UNIVERSITY OF MARYLAND

U.S. DEPARTMENT OF ENERGY SOLAR DECATHLON 2017

reACT

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U.S. DEPARTMENT OF ENERGY SOLAR DECATHLON 2017

STRUCTURAL DESIGN CALCULATIONS

February 23, 2017

Design Load Assumptions:

(from Solar Decathlon Building Code)
Design Wind Speed: 115 mph
Wind Exposure: C
Site Soil Class: D
Seismic Site Class: B
Interior Floor Live Load: 50 psf
Exterior Deck Live Load: 100 psf
Roof Live Load: 30 psf
Ground Snow Load: 35 psf

Dead Loads Calculation:

(calculated for roof panel thickness of 12")
SIP Panel = 9 psf
PV Panels = 5 psf
Misc = 1 psf
Total Dead Load = 15 psf

Snow Load Calculation

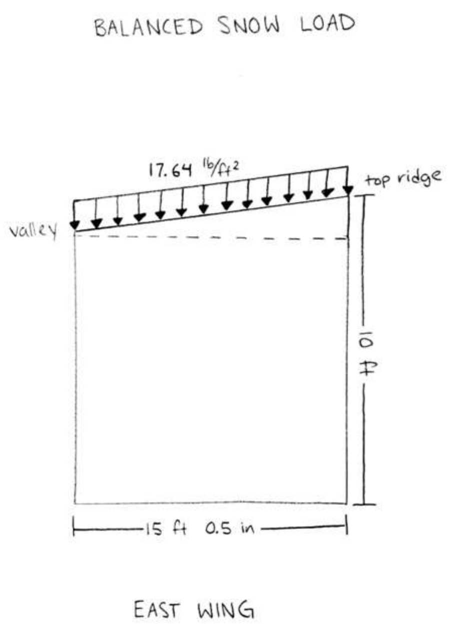
Live Loads:
$P_{interior}$ = **50** lb/ft^2
$P_{exterior}$ = **100** lb/ft^2
P_{roof} = **30** lb/ft^2

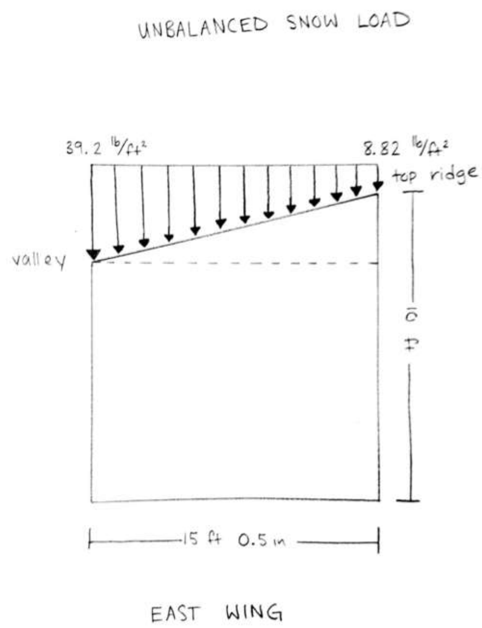
Snow Loads:
Risk Category: 1
$I_s = 0.8$
$C_e = 0.9$
$C_t = 1.0$
$p_g = 35$ lb/ft^2

Balanced Snow Load:
$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g =$ **17.64** lb/ft^2

Unbalanced Snow Load (valleys):
$p_{u\text{-}valley} = (2 \times p_f) / C_e =$ **39.20** lb/ft^2

Unbalanced Snow Load (top ridge):
$p_{u\text{-}top} = p_f / 2 =$ **8.82** lb/ft^2





Wind Loads Calculation:

$P_{net} = 0.00256 * V^2 * C_{net} * K_z * K_{zt}$

Wind Pressure (walls):
-assume positive inside pressure
V = 115 mph
$K_z = 0.9$ (from ASCE 7-10 Table 27.3-1)
$K_{zt} = 1$ (from ASCE 7-10 Table 26.8-1)
C_{net} winward = 0.43 (from IBC 2015 Table 1609.6.2)
C_{net} leeward = -0.51 (from IBC 2015 Table 1609.6.2)
C_{net} side = -0.66 (from IBC 2015 Table 1609.6.2)

Wind Pressure - P_{net} on walls:
Windward P_{net} = **13.10** lb/ft^2
Leeward P_{net} = **-15.54** lb/ft^2
Side P_{net} = **-20.11** lb/ft^2

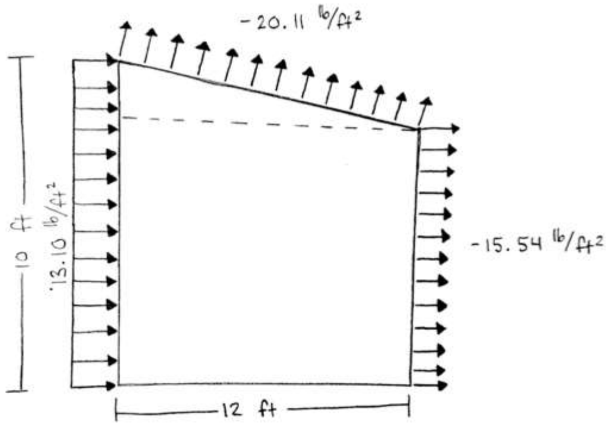
Wind Pressure - P_{net} on roof:
-sloped roof <10 degrees
-used highest C_{net} for calculations
V = 115 mph
K_z = 0.9 (from ASCE 7-10 Table 27.3-1)
K_{zt} = 1 (from ASCE 7-10 Table 26.8-1)
C_{net} windward = -1.09 (from IBC 2015 Table 1609.6.2)
C_{net} leeward = -0.66 (from IBC 2015 Table 1609.6.2)
C_{net} parallel = -1.09 (from IBC 2015 Table 1609.6.2)

Windward P_{net} = **-33.21** lb/ft^2
Leeward P_{net} = **-20.11** lb/ft^2
Side P_{net} = **-33.21** lb/ft^2

Our supposition is that because the exposed area of each wing of the house is greatest on the East and West faces, and because the shear walls that must resist those forces are shortest in this same direction, that the wind forces in the East-West direction will control.

<u>Wind force on East Wing Module (wind from East):</u>
East wall of East module consists of walls W4E1-7. West wall of East module consists of walls W4W1-2.
H_{E_East} = 10 ft *(Height of East wall of East module, W4E1-7)*
L_{E_East} = 29.75 ft *(Length of East wall of East module)*
A_{E_East} = 10*29.75 = 297.5 ft^2 *(Area of East wall of East module)*
$F_{E\_win} = A_{E\_East}*P_{net\_win}$ = 297.5*13.10 = 3897 lb *(Winward force on East module)*
H_{E_West} = 7.583 ft
L_{E_West} = 13.54 ft
A_{E_West} = 13.542*7.583 = 102.7 ft^2
$F_{E\_lee} = A_{E\_West}*P_{net\_lee}$ = 102.7*(-15.54) = -1596 lb *(Leeward force on East module)*
$F_{East} = F_{E\_win} - F_{E\_lee}$ = 5493 lb *(Total wind force from Eastern direction)*
$F_{E\_bot} = F_{E\_top} = F_{East}/2$ = 2746.5 lb *(Force into bottom and top of E module from E wind)*

WIND FROM WEST



WEST WING

<u>Wind force on West Wing Module (wind from West):</u>

West wall consists of walls W1W1-7. East wall consists of walls W1E2, W1E3, W1E5.

