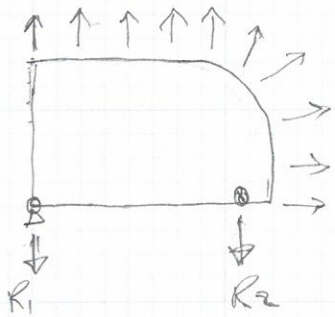
SIDE VIEW



PLAN

Projected Area = 25 ft²

Case 1) WIND LOAD

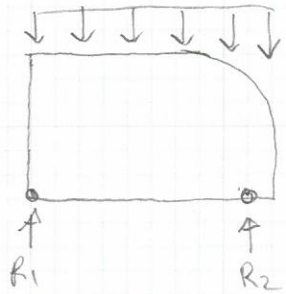


P = -19.2 psf.

V = P. A = (19.2)(25)
= 480 lbs.

$R_1 = R_2 = 480/2 = 240$ lbs

Case 2) SNOW LOAD



SL = 31 psf.

V = (SL)(A) = (31)(25)

V = 775 lbs

$R_1 = R_2 = 775/2 = 388$ lbs

LOAD COMBINATIONS (ASD) ASSUMED CANOPY DEAD LOAD = 250 lbs

D + 0.75 S = 250 + 0.75(775) = 832 lbs
D + W 250 + 480 = 730 lbs
D + 0.75 W + 0.75 S 250 + 0.75(480) + 0.75(775) = 1192 lbs

$R_1 = R_2 = 1192 \text{ lb}/2 = 596$ lbs

# of screws req'd at plate connection & AWNING TIE BACK

USE #12 SCREW - ASSUME 33 MIL STEEL PLATE

#12 SHEAR CAPACITY = 188 lbs (SSMA P. 58)

596/188 ≈ 4 SCREWS

SCREW CALCS FOR SHEAR RESISTANCE AT EXISTING WOOD TRANSOM

Shear = (596 lb)/(5 ft)(12 in/ft) = 10 lb/in

at 6" SPACING V = 10.0 lb/in x 6" = 60 lbs

#12 SCREW CAPACITY = 165 lbs (NDS Table 11 M)

60 < 165 lb O.K