The West (winward) wall is the same height and length of the East module's East wall. The East (leeward) wall is the same height and length of the East module's West wall.

Therefore $F_{West} = F_{East}$

$\mathbf{F_{W\_top} = F_{W\_bot} = F_{E\_bot} = 2746.5\ lb}$

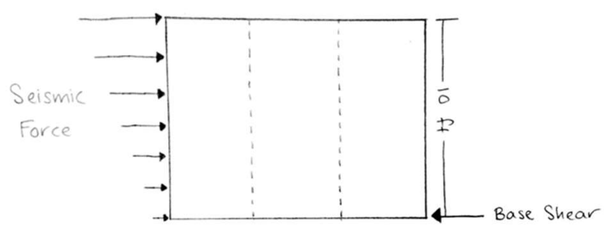
Seismic Loads:

Total mass of the top half of structure (transferring seismic load to the roof diaphragm)
W = **6104** lb calculated for top half of West module

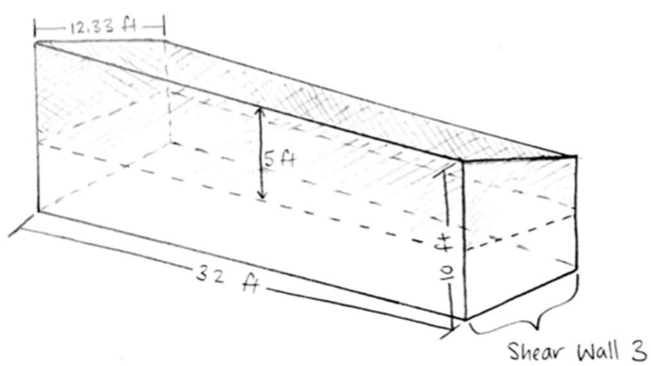
Coefficients:
Risk Category: II
$C_{vx} = 0.946$
$R = 6.5$
$I_e = 1.00$
$S_s = 17.5\% = 0.175$
$F_a = 1.00$

$S_{MS} = F_a S_s = 0.175$
$S_{DS} = (⅔)S_{MS} = 0.117$
$C_s = S_{DS} / (R / I_e) = 0.018$

$V = C_s W / 2 =$ **54.94** lb transferred to each shear wall



Shear Wall 3



WEST WING

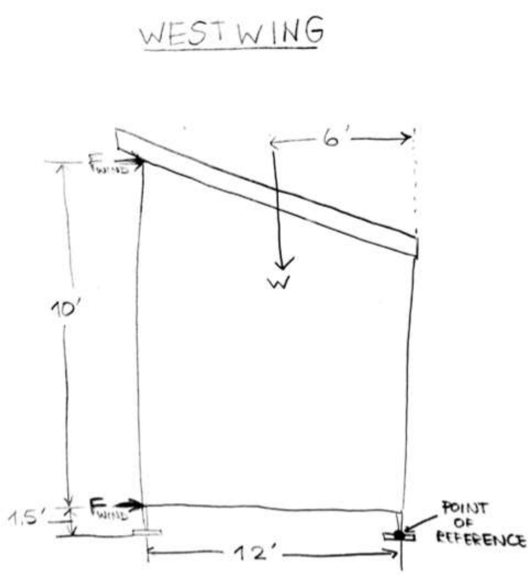
$F_x = C_{vx}V = (0.946)(54.63) =$ **51.68** lb
(this is significantly less than the shear forces calculated for wind, so *<u>wind loads will govern in the following calculations</u>*)

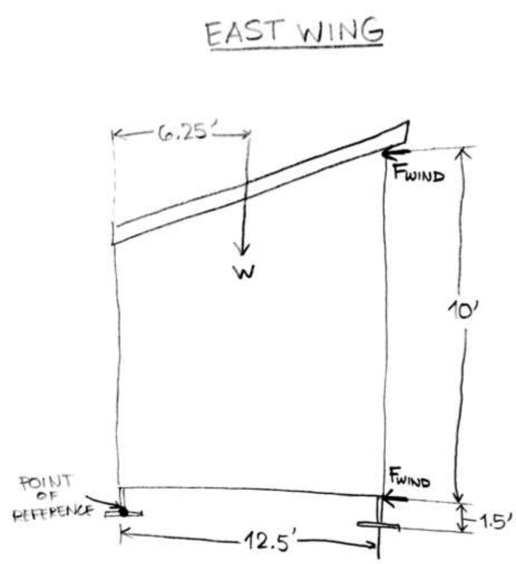
Overturning

<u>East Wing</u>
Weight = 10349.3 lb *(Total combined weight of roof, wall, and floor panels of E module)*
L_c =6.25 ft *(Distance from E or W wall to center of module)*
H_j = 1.5 ft *(Height from bottom of jacks to bottom of the floor)*
$\alpha = Weight*L_c/(F_{E\_bot}*H_j + F_{E\_top}*(H_j+H_{E\_East})) = 10349.3*6.25/(2746.5*1.5+2746.5*11.5) =$ **1.81**
α = *(Ratio of weight to wind force. Essentially the safety factor)*
Since 1.81 >> 1, there will not be overturning from wind from East direction.

<u>West Wing</u>
Weight =10136.6 lb *(Total combined weight of roof, wall, and floor panels of W module)*
L_c = 6 ft
H_j = 1.5 ft
$\beta = 10136.6*6/(2746.5*1.5+2746.5*11.5) =$ **1.70** *(Weight/wind force ratio.)*
Since 1.7>>1, there will not be overturning from wind from West direction.

Wind from North and South directions not considered because the wind force on each module from those directions is substantially less than those from the East and West.





Gravity Loads on Panels

I. Allowable Loads for SIP Panels, beams, columns, and posts

Combined (factored) Load (LRFD):
$W_u = 1.2DL + 1.6(SL \text{ or } L_{roof})$

Roof Panels: INSULSPAN 12 ¼ ‘’ thick (L/360); 12 ft span (for inner East wing, panel R4A7)
Combined Load (LRFD), (using unbalanced snow load for valleys to be conservative):
$W_u = 1.2DL + 1.6(SL \text{ or } L_{roof}) = 1.2(15) + 1.6(39.2) =$ **80.72** psf
W_{allow} = **142** psf *(according to Insulspan Out of Plane Load Tables below)*
$W_{allow} > W_u \rightarrow 142 > 80.72$
$SF = W_{allow} / W_u = 142 / 80.72 =$ **1.76**
Therefore, span appears feasible for general loading.
(This is conservative for West wing - 11 ft span)

Table R-3-DL ROOF AND FLOOR TRANSVERSE DESIGN LOAD (psf)

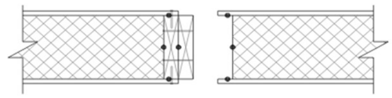
DOUBLE 2 x LUMBER SPLINES @ 4'-0" On Center																			
Thickness		Allowable Deflection	PANEL SPAN (feet)																
SIP	EPS		4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
4 1/2"	3 5/8"	L/360	162	115	68	54	40	33	26	22	18	15	13	–	–	–	–	–	–
		L/240	**195**	147	100	79	59	49	39	33	27	23	20	–	–	–	–	–	–
		L/180	**195**	**162**	**129**	103	78	65	52	44	36	31	26	–	–	–	–	–	–
6 1/2"	5 5/8"	L/360	246	200	155	119	84	69	55	46	38	32	27	24	21	18	16	14	13
		L/240	**248**	**210**	**173**	**148**	**124**	103	82	69	57	49	41	36	31	27	24	21	19
		L/180	**248**	**210**	**173**	**148**	**124**	**111**	**99**	**87**	**74**	63	52	47	41	36	32	29	26
8 1/4"	7 3/8"	L/360	**267**	**228**	**190**	166	142	115	89	75	62	53	45	39	34	30	26	23	21
		L/240	**267**	**228**	**190**	**169**	**148**	**129**	**111**	**100**	**90**	78	66	57	49	44	39	35	31
		L/180	**267**	**228**	**190**	**169**	**148**	**129**	**111**	**100**	**90**	**82**	**75**	**69**	**63**	57	51	46	41
10 1/4"	9 3/8"	L/360	**295**	**245**	**196**	**190**	**185**	**160**	**136**	116	97	83	70	61	53	47	41	37	33
		L/240	**295**	**245**	**196**	**190**	**185**	**160**	**136**	**120**	**105**	**96**	**88**	**81**	**75**	**69**	**64**	56	48
		L/180	**295**	**245**	**196**	**190**	**185**	**160**	**136**	**120**	**105**	**96**	**88**	**81**	**75**	**69**	**64**	**59**	**55**
12 1/4"	11 3/8"	L/360	**322**	**268**	**215**	**202**	**190**	**175**	**161**	**142**	**123**	**111**	**99**	88	78	69	61	54	48
		L/240	**322**	**268**	**215**	**202**	**190**	**175**	**161**	**142**	**123**	**111**	**99**	**91**	**84**	**78**	**72**	**67**	**63**
		L/180	**322**	**268**	**215**	**202**	**190**	**175**	**161**	**142**	**123**	**111**	**99**	**91**	**84**	**78**	**72**	**67**	**63**

Revision : January 20, 2014

Wall Panels: INSULSPAN 8 ¼ ‘’ thick; height of 10 ft (for inner East wing, panel W4W2, supporting roof panel R4A7)
$W_u = W_u \times (\text{panel length} / 2) = 80.72 \times (15.04 / 2) =$ **607** lb/ft
W_{allow} = **2720** lb/ft *(from Insulspan axial load table below)*
$W_{allow} > W_u \rightarrow 2672 > 607$
$SF = W_{allow} / W_u = 2720 / 607 =$ **4.48**
Therefore, wall appears feasible for general loading.

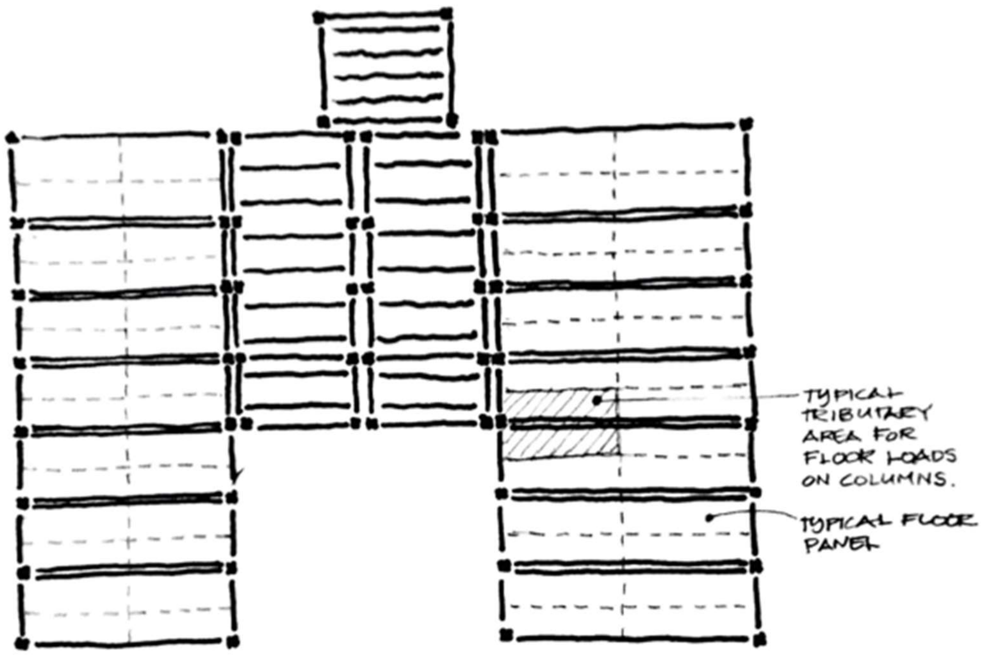
(This is conservative for outer wall panels, which will support smaller snow load)

Table W-2-DLVL WALL PANEL DESIGN LOAD

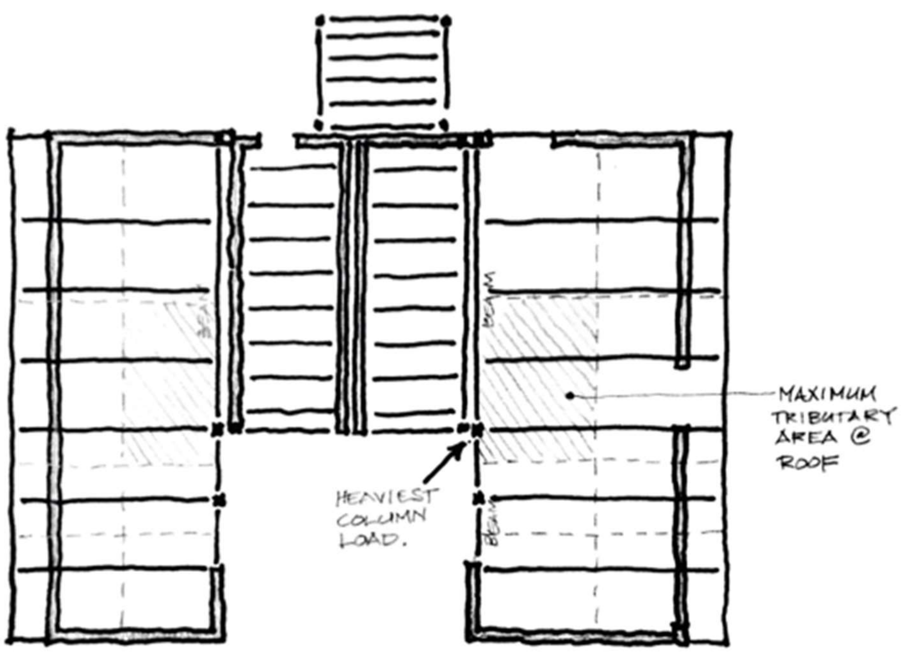


DOUBLE LVL LUMBER SPLINE @ 4'-0" On Center															
Thickness		**Allowable**	**PANEL SPAN (feet)**												
SIP	**EPS**	**Deflection**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**
ALLOWABLE WIND LOAD (psf) - END SUPPORT															
4 1/2"	3 5/8"	L/360	45	36	28	23	19	16	14	–	–	–	–	–	–
		L/240	**46**	**41**	**37**	**33**	29	25	21	–	–	–	–	–	–
		L/180	**46**	**41**	**37**	**33**	**31**	**28**	26	–	–	–	–	–	–
6 1/2"	5 5/8"	L/360	**47**	**42**	**38**	**34**	**31**	**29**	**27**	**25**	23	20	18	16	14
		L/240	**47**	**42**	**38**	**34**	**31**	**29**	**27**	**25**	**24**	**22**	**21**	**20**	**19**
		L/180	**47**	**42**	**38**	**34**	**31**	**29**	**27**	**25**	**24**	**22**	**21**	**20**	**19**
8 1/4"	7 3/8"	L/360	**48**	**43**	**38**	**35**	**32**	**30**	**27**	**26**	**24**	**23**	**21**	**20**	**19**
		L/240	**48**	**43**	**38**	**35**	**32**	**30**	**27**	**26**	**24**	**23**	**21**	**20**	**19**
		L/180	**48**	**43**	**38**	**35**	**32**	**30**	**27**	**26**	**24**	**23**	**21**	**20**	**19**
10 1/4"	9 3/8"	L/360	**49**	**44**	**39**	**36**	**33**	**30**	**28**	**26**	**25**	**23**	**22**	**21**	**20**
		L/240	**49**	**44**	**39**	**36**	**33**	**30**	**28**	**26**	**25**	**23**	**22**	**21**	**20**
		L/180	**49**	**44**	**39**	**36**	**33**	**30**	**28**	**26**	**25**	**23**	**22**	**21**	**20**
ALLOWABLE WIND LOAD (psf) - FACE SUPPORT OR MODIFIED END SUPPORT															
4 1/2"	3 5/8"	L/360	45	36	28	23	19	16	14	–	–	–	–	–	–
		L/240	67	54	42	35	29	25	21	–	–	–	–	–	–
		L/180	88	72	56	47	39	33	28	–	–	–	–	–	–
6 1/2"	5 5/8"	L/360	104	84	65	54	43	37	31	27	23	20	18	16	14
		L/240	150	122	95	79	63	54	45	39	33	29	26	23	21
		L/180	**156**	140	124	103	82	70	58	51	44	39	34	30	27
8 1/4"	7 3/8"	L/360	**179**	144	110	92	75	64	53	46	39	34	30	26	23
		L/240	**179**	**165**	**152**	130	109	93	77	66	56	49	43	38	34
		L/180	**179**	**165**	**152**	**143**	**135**	117	100	86	73	64	56	50	44
10 1/4"	9 3/8"	L/360	**185**	**179**	**174**	148	122	104	87	75	64	56	49	43	38
		L/240	**185**	**179**	**174**	**164**	**154**	140	126	110	94	82	71	63	55
		L/180	**185**	**179**	**174**	**164**	**154**	**147**	**140**	131	122	107	92	82	72
ALLOWABLE AXIAL LOAD (plf)															
4 1/2"	3 5/8"		2865	2728	2592	2455	2318	2138	1957						
6 1/2"	5 5/8"		2762	2799	2835	2872	2908	2945	2982	3018	3055	3091	3128	3164	3201
8 1/4"	7 3/8"		2672	2696	2720	2745	2769	2793	2817	2841	2865	2890	2914	2938	2962
10 1/4"	9 3/8"		2672	2696	2720	2745	2769	2793	2817	2841	2865	2890	2914	2938	2866

Gravity Loads on Beams



TRIBUTARY AREAS ON FLOOR



ROOF TRIBUTARY AREAS

From the framing sketches shown above, it is evident that the greatest load experienced by beams is at the opening between kitchen and dining room in the East Wing (supporting roof panels R4A1-R4A4)

In accordance with the ANSI/AF&PA NDS-2015 using the LRFD method
Combined Load (LRFD), (using unbalanced snow load for valleys): 80.72 psf
$W_u = W_u$ x (panel length / 2) = 80.72 x (15.04 / 2) = **607** lb/ft
Beam length = 16.875 ft → assume 17 ft
Maximum bending moment:
$M_{max} = (W_u \times L^2) / 8 = (607 \times 17^2) / 8$ = 21,928 lb-ft = **263,136** lb-in

Beam Type (selected using LP SolidStart LVL Technical Guide 2900F_b - 2.0E):
Allowable Stress Design Values:
F_b = 2900 psi
F_v = 285 psi
F_c = 3200 psi
$F_{c\text{-}perp}$ = 750 psi
E = 2.0 x 10^6 psi

Member Details:
b = 5 ¼ ''
d = 14''
L = 17 ft
A = 14 x 5.25 = 73.5 in^2
I = (b)(d^3) / 12 = (5.25)(14^3) / 12 = 1200.5 in^4
Stress: $f_b = (y)(M_{max}) / I$ = (7)(263,136) / 1200.5 = 1535 psi

Adjustment Factors:
Φ_b = 0.85
K_{Fb} = 2.54
λ = 1.00
C_t = 1.00
C_{Fb} = 1.00
C_i = 1.00
C_L = 0.968

Design (allowed) Bending Stress:
$F_b' = F_b \times K_{Fb} \times \Phi_b \times \lambda \times C_t \times C_{Fb} \times C_i \times C_L$ = **6061** psi

Applied Bending Stress:
f_b = 1535 psi
f_b / F_b' = 0.253
Therefore, beam appears feasible for general loading.

Header Beam Design
(above door on East wing; supporting roof panel R4A4)
(In accordance with the ANSI/AF&PA NDS-2015 using the LRFD method)
Combined Load (LRFD), (using balanced snow load for top ridge of roof):
W_u = 1.2DL + 1.6(SL or L_{roof}) = 1.2(15) + 1.6(17.64) = **46.25** psf
W_u = W_u x (panel length / 2) = 46.25 x (15.04 / 2) = **347.8** lb/ft
Header length = 4.83 ft → assume 5 ft
M_{max} = (W_u x L^2) / 8 = (348 x 5^2) / 8 = 1088 lb-ft = **13,056** lb-in

Beam Type (selected using NDS 2015 Supplement):
Allowable Stress Design Values:
F_b = 2050 psi
F_v = 175 psi
F_c = 1800 psi
$F_{c\text{-}perp}$ = 480 psi
E = 1.6 x 10^6 psi

Member Details:
Southern Pine, non-dense select structural, 2''-4'' wide (other types of wood or lower grades may not be sufficient for the stresses calculated on this member - check before using something else)
b_{nom} = 2 ''
b = 1 ½ ''
d_{nom} = 8 ''
d = 7 ¼ ''
L = 5.7 ft
A = 1.5 x 7.25 = 10.875 in^2
I = (b)(d^3) / 12 = (1.5)(7.25^3) / 12 = 47.63 in^4
Stress: f_b = (y)(M_{max}) / I = (3.625)(13,056) / 47.63 = **994** psi

Adjustment Factors:
Φ_b = 0.85
K_{Fb} = 2.54
λ = 1.00
C_t = 1.00
C_{Fb} = 1.2
C_i = 1.00
C_L = 1.00

Design Bending Stress:
F_b' = F_b x K_{Fb} x Φ_b x λ x C_t x C_{Fb} x C_i x C_L = **5311.14** psi

Applied Bending Stress:
f_b = 994 psi

f_b / F_b' = **0.187**
Therefore, header appears feasible for general loading.

<u>Header Design</u>
(above window on East wing; roof panel R4A7)
Combined Load (LRFD):
(using balanced snow load for top ridge of roof)
W_u = 1.2DL + 1.6(SL or L_{roof}) = 1.2(15) + 1.6(17.64) = **46.25** psf
W_u = W_u x (panel length / 2) = 46.25 x (15.04 / 2) = **347.8** lb/ft
Header length = 5.625 ft → assume 5.7 ft
M_{max} = (W_u x L^2) / 8 = (348 x 5.7^2) / 8 = 1413.32 lb-ft = **17,000** lb-in

Beam Type (selected using NDS 2015 Supplement):
Allowable Stress Design Values:
F_b = 2050 psi
F_v = 175 psi
F_c = 1800 psi
$F_{c\text{-}perp}$ = 480 psi
E = 1.6 x 10^6 psi

Member Details:
Southern Pine, non-dense select structural, 2''-4'' wide (other types of wood or lower grades may not be sufficient for the stresses calculated on this member - check before using something else)
b_{nom} = 2 ''
b = 1 ½ ''
d_{nom} = 8 ''
d = 7 ¼ ''
L = 5.7 ft
A = 1.5 x 7.25 = 10.875 in^2
I = (b)(d^3) / 12 = (1.5)(7.25^3) / 12 = 47.63 in^4
Stress: f_b = (y)(M_{max}) / I = (3.625)(17,000) / 47.63 = **1293.83** psi

Adjustment Factors:
Φ_b = 0.85
K_{Fb} = 2.54
λ = 1.00
C_t = 1.00
C_{Fb} = 1.2
C_i = 1.00
C_L = 0.7

Design Bending Stress:
$F_b' = F_b \times K_{Fb} \times \Phi_b \times \lambda \times C_t \times C_{Fb} \times C_i \times C_L =$ **3717.8** psi

Applied Bending Stress:
$f_b = 1293.83$ psi
$f_b / F_b' = 0.348$
Therefore, header appears feasible for general loading.

<u>Header Design</u>:
(above door on East side of courtyard, wall panel W4W1; roof panels R4A5, R4A6)
Combined Load (LRFD):
(using unbalanced snow load for valleys)
$W_u = 1.2DL + 1.6(SL \text{ or } L_{roof}) = 1.2(15) + 1.6(39.2) =$ **80.72** psf
$W_u = W_u \times (\text{panel length} / 2) = 80.72 \times (15.04 / 2) =$ **607** lb/ft
Header length = 8 ft
$M_{max} = (W_u \times L^2) / 8 = (607 \times 8^2) / 8 = 4856$ lb-ft = **58,272** lb-in

Beam Type (selected using LP SolidStart LVL Technical Guide 2900F_b - 2.0E):
Allowable Stress Design Values:
$F_b = 2900$ psi
$F_v = 285$ psi
$F_c = 3200$ psi
$F_{c\text{-}perp} = 750$ psi
$E = 2.0 \times 10^6$ psi

Member Details:
b = 3 ½ ‘’
d = 7 ¼ ’’
L = 8 ft
$A = 7.25 \times 3.5 = 25.38 \text{ in}^2$
$I = (b)(d^3) / 12 = (3.5)(7.25^3) / 12 = 111.15 \text{ in}^4$
Stress: $f_b = (y)(M_{max}) / I = (3.625)(58,272) / 111.15 =$ **1900.46** psi

Adjustment Factors:
$\Phi_b = 0.85$
$K_{Fb} = 2.54$
$\lambda = 1.00$
$C_t = 1.00$
$C_{Fb} = 1.2$
$C_i = 1.00$
$C_L = 0.98$

Design Bending Stress:
$F_b' = F_b \times K_{Fb} \times \Phi_b \times \lambda \times C_t \times C_{Fb} \times C_i \times C_L =$ **7363.05** psi

Applied Bending Stress:
f_b = 1900.46 psi
f_b / F_b' = 0.258
Therefore, header appears feasible for general loading.

Gravity Loads on Columns

Based on the sketches of tributary are above, it is evident that the heaviest loaded column will be that supporting the beam over the kitchen-dining room opening located at bottom left corner of dining area, supporting 16 ft beam)
Load on column (calculated by halving total load on beam):
P = (607 lb/ft) x (17 ft / 2) = **5160** lb

Column Type:
Allowable Stress Design Values:
F_b = 1752 psi
F_v = 160 psi
F_c = 1750 psi
$F_{c\text{-}perp}$ = 215 psi
E = 1.494 x 10^6 psi
E_{min} = 759,350 psi

Member Details:
b = 3 ½ ''
d = 7 ¼ ''
h = 8 ft
A = 7.25 x 3.5 = 25.38 in^2
$I_x = (b)(d^3) / 12 = (3.5)(7.25^3) / 12 = 111.15\ in^4$
$I_y = (d)(b^3) / 12 = (7.25)(3.5^3) / 12 = 25.9\ in^4$
Compressive Stress: f_c = P / A = **203.31** psi

Adjustment Factors:
C_{Mc} = 0.80
C_t = 1.00
C_{Fc} = 1.05
C_{Fb} = 1.3
C_i = 0.80
C_P = 0.94
K_{Fc} = 2.40
Φ_c = 0.9
Λ = 1.00

Strength in compression parallel to grain:
Design compressive stress:
$F_c' = F_c \times C_t \times C_{Fc} \times C_i \times C_P$ = **1382** psi
Applied compressive stress:
f_c = 203.31 psi
f_c / F_c' = 0.147
Therefore, column appears feasible for general loading.

Wood blocking under columns:
P = **5160** lb
(this is from the column load calculated above; the column at the corner of floor panel F4A4 is supporting half the load from the 16 ft beam, so this should be conservative for other wood blocking in the house)

Beam Type (selected using NDS 2015 Supplement):
Allowable Stress Design Values:
F_b = 2050 psi
F_v = 175 psi
F_c = 1800 psi
$F_{c\text{-}perp}$ = 480 psi
E = 1.6 x 10^6 psi

Member Details:
Southern Pine, non-dense select structural, 2''-4'' wide (other types of wood or lower grades may not be sufficient for the stresses calculated on this member - check before using something else)
b_{nom} = 4 ''
b = 3 ½ ''
d_{nom} = 8 ''
d = 7 ¼ ''
A = 3.5 x 7.25 = 25.375 in^2
I = $(b)(d^3) / 12 = (3.5)(7.25^3) / 12$ = 111.15 in^4
Stress: f_c = P / A = 5160 / 25.375 = **203.35** psi

Adjustment Factors:
C_{Mc} = 0.80
C_t = 1.00
C_{Fc} = 1.05
C_{Fb} = 1.3
C_i = 0.80
C_P = 0.94
K_{Fc} = 2.40
Φ_c = 0.9
Λ = 1.00

Strength in compression parallel to grain:
Design compressive stress:
F_c' = F_c x C_t x C_{Fc} x C_i x C_P = **1618.7** psi
Applied compressive stress:
f_c = 203.35 psi
f_c / F_c' = 0.126
Therefore, blocking appears feasible for general loading.

Wooden Foundation Posts:
Column: P_{column} = **5160** lb
Floor Panels: $P_{floor} = (½)(1.2DL + 1.6L_{floor}) = (½)[1.2(354.862)+1.6(2333.33)]$ = **2079.58** lb
(calculated using factored load of floor panels F4A4 and F4A5, post supports half of each panel)
Total Load: $P = P_{column} + P_{floor}$ = **7239.58** lb

Member Details:
Southern Pine, non-dense select structural, 2''-4'' wide (other types of wood or lower grades may not be sufficient for the stresses calculated on this member - check before using something else)
b_{nom} = 4 ''
b = 3 ½ ''
d_{nom} = 4 ''
d = 3 ½ ''
$A = 3.5 \times 3.5 = 12.25\ in^2$
$I = (b)(d^3) / 12 = (3.5)(3.5^3) / 12 = 12.51\ in^4$
Stress: $f_c = P / A = 7240 / 12.25$ = **591** psi

Allowable Stress Design Values:
F_b = 2050 psi
F_v = 175 psi
F_c = 1800 psi
$F_{c\text{-}perp}$ = 480 psi
$E = 1.6 \times 10^6$ psi

Adjustment Factors:
C_{Mc} = 0.80
C_t = 1.00
C_{Fc} = 1.15
C_{Fb} = 1.3
C_i = 0.80
C_P = 0.94
K_{Fc} = 2.40
Φ_c = 0.9
Λ = 1.00

Strength in compression parallel to grain:
Design compressive stress:
$F_c' = F_c \times C_t \times C_{Fc} \times C_i \times C_P$ = **1772.8** psi
Applied compressive stress:
f_c = 591 psi
f_c / F_c' = **0.333**
Therefore, post appears feasible for general loading.

Roof Diaphragm Loads

Assumptions

- Pressure forces towards interior of structure are assumed positive.
- Suction forces from pressure are assumed negative.
- Suction forces on roof are perpendicular to slope of roof.
- Lateral forces from suction of roof are F*sin(9 degrees).
- When wind forces come from North or South directions, roof suction force is out of plane and thus irrelevant.
- Largest shear force (psf) is assumed to be in West module on North and South shear walls (W1N1-3 & W1S1-3) when wind is blowing from E→ W. *Explanation: Wind pressure should be the same in all modules and directions, the largest exposed areas are on the E and W walls of the East and West wings, but the shear walls of the West Wing are slightly shorter than those in the East Wing.*

Wall to Roof Diaphragm - West Wing

Windward wall = wall G = panels W4E4, W1E5
Leeward wall = wall A = panels W1W1-7
Roof = leeward roof = panels R1A1-7

Windward p_{net} = 13.10 psf (see calc above)
Leeward p_{net} = -15.54 psf (see calc above)
Leeward p_{net} = -20.11 psf (see calc above)

$F_w = A_{wall} * p_{net\text{-}wall}$ *(wind load on wall surface)*
$F_R = (F_{w\_A} / 2) + (F_{w\_G} / 2) - (F_w * \sin(9\ deg))$ *(wind load on roof diaphragm)*
$F_F = (F_{w\_A} / 2) + (F_{w\_G} / 2)$ *(wind load on floor diaphragm)*

Half the force on the wall area above is assumed to be transferred to the roof diaphragm, and the other half will be transferred to the floor diaphragm.

Surface	Height (ft)	Length (ft)	Area (sf)	F_w (lb)
W wall of W wing	10	29.75	297.5	4623.15
S wall of W wing	7.583	12.875	97.64	1277.25
Roof W wing*	12.33	31.25	385.42	7750.73
Floor W wing*	10.33	29.75		

* Heights of Roof and floor are actually their 'widths' for this calculation

F_R = 4623.15/2 + 1277.25/2 - 7750sin(9) = 4202.09 lb (in W direction)
F_F = 4623.15/2 + 1277.25/2 = 3588.83 lb (in W direction)

$\boxtimes_R = F_R/L_R$ = 4202.09/31.25 = 134.45 psf *(distributed load on roof diaphragm)*

$\boxtimes_F = F_F/L_F$ = 3588.83/29.75 = 122.70 psf *(distributed load on floor diaphragm)*

Half of the roof diaphragm is transferred to the North shear wall and half to the South shear wall.

$M_{max} = \boxtimes_R * L_R^2/8 = 134.45*31.25^2/8$ = 16412.35 lb-ft *(Maximum moment in roof diaphragm)*

$C_u = M_{max}/d$ = 15311.28/10.33 = 1588.8 lb *(Maximum chord force in diaphragm)*

FASTENER REQUIREMENTS FOR DIAPHRAGM SHEAR

The length in which the Max Shear Force to the shear walls in the East-West Direction (V_N =2101.045) can be distributed will be the length of the walls as they will be fully connected to the roof diaphragm.

Therefore: $\omega_N = V_N/d$ = 2101.05/10.33 = 203.39 lb/ft *(Distributed shear force on N shear wall)*

The fastener will be a 3 in. long ¼ in. thick A36 steel angle bracket. The bolts are made of galvanized shear. Three aspects of the fastener can fail, the plate, the bolts, and the wood that the plate is fastened into.

The bolts can fail from shear force from suction on the roof and shear force from the roof.
The plate can fail in shear and from bending.
The wood can fail from tension, compression, and shear.

There are 4 fasteners on the North and South walls of the West modules (one at each intersection between wall panels).

Assumptions:

- The load is evenly distributed with a distributed shear force of 203.99 lb/ft or 16.95 lb/in.
- Each fastener is responsible for absorbing half the force between adjacent joints.
- The joint that experiences the largest load is located between walls W1N2 and W1N3
- This joint is responsible for absorbing 48 in. worth of shear force.

$V_{J\_max} = \omega_N$*48 in.* (1 ft/12 in) = 813.57 lb *(Maximum shear force at a joint)*

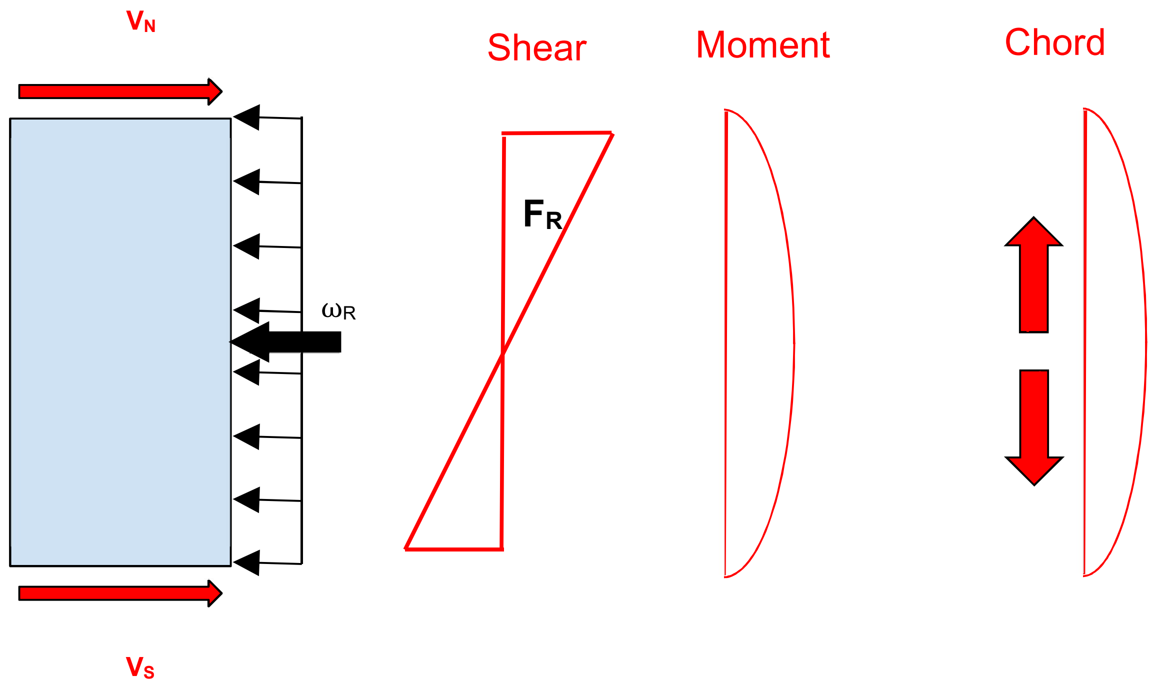


Plate:
Calc 1 - shear at intersection of plates in angle bracket.

Member details of one plate	
Thickness	t = **0.25** in.
Length	L = **8** in.
Depth	d = **3** in.
Area of intersection	$A_i = t \times d$ = **0.75** in^2
Yield strength of A36 steel	σ_y = **36000** psi
Min. ultimate strength of A36 steel	σ_u = **58000** psi
Shear strength A36 steel	$\tau = \sigma_u$*0.6 =**34800** psi
Distributed shear force at intersection	$\tau_I = V_{J\_max}/A$ = **1084.76** psi

$\tau >> \tau_I$ ***therefore the intersection of the angle bracket will not fail from shear***

DIAPHRAGM CHORD DESIGN

Panel edge has a 2x8 serving as a diaphragm chord. However, at the top of every panel joint the chord has a cutout. At these joints the chord is essentially a 2x4.

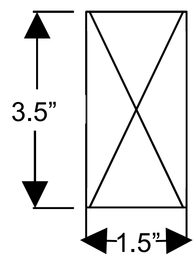
STRUCTURAL WOOD BEAM DESIGN (NDS) - COMPRESSION

In accordance with the ANSI/AF&PA NDS-2012 using the LRFD method

Analysis results

Design axial compression (2x4)

P = 1588



Sawn lumber section details

Nominal breadth of sections	
Nominal breadth of sections	b_{nom} = **2** in
Dressed breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **4** in
Dressed depth of sections	d = **3.5** in
Number of sections in member	N = **1**
Overall breadth of member	b_b = N x b = **1.5** in
Species, grade and size classification assumption)	Hem-Fir, No. 2 grade, 2” & wider (this is an
Bending parallel to grain	F_b = **850** psi
Tension parallel to grain	F_t = **525** psi
Compression parallel to grain	F_c = **1300** psi
Compression perpendicular to grain	F_{c_perp} = **405** psi
Shear parallel to grain	F_v = **150** psi
Modulus of elasticity	E = $\mathbf{1.3*10^6}$ psi
Modulus of elasticity, stability calculations	E_{min} = $\mathbf{4.7*10^5}$ psi
Mean shear modulus	G_{def} = E/16 = **81250** psi

Member details

Service condition	**Dry**
Unbraced length in y-axis	L_y = **4** ft
Effective length factor in y-axis	K_y = **1**
Effective length in y-axis	L_{ey}= L_y x K_y = **4** ft

Section properties

Cross sectional area of member $A = N \times b \times d =$ **5.25** in^2

Adjustment factors

Resistance factor for compression - Table 2.3.6 $\phi_c =$ **0.90**

Resistance factor for modulus of elasticity - Table 2.3.6

$\phi_s =$**0.85**

Format conversion factor for compression - Table 2.3.5

$K_{Fc} =$ **2.40**

Format conversion factor for modulus of elasticity - Table 2.3.5

$K_{FE} =$ **1.76**

Time effect factor - Table N.3 $\lambda =$ **1.00**

Wet service factor for compression - Table 4A $C_{Mc} =$ **1.00**

Temperature factor - Table 2.3.3 $C_t =$ **1.00**

Size factor for compression $C_{Fc} =$ **1.15**

Incising factor for modulus of elasticity - Table 4.3.8

$C_{iE} =$ **1.00**

Incising factor for bending, shear, tension & compression - Table 4.3.8

$C_i =$ **1.00**

$C_{ME} =$ **1.00**

Adjusted modulus of elasticity for column stability

$E_{min}' = E_{min} \times K_{FE} \times \phi_s \times C_{ME} \times C_t \times C_{iE} =$ **703120** psi

Reference compression design value $F_c^* = F_c \times K_{Fc} \times \phi_c \times \lambda \times C_{Mc} \times C_t \times C_{Fc} \times C_i =$ **3229** lb

Critical buckling design value or compression $F_{cE} = 0.822 \times E_{min}' / (L_{ey} / b)^2 =$ **564** psi

$c =$ **0.80** *(because sawn lumber)*

Column stability factor - eq.3.7-1 $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{}[((1 + (F_{cE} / F_c^*))$

/ (2

$\times c))^2 - (F_{cE} / F_c^*) / c] =$ **0.168**

Beam stability factor - cl.3.3.3 $C_L =$ **1.00**

Strength in compression parallel to grain - cl.3.6.3

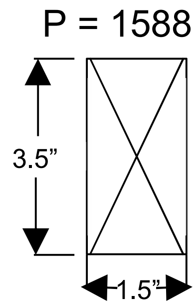
Design compressive stress $F_c' = F_c \times K_{Fc} \times \phi_c \times \lambda \times C_t \times C_{Fc} \times C_i \times C_p =$ **711.5** psi

Applied compressive stress $f_c = P / A =$ **302.5** psi

$f_c / F_c' =$ **0.425**

<u>STRUCTURAL WOOD BEAM DESIGN (NDS) - TENSION</u>
In accordance with the ANSI/AF&PA NDS-2012 using the LFRD method

Analysis results
Design axial tension



Sawn lumber section details
Nominal breadth of sections b_{nom} = **2** in
Dressed breadth of sections b = **1.5** in
Nominal depth of sections d_{nom} = **4** in
Dressed depth of sections d = **3.5** in
Number of sections in member N = **1**
Overall breadth of member b_b = N x b = **1.5** in
Species, grade and size classification Hem-Fir, No. 2 grade, 2” & wider (this is an assumption)
Bending parallel to grain F_b = **850** psi
Tension parallel to grain F_t = **525** psi
Compression parallel to grain F_c = **1300** psi
Compression perpendicular to grain F_{c_perp} = **405** psi
Shear parallel to grain F_v = **150** psi
Modulus of elasticity E = $\mathbf{1.3*10^6}$ psi
Modulus of elasticity, stability calculations E_{min} = $\mathbf{4.7*10^5}$ psi
Mean shear modulus G_{def} = E/16 = **81250** psi

Member details
Service condition **Dry**

Section properties
Cross sectional area of member A = N x b x d = **5.25** in^2

Adjustment factors
Resistance factor for tension - Table 2.3.6 ϕ_t = **0.80**
Resistance factor for modulus of elasticity - Table 2.3.6
ϕ_s =**0.85**
Format conversion factor for tension - Table 2.3.5
K_{Ft} = **2.70**

Format conversion factor for modulus of elasticity - Table 2.3.5
K_{FE} = **1.76**
Time effect factor - Table N.3 λ = **1.00**
Temperature factor - Table 2.3.3 C_t = **1.00**
Size factor for tension C_{Ft} = **1.5**
Incising factor for modulus of elasticity - Table 4.3.8
C_{iE} = **1.00**
Incising factor for bending, shear, tension & compresion - Table 4.3.8
C_i = **1.00**

Tension parallel to grain - cl.3.8.1
Design tensile stress $F_t' = F_t \times K_{Ft} \times \phi_t \times \lambda \times C_t \times C_{Ft} \times C_i$ = **1701** psi
Applied tensile stress $f_t = P / A$ = **302.5** psi
f_t / F_t' = **0.178**

<u>FAILURES IN FLOOR CONNECTIONS</u>

Withdrawal of bolts:
Wind load into the roof can be the cause of withdrawal in bolts connecting the walls to the floor.

West Module:
W = 7607 lb *(Combined weight of roof and wall dead load)*
F_{top} = 2746.5 lb *(Wind force into top of diaphragm system)*
L_c = 6 ft *(Distance from wall to center of floor)*
H = 10 ft *(Height of W wall of W module)*
$\alpha = W*L_c / (H*F_{top})$ = 7607*6/(2746.5*10) = **1.66**
1.66 >> 1, therefore floor connection in West module will not fail from bolt withdrawal.

East Module:
W = 7787 lb *(Combined weight of roof and wall dead load)*
F_{top} =2746.5
L_c = 6.25 ft
H = 10 ft
β = 7787*6.25/(2746.5*10) = **1.77**
1.77 >> 1, therefore floor connection in East module will not fail from bolt withdrawal.

Floor Shear (welds):

Assumptions:
- ⅛” thick plates made of A36 steel → a_{max} = 6 mm = 0.236 in
- Fillet welds (fail in shear)
- 2” is smallest weld length
- E70 filler material (F_y = 58 ksi)

Used in calculations:

- Eq. J2.4 in AISC
- Max Base Shear is from Eastern wind on Shear Wall 3 (W1S1-W1S3) = 5814.5 lb
- Max shear on single floor connection = 5814.5/4 = 1453.6 lb

Calc:

L_w = 2 in. *(Length of weld)*
F_{EXX} = 58 ksi *(Strength of E70 filler)*
F_{EXX_S} = 0.6*58 = 34.8 ksi *(Strength of E70 filler in shear)*
a_w = 0.236 in. *(Height of weld)*
Φ = 0.75
$P_u = L_w*\Phi*F_{EXX\_S}*\cos45*a_w$ = 2*0.75*34.8*0.707*0.236 = 8710 lb *(Ultimate Weld Strength)*
8710>>1453.6, therefore weld will not fail from base shear.

Bolt Shear:

Max shear is on shear wall 3, walls W1S1-W1S3.
Situation I_S is most likely failure scenario.
N = 4 *(Number of bolts per plate)*
D = ⅜ in. *(Diameter of bolt)*
L = 2 in. *(Length of bolt)*
V_{max} = 1453.6 *(Max shear on a connection)*
$V_{b\_max} = V_{max}/N$ = 1453.6/4 = 363.4 lb *(Max shear on single bolt)*
F_{es} = 2950 psi *(from Table 12.3.3 of NDS)*
$R_d = 4K_\Theta = 4*(1+0.25(\Theta/90)) = 4*(1.25) = 5$ *(from Table 12.3.1B of NDS)*
$f_v' = D*L*F_{es}/R_d$ = ⅜*2*2950/5 = 442.5 *(Design load)*
$SF = f_v'/f_v$ = 442.5/363.4 = 1.22 *(Safety factor)*
1.22>1, therefore bolts will not fail in shear.

<u>ROOF FASTENER FAILURE</u>

Withdrawal:

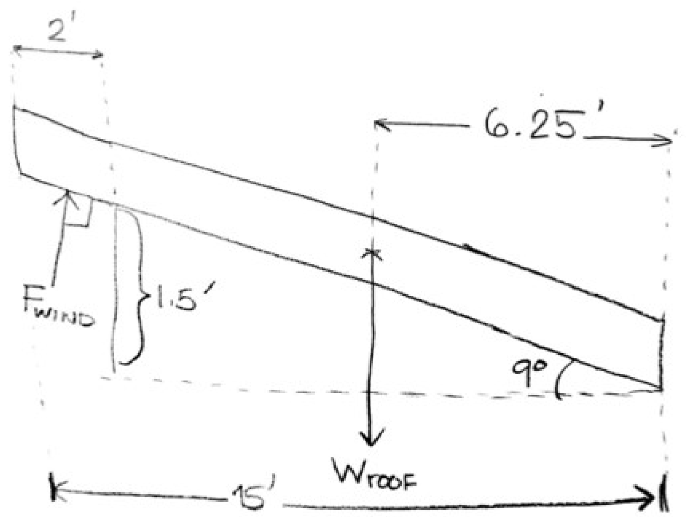
Assumptions:

- Largest potential for failure is on the roof of the east wing since it has the largest area of any roof.
- Fastener between roof panels 1 & 2 and panels 6 & 7 have largest tributary areas, meaning they are most likely to fail from withdrawal.
- Largest suction force on roof happens when wind is coming from West direction because it makes the roof winward, giving the roof a higher P_{net} value (P_{net_w} = -33.21) than the leeward direction (P_{net_L} = -20.11).

A = 475.26 ft^2 *(Area of roof)*
L = 31.25 ft *(Length of roof)*
$w_{1/2} = w_{6/7}$ = 0.5*4 + 0.5(5+7.5/12) = 4.8125 ft *(width of fastener tributary area)*
$F_R = A*P_{net\_w}$ = 475.26*39.24 = 15784.68 (perpendicular to roof) *(Suction force on roof)*
$\omega = F_R/L$ = 15784.68/31.25 = 505.11 lb/ft *(suction force per linear ft)*

F_B = 505.11*4.8125 = 2430.84 lb *(Upward force on bolts)*
Since F_B is less than the weight of the roof, the bolts will not fail from withdrawal.

Allowable tensile stress in bolts
W_R = 900 + 5229.33 = 6129.33
S_t = 70 ksi *(conservative estimation of strength of bolts)*
D = ⅜ in *(diameter of bolts)*
A_s = 0.106 in^2 *(Tensile stress area according to Fastenel)*
P_{allow} = S_t * (A*4) = 29680 lb *(Allowable tensile force in bolt according to Fastenel)*
F_{B_S} = 15567.2-6129.33 = 9437.87 lb *(Suctional force on bolts from roof)*
9437.89 << 29680, therefore the bolts will not fail due to tension from suctional roof force.



Gravity Loads on Decking

Based on a survey of the decking tributary areas, it was determined that the greatest loads on support beams and columns would occur in the middle of the Rear-East (NE) deck.

Governing loads:

- Column: 5 (between A and B in drawing S-102
- Beam: 5 (facing 4.1, top beam)

DL = 4.625 psf *(Dead load per sq. ft.)*
LL = 100 psf *(Live load per sq. ft.)*
CL = 1.2DL + 1.6LL = 165.55 *(Combined load)*

Column:
$A_C = (⅛(95*108+108*77) + ¼(108*70))/144\ in^2 = 29.25\ ft^2$ *(Tributary area of column 5)*
$L_C = A_C*CL = 29.25*165.55 = 4842.3$ lb *(Load on column 5)*
$A_B = ½*108*½*95/144\ in^2 = 17.82\ ft^2$ *(Tributary area of beam)*
$L_B = A_B*CL = 17.82*165.55 = 2950.1$ lb *(Load on beam)*
$\omega = 2950.1 / 4.5 = 655.6$ lb/ft *(Load per linear foot on beam)*

Tension on Deck Beams:
L = 4.5 ft. *(Length of beam)*
$M = \omega L^2/8 = 655.6*4.5^2/8 = 1659.5$ lb*ft = 19914 lb*in *(Moment equation)*
$I = 1/12*b*h^3 = 1/12*(1.5")(9.25")^3 = 98.938\ in^4$ *(Moment of Inertia)*
c = 9.25/2 = 4.625 in. *(Centroid)*
$\sigma = Mc/I = 19914*4.625/98.932 = 930.96$ psi *(Tensile stress equation)*

Foundation Footing Design

As described above, the greatest single load on any footing will be ***7240 lb*** at the column supporting the S end of the beam spanning the opening between kitchen and dining room.

If the soil bearing capacity is 2500 psf, then the required area of the footing under this column would be

A = 7240 lb / (2500 lb/sq.ft.) = 2.896 sq.ft.

In general, therefore, a square footing 20.5” on a side would be adequate for this purpose. Although the Drawings current reference footings that are 30 “ square, ***a more logical size would be 24” square.*** This revision is still being considered, including the effect on the footing thickness if the material is stacked plywood